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Contents

Coleman Charles

Compaction of Sands by Vibration Alone

E. D. HONIGS, A. A. VALENTE and L. D. GRAVES, University of Notre Dame

• THIS RESEARCH was started as a continuation of the work of Lino Gomes (1). The following recommendations for future research were given in that paper.

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

The current investigators are concentrating on the first and second recommendations but hope to include some work on the third. This paper concerns itself with the first two only. Other researchers reported at the 1962 meeting that saturated sand compacts under vibration like dry sand.

APPARATUS

The compaction apparatus was constructed by attaching an aluminum plate 3.95 in. in diameter and 0.125 in. thick to the cone of a heavy-duty radio loud-speaker (Fig. 1).

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

The sand to be compacted was contained in an ordinary steel Proctor mold. This mold was supported on a hydraulic jack to permit raising and lowering the mold during the compaction process. The loud-speaker was supported over the Proctor mold in such a way that the aluminum plate could enter but not touch the mold and contact the soil as the jack was raised.

MATERIAL

Three sands, two crushed limestones, a crushed quartzite, and some plastic pellets were used. Gradations covering more than one sieve size are shown in Figure 2, whereas single sieve sizes are shown in the tables of results. The concrete sand is the same as that used by Gomes and the sizes were obtained by sieving a concrete sand from pits near South Bend, Ind., in a glacial outwash area. The silica sand is manufactured silica sand from near Ottawa, Ill. The Plymouth sand is a natural sand soil used as fill

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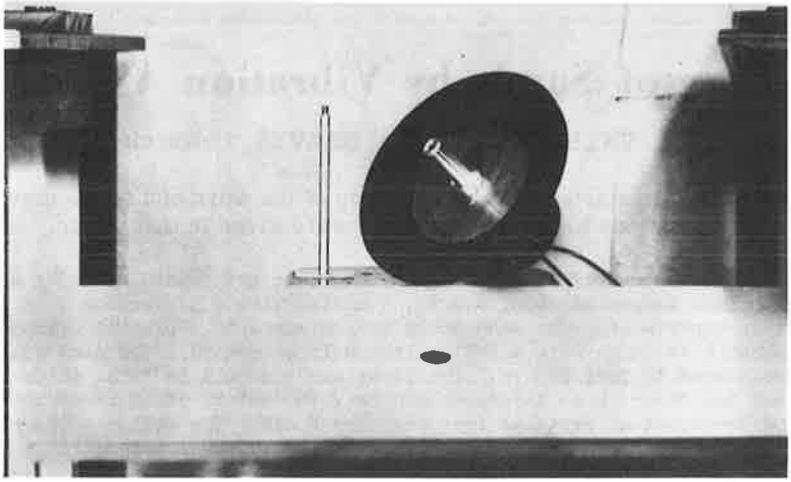

Figure 1. Details of speaker and tamper.

under a schoolhouse floor near Plymouth, Ind. The two crushed limestones are from quarries near Huntington and Pipe Creek, Ind. The quartzite and plastic pellets are from laboratory supplies.

PROCEDURE

In the compaction process 3 in. of sand was placed in the Proctor mold by pouring through a funnel with no free fall. The mold was then raised by the hydraulic jack until the metal tamping plate made contact with the sand surface. Because this contact could not be established visually, it was determined by sound. After some practice, it was easy to note the particular humming sound that came from the speaker at the point of firm but not excessive contact pressure. The load on the tamper at this point was 0.375 lb or a contact pressure of 0.03 psi. With the contact pressure maintained constant by sound, the tamper was vibrated for the chosen length of time at the chosen frequency. At the end of the vibration time, the mold was lowered away from the tamper, another inch of sand was placed through a funnel, and the vibration process repeated. Four more approximately 1-in. layers of sand were compacted in this fashion, making a total of about 6.5 in. of sand in the mold. At the end of the compaction, the mold was removed from the compaction apparatus, the collar taken off the mold, and the sand trimmed even with the mold as in the ordinary Proctor test. The mold and sand were then weighed to determine the density obtained.

TABLE 1

DENSITIES OBTAINED IN LAYERS OF CONCRETE SAND GRADING A

Trial No.	Vibration		Density (pcf) of Sand After Compaction of					
	Frequency (cps)	Time (min)	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Mold
1	25	6	115.5	116.1	119.5	119.8	119.0	116.4
2	25	6	112.5	-	-	120.0	119.2	120.0
3	25	6	118.0	119.5	119.0	119.2	121.0	119.7

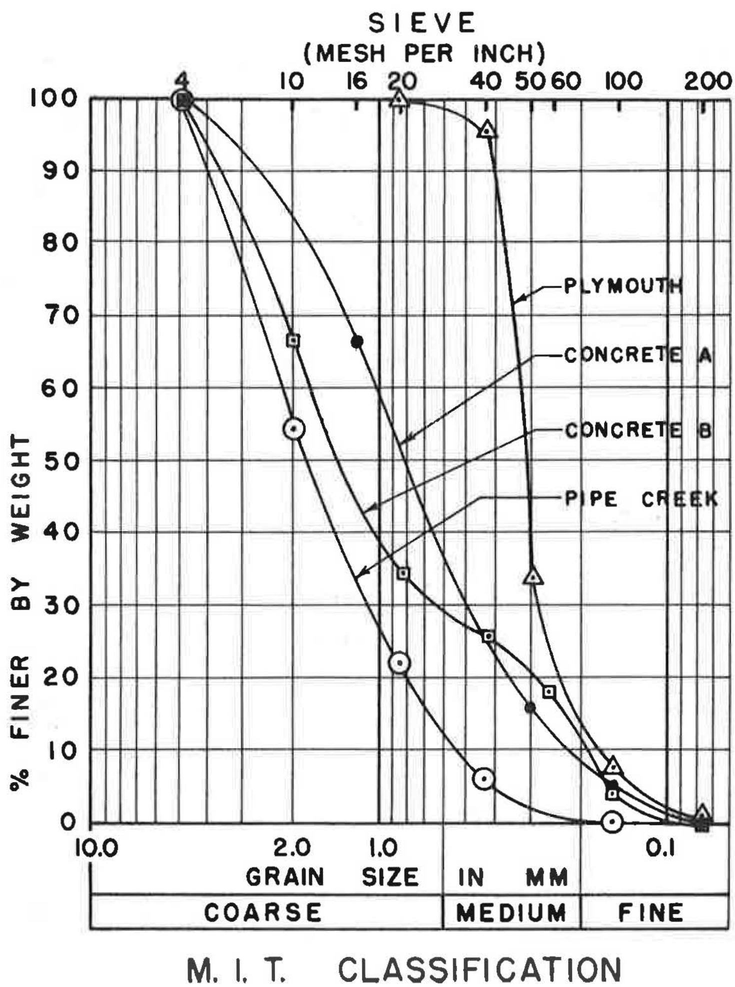

Figure 2. Mechanical analysis of sands.

RESULTS

The results of this research are given in Tables 1 through 6 and in Figure 3. Tables 1 and 2 give the results of tests made to compare the essentially two-dimensional results of the research by Gomes with the three-dimensional results of this study. The density of the soil increased as each layer was added. Also, the process of removing the collar must have loosened the sand because the density in the mold alone was less than that before the collar was removed.

Table 3 gives the results of compaction tests run at various frequencies of vibration on a few sands having essentially one-size gradings. A frequency of 25 cycles per second (cps) caused the greatest density for every sand but the variation in density was not as much as that noted by Gomes for the better graded sand. The greatest amount of energy was delivered by the tamper at 25 cps.

Table 4 gives data showing the effect of time of vibration on the density of a variety of sands and gradings. The data seem to indicate that all types and gradings of sand have about the same exponential relation between time and degree of compaction as that found by Gomes for the well-graded concrete sand. The better graded samples reached higher densities and lower void ratios than the one-size sands.

TABLE 2

DENSITIES OBTAINED IN LAYERS OF PLYMOUTH SAND

Trial No.	Vibration		Density (pcf) of Sand After Compaction of					
	Frequency (cps)	Time (min)	Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Mold
1	25	6	96.8	96.5	97.3	98.5	98.8	96.6
2	25	6	98.0	98.2	99.0	99.5	100.0	98.4
3	20	6	95.0	97.5	97.3	99.0	98.3	97.5
4	30	6	99.5	100.5	100.5	101.5	102.2	99.2

TABLE 3

EFFECT OF VIBRATION FREQUENCY ON SAND DENSITY

Sand	Grading	Vibration		Energy (in.-lb)	Density (pcf)
		Frequency (cps)	Time (min per layer)		
Concrete	20-40	25	6	302	100.8
		23.5	6	293	100.3
Concrete	40-60	25	6	302	98.7
		28	6	256	98.4
Plymouth	40-100	20	6	270	97.5
		25	6	302	98.7
		30	6	223	97.8
Plastic cubes	4-10	20	3	135	33.6
		25	3	151	34.8
		30	3	112	34.2

The data in Table 5 indicate that the particle shape and surface smoothness of one-size sands have a large effect on the void ratio reached under a vibration time of 6 min per layer and a vibration frequency of 25 cps. The data are very consistent for sands with great differences in specific gravity and significant differences in particle sizes.

The data of Table 6 show that there is no consistent relation between the specific gravity of a sand and the void ratio reached after 6-min per layer compaction at a frequency of 25 cps. In the concrete sand, gradings having the same particle shape and surface smoothness reached lower void ratios when the particle sizes were larger and sands with the same grain shape and size reached lower void ratios when the specific gravity was higher.

Figure 3 shows the results of standard Proctor (AASHO T 99, method A) compaction on grading B of the concrete sand, the graded and standard Ottawa sand, and the Plymouth sand. In addition to the normal erratic curves, the one-size gradings in the dry state reached densities higher than the vibration densities, whereas the better graded sand did not.

ANALYSIS OF RESULTS

Apparently, the conclusion by Gomes that the greatest density occurs some distance below the tamper holds true for the three-dimensional case represented by the results

TABLE 4

EFFECT OF VIBRATION TIME ON SAND DENSITY

Sand	Grading	Vibration: Frequency (cps)	Vibration: Time (min per layer)	Density (pcf)	Void Ratio
Concrete	A	25	1.0	120.9	0.32
		25	2.0	121.8	0.31
		25	3.0	123.0	0.30
Concrete	B	25	1.5	120.9	0.32
		25	2.0	121.5	0.31
		25	3.0	123.6	0.29
		25	6.0	123.9	0.29
Concrete	4 - 10	25	1.5	101.0	0.58
		25	2.0	102.0	0.56
		25	3.0	102.6	0.55
		25	6.0	102.0	0.56
Concrete	10 - 20	25	2.0	97.5	0.64
		25	3.0	99.0	0.61
		25	6.0	99.9	0.60
Concrete	20 - 40	25	2.0	99.3	0.60
		25	3.0	100.0	0.59
		25	4.0	100.8	0.58
		25	5.0	100.5	0.59
		25	6.0	100.8	0.58
Concrete	40 - 60	25	1.0	96.8	0.65
		25	2.0	98.0	0.63
		25	3.0	97.7	0.64
		25	4.0	98.0	0.63
		25	5.0	98.0	0.63
		25	6.0	98.0	0.63
Ottawa	Std.	25	1.0	104.5	0.59
	20 - 30	25	1.5	105.6	0.57
		25	3.0	106.0	0.57
		25	6.0	105.6	0.57
Ottawa	Graded	25	1.5	103.0	0.61
	30 - 100	25	3.0	103.8	0.60
		25	6.0	103.8	0.60
Plymouth	40 - 100	25	0	98.1	0.62
		25	0.5	98.7	0.61
		25	1.0	98.7	0.61
		25	1.5	99.0	0.60
		25	3.0	99.3	0.60
		25	6.0	98.7	0.61
		25	10.0	99.9	0.59
Plastic spheres	20 - 30	25	1.5	40.3	0.61
			3	43.2	0.50
			6	43.5	0.50
Plastic cubes	4 - 10	25	1.5	33.9	0.67
			3	34.8	0.62
			6	35.4	0.60

of this study. The increase of over-all density as each layer was added can be explained by the fact that as each layer was compacted the maximum density layer became a greater portion of the whole. The loosening of the sand when the collar was removed may have been due to an expansion of the sand with shear when the collar was rotated to remove it from the mold.

TABLE 5

EFFECT OF PARTICLE SHAPE ON SAND DENSITY

Sand	Grading	Specific Gravity	Particle Shape	Void Ratio
Plastic spheres	20 - 30	1.04	Spherical, smooth	0.50
Ottawa	20 - 30	2.66	Bulky, rounded	0.57
Concrete	20 - 40	2.55	Bulky, rounded	0.58
Plastic cubes	4 - 10	0.91	Cubical, smooth	0.60
Plymouth	40 - 100	2.54	Bulky, rough, rounded	0.61
Limestone	4 - 10	2.57	Bulky, sharp	0.65
Limestone	4 - 40	2.64	Flat, very sharp	0.69
Quartzite	4 - 10	2.66	Bulky, rough, sharp	0.75

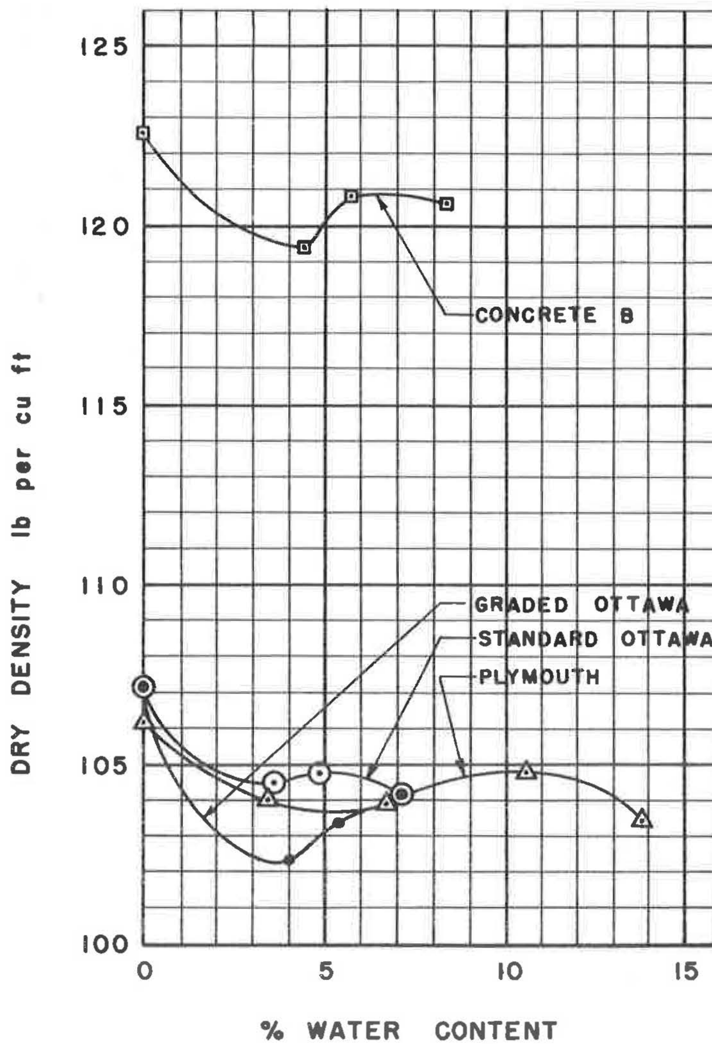

Figure 3. Proctor-moisture-density relation curves.

TABLE 6

EFFECT OF SPECIFIC GRAVITY ON SAND DENSITY

Sand	Grading	Particle Shape	Specific Gravity	Void Ratio
Ottawa	20 - 30	Bulky, rounded	2.66	0.57
Ottawa	30 - 100	Bulky, rounded	2.66	0.60
Quartzite	4 - 10	Bulky, rough, sharp	2.66	0.75
Limestone	4 - 40	Flat, very sharp	2.64	0.69
Limestone	4 - 10	Bulky, sharp	2.57	0.65
Concrete	4 - 10	Bulky	2.55	0.56
Concrete	10 - 20	Bulky	2.55	0.60
Concrete	20 - 40	Bulky, rounded	2.55	0.58
Concrete	40 - 60	Bulky, rounded	2.55	0.63
Plymouth	40 - 100	Bulky, rough, rounded	2.54	0.61
Plastic spheres	20 - 30	Spherical, smooth	1.04	0.50
Plastic cubes	4 - 10	Cubical, smooth	0.91	0.69

The energy delivered by the tamper seems to be another reason behind the optimum compaction at 25 cps rather than a simple critical frequency as the data obtained by Gomes indicated. The amplitude of the tamper became less as the vibration frequency increased varying in a hyperbolic fashion, whereas the force became less as the amplitude decreased varying in a straightline manner. This resulted in a smaller amount of energy being delivered at frequencies above and below 25 cps. However, energy alone does not seem to be the whole reason for the best compaction because the Proctor procedure with its 4,950 in.-lb of delivered energy did not compact the graded concrete sand to as dense a state as 300 in.-lb delivered by the vibration tamper. On the other hand, the one-size sands reached a greater density under Proctor compaction. Thus, factors other than total energy (such as particle gradation, shape, size, and weight as well as manner of energy delivery) must affect the density a sand reaches under compaction. Of course, the data indicate that with other factors held constant the greatest amount of delivered energy results in the greatest density. Also, the compacting effect of the vibration energy is greatest when the sand is loose and decays in an exponential fashion as the sand densifies.

When the compactive energy is held constant, the particle shape and surface roughness seem to have a greater effect on the density reached under vibration than other factors, such as specific gravity or size of sand particles. Although, when all other factors were held constant, sands with higher specific gravities tended to reach higher densities and sands with larger one-size grains also reached higher densities, the differences noted were much less than those caused by particle shape and roughness even when the specific gravity and grain size were variable. This would indicate that the densifying process under a vibrating plate depends less on the attraction of gravity (grains falling into place) than on a shearing action (grains sliding on each other).

The fact that the maximum density occurs at some distance below the plate also points to the importance of shear because that is where the maximum shear occurs. The shear concept might also be used to explain why the large energy of the Proctor procedure did not cause a lower density in the relatively easily compacted graded sand. The force applied was so large that the sand actually moved sidewise and upward along the hammer causing such a large strain that the sand was loosened from its previously compacted state. So, while the sand immediately under the hammer tended to compress to a lower void ratio, that at some depth and to the side became loosened. The net effect was a smaller density than that caused by the much smaller energy of the vibration. In the less easily compacted one-size sands, a smaller amount of excessive shear strain occurred in the Proctor compaction and more of the force was effective in compressing the sand, thus the net effect was greater than that caused by the vibration.

All of the preceding discussion leads to the theory that compaction in clean sand can best be accomplished by applying the correct amount of shearing strain. This shearing strain could be applied in many ways, such as a direct sliding or torsional strain on thin layers or one of the many forms of stressing to cause strain. It is well known that static compressional loading will not produce high densities in a confined sand because the load itself causes large normal stresses between the grains thus giving them resistance to sliding and preventing the development of sufficient strain to cause the particles to rearrange themselves in a more dense manner. Thus, it would seem wise to keep the compressional loads light to permit the necessary strains to occur more easily. This is what Gomes and the authors tried to do with their repeated application of very light loadings. It now appears that not enough strains were caused to reach the lowest possible void ratios even in the well-graded sand. (Hough's textbook reports a minimum void ratio of 0.20 for a sand like the well-graded concrete sand, whereas 0.29 was the lowest void ratio reached in this study.)

Greater strains could be caused by more applications of the loads. This was done with the Plymouth sand when it was vibrated for 10 min but this approach is inefficient because of the exponential decay in the compacting effect. Larger amplitudes of vibration would also cause more strains and more compaction as would larger tamper pressures, but amplitude, frequency, and tamper pressure were all interdependent in the equipment used in this investigation, so their effects could not be studied separately.

CONCLUSIONS

The following conclusions are made from the results of this study:

1. The vibration method of compaction as suggested by Gomes is efficient for the well-graded dry sands of this study and results in low void ratios, but not the lowest possible.
2. The density of the sand should be measured before the collar is removed from the mold, or the collar removed in such a way that the sand is not loosened.
3. The maximum density region of the sand seems to be at some distance below the tamping plate, as found by Gomes.
4. One-size sands do not compact to low void ratios under the vibration time, frequency, and amplitude used in this study.
5. Variations in frequency of vibration do not affect the resulting density as much in one-size sands as in well-graded sands.
6. The highest density in both one-size and well-graded sands was reached at the frequency and amplitude of vibration delivering the most energy to the sand.
7. The exponential relation between time of compaction and density found by Gomes applied to both well-graded and one-size sand.
8. One-size sands with rounded and smooth-surfaced grains reach much higher densities under vibration compaction than sand with sharp and rough-surfaced grains.
9. When other variables are held constant one-size sands of higher specific gravities reach slightly higher densities under vibration compaction.
10. When other variables are held constant, one-size sands with larger particles reach slightly higher densities under vibration compaction.
11. Standard Proctor compaction caused greater compaction in the one-size sands than the vibration compaction but not in the well-graded sands.
12. Standard Proctor compaction caused greater densities in dry sands than in sands having water contents up to 14 percent.
13. The densifying process in a sand under a vibrating plate depends more on particles sliding on each other than on particles falling into place from gravity.
14. The vibration frequency, amplitude, and contact pressure used in this study did not cause the lowest possible void ratios in the sands because they did not cause the right amount of shearing strain to occur.

FUTURE RESEARCH

This study is continuing and equipment is being designed in which vibration amplitude, frequency, and contact pressure can be controlled independently. Thus, total energy delivered and manner of delivery can be controlled. With this equipment the effect of amplitude will be evaluated first because it seems the most promising; then further work will be done on frequency and contact pressure. Should this repeated compression approach fail to produce maximum possible densities, a direct torsional shear strain approach will be tried.

REFERENCES

1. Gomes, L., and Graves, L., "Stabilization of Beach Sand by Vibrations." HRB Bull. 325, 44-54 (1962).
2. Felt, E. J., "Laboratory Methods of Compacting Granular Soils." ASTM Special Tech. Publ. 239 (1958).
3. "Relative Density of Cohesionless Soils." Earth Manual, 1st Ed., U. S. Bureau of Reclamation.
4. Hutchinson and Townsend, "Some Grading-Density Relationships for Sand." Proc., 5th Internat. Conf. on Soil Mechanics and Foundation Engineering (1961).
5. Winterkorn, H. F., "Macromeritic Liquids." Symposium on Dynamic Testing of Soils, ASTM Special Tech. Publ. 156.
6. Gumenski, B. M., and Komarov, N. S., "Soil Drilling by Vibration." Translated from Russian by Consultants Bureau, N. Y.
7. Berger, L., "Effect of Static Loading and Dynamic Forces on the Density of Cohesionless Soils." Unpublished thesis, M. I. T. (1940).
8. Johnson, A. W., and Sallberg, J. R., "Factors Influencing Compaction Test Results." HRB Bull. 319 (1962).

Discussion

W. H. CAMPEN, and L. G. ERICKSON, Omaha Testing Laboratories, Omaha, Neb.— The authors have done work along a line that deserves immediate attention. Many who furnish control service in connection with construction of embankments, trench backfills and pavement subgrades, subbases and bases are very well aware of the fact that impact methods used for establishing maximum laboratory density are not generally satisfactory for cohesionless mixtures. A method is therefore needed for evaluating such mixtures.

Based on the writers' research and that of others (1), the conclusion has been reached that eventually a method will be developed based on inundation plus vibration. Apparently a small, low-energy vibrator will suffice.

With such a method, it will be necessary to distinguish between truly cohesionless and borderline mixtures. There are mixtures that show no plastic limit by test but contain enough clayey or plastic particles to prevent consolidation by inundation plus vibration. Such mixtures can be evaluated by impact methods.

REFERENCE

1. D'Appolonia, E., ASTM Special Publ. 156.

Effect of Vibration on Soil Properties

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This study presents the response of various gradations of a clean, granular sand to vibratory pressures and motions. The research is based primarily on the assumption that subgrade settlement is attributable to a microseismic vibratory phenomenon caused by vehicular traffic vibration.

The effect of vibration on the settlement, density, and vibration pressure of soil was determined for various frequencies, amplitudes, moisture contents, and soil gradations. The results indicate that a relatively insignificant vibratory motion can cause a large volume change in granular soils.

The effect of vibration of soil on the amount of vibration pressure transmitted through the soil was investigated. The variation in the magnitude of the vibratory pressure of the granular soil indicated a critical frequency or resonance of the soil particles. Further tests indicated the independence of the critical frequency from the loading condition.

• A RECENT comprehensive report on the factors affecting the compaction of soils indicates that vibratory compactors may compact certain types of cohesionless soils to a much greater unit weight and in thicker lifts than heavier static rollers (8). Therefore, it is possible that some soils compacted to specification density by heavy static rollers during construction might, after the roadway is opened to traffic, settle due to the vibration of the lighter, but still quite heavy, dynamic truck loading. Additional settlement might also occur as a result of dynamic precompaction of subsoil by a vibratory compactor at a less effective frequency than the frequency of the in-service vehicular traffic vibrations. The possibility of this additional compaction is magnified by the increased speeds and loads of present-day vehicles. Vibratory settlement and the practice of dynamic precompaction of subsoils have stimulated research in the field of soil dynamics. It is becoming apparent that not only static but also dynamic loads must be considered in the design of highway soil structures.

The basic dynamic soil properties must be isolated, understood, and evaluated. Such dynamic soil values as modulus of elasticity, energy dissipation, and resonance phenomena are felt by some authors to be fundamental in predicting the behavior of soils subjected to dynamic loads (3). It has also been noted by others that vibratory loads reduce the angle of internal friction and transform a soil into a viscoelastic medium (1). However, there is some doubt as to the nature of the most fundamental soil properties and how these properties are affected by vibration. After a review of the literature on soil vibration, it is apparent not much "basic" research has been done and, as a result, more studies should be made to isolate and examine the basic dynamic soil properties.

This paper reports the results of an evaluation of the fundamental properties that govern the response of soils to vibratory loads. The objective was to study the changes in soil properties caused by soil pressure waves, with particular emphasis on the effect of soil waves adjacent to rigid highway structures. The first step necessary to accomplish this objective has been an evaluation of the fundamental soil properties that govern the response of soils subjected to vibratory loads. The second step in this research,

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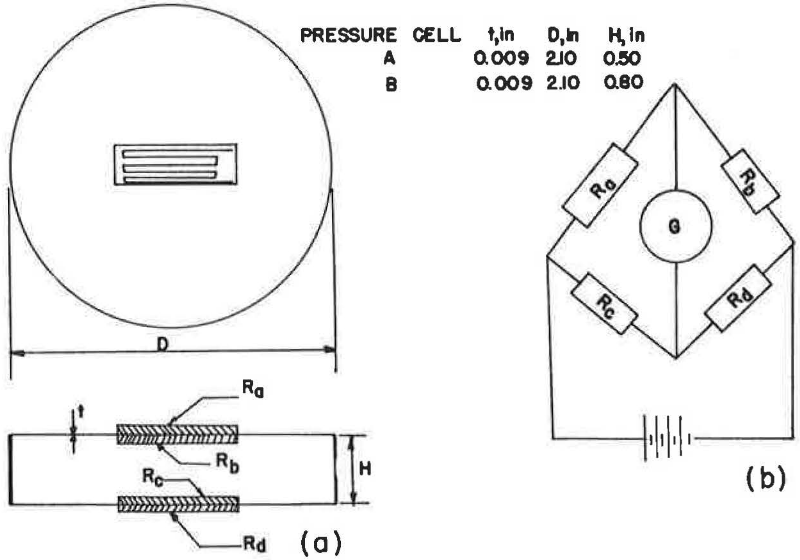


Figure 1. Construction and instrumentation details of diaphragm-type pressure cells: (a) pressure cell; (b) instrumentation diagram.

which is now in progress, is an attempt to correlate these changes in soil properties with the effects of highway traffic. The objectives of this research are based primarily on the assumption that subgrade settlement might be attributed to a microseismic vibratory phenomenon caused by vehicular traffic vibration.

EQUIPMENT AND PROCEDURE

The type of test instrumentation was very important in this study because the settlement, deformation, and pressure-time relationships were necessary. Therefore, not only the magnitude of the settlement, pressure, and deformation were necessary but also their relationship to time and to forces that are a function of time.

Soil Pressure Cells

Two types of diaphragm pressure transducers were built to measure the soil pressure. Each pressure gage was instrumented with four bonded resistance-type strain gages which made up the four legs of one Wheatstone bridge. Two strain gages were located on the interior of the pressure cell diaphragms and two on the exterior of the pressure cell diaphragms. This provided a pressure cell four times as sensitive as a typical diaphragm-type cell with one strain gage. This extreme sensitivity was necessary to measure the small soil pressures studied in this research.

Figure 1 shows the dimensions of each type of cell, the strain gages, and the location of these gages in a Wheatstone bridge. These two cells have approximately the same dimensions; however, the amount of fixity of diaphragms and the difference in materials resulted in a much different calibration factor. The calibration curves are shown in Figures 2 and 3 for pressure cells A and B, respectively.

Strain Equipment

The strain readings were recorded by a Sanborn strain gage preamplifier (Model 150-1100) used in conjunction with a Model 150-400 amplifier and a model 152 direct-writing recorder. This equipment produces a continuous record of strain for which the time base can be varied by the speed of the recording chart paper. The speeds available vary from 0.25 to 100.00 mm per sec. A 1-sec timer was used to actuate an event marker on the edge of the record for an accurate time base. The amplification of the strain can be increased to more than two chart divisions per microinch per inch of strain.

Direct Shear and Triaxial Equipment

Modifications were made on the standard direct shear and triaxial equipment to provide a dynamic pressure condition. In each case the type of dynamic pressure was different; however, it is possible that either condition might be applicable to the in-service vibratory condition resulting from vehicular traffic.

Direct Shear.—The dynamic or vibratory pressure was exerted in the direct shear test by varying the normal load. A small electric motor with an eccentric weight was placed on the normal load hanger of the direct shear equipment (Fig. 4). Both the amount of the vibrating load and the frequency could be varied by changing the amount of the eccentricity and the speed of the motor. This vibratory force was superimposed on the usually constant normal load. The rate of application of the shear load remained constant as in the static test. The remainder of the procedure used in this experiment was identical to that used in a static direct shear test (9).

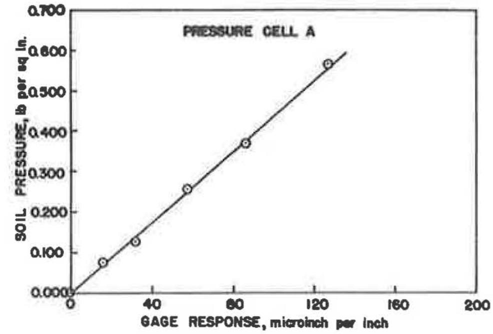


Figure 2. Response of pressure cell A.

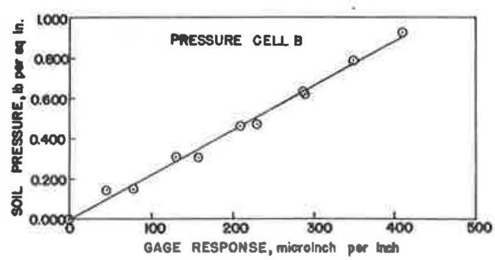


Figure 3. Response of pressure cell B.

Triaxial Test.—In the triaxial test, the hydrostatic pressure which is exerted laterally on the specimen and also adds to the normal applied load was modified to be applied dynamically. This was done by the use of an underwater loudspeaker. This device was coupled with the hydrostatic pressure fluid by a length of $\frac{1}{4}$-in. tubing and is shown mounted with the triaxial equipment in Figure 5. The power was supplied to the loudspeaker by means of an audio-oscillator and amplifier. In this experiment the

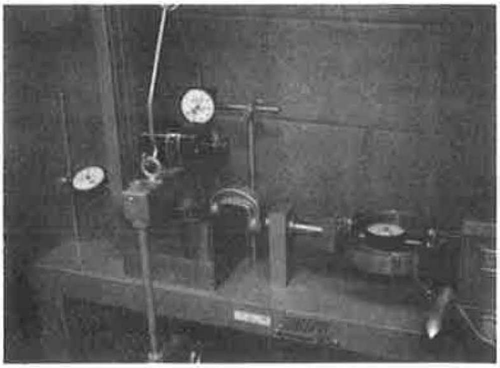


Figure 4. Direct shear equipment with pulsating normal load.

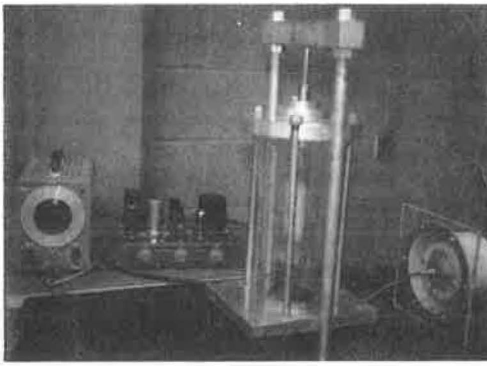
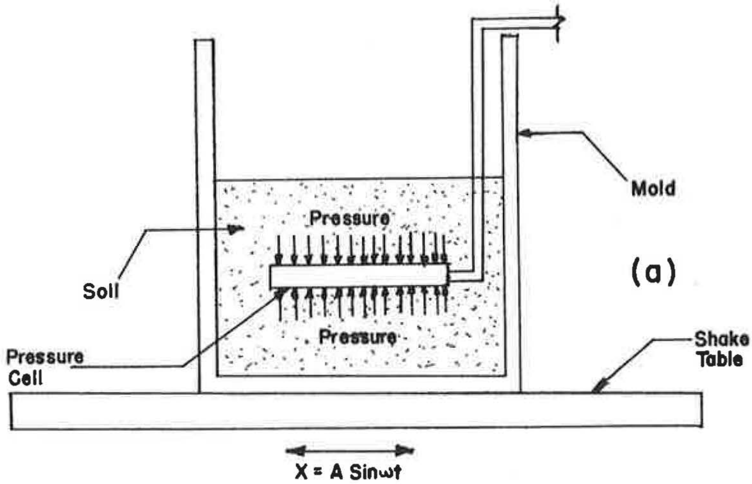


Figure 5. Triaxial equipment with pulsating hydrostatic pressure.



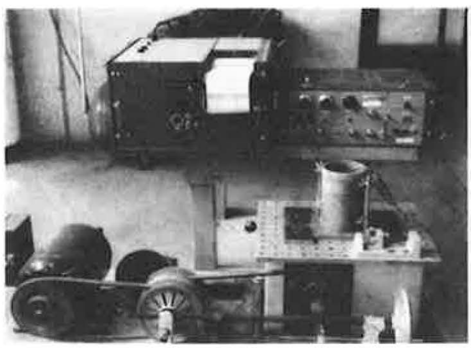


Figure 6. Vibrating table used for settlement, density, and vibration pressure tests: (a) schematic of vibratory motion and recorded pressure; (b) vibration equipment.

pulsating vibratory pressure was superimposed on both the lateral or confining pressure and on the normal applied load. This experiment was conducted as a typical triaxial test in which a precast specimen is placed in the chamber and tested (9).

Settlement, Density and Vibration Pressure Equipment and Procedures

The effect of vibration on the settlement, density, and vibration pressure of soil was determined by the use of a vibrating table (Fig. 6). This vibrating table is motor driven in the horizontal plane by an eccentric drive. The amplitude is variable from 0 to 0.25 in. and the frequency can be varied from approximately 20 to 90 cycles per second (cps). Therefore, a relatively large range of amplitudes and frequencies is available.

Soil Settlement.—The effect of vibration on the settlement of soil was determined for a vibratory motion of the soil. That is, the soil was placed in a $^1/_{30}$-cu ft mold with a surcharge weight on the soil surface and the entire mold vibrated horizontally. The soil was placed in the mold at a minimum relative density and during vibration the settlement was determined by continuously measuring the height of the soil surface. The densities were then computed by using the known weight and volume of soil. The relative minimum density of the soil at the beginning of the test was obtained by placing a known weight of material in the mold loosely and then inverting the mold once very slowly. The resulting soil was considered to be at the minimum relative density. The surcharge weight was then placed on the soil surface and the vibration test begun.

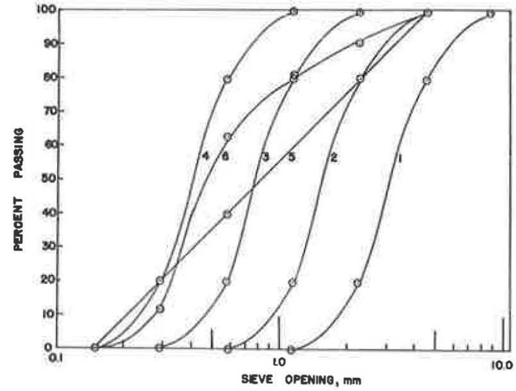


Figure 7. Soil gradations.

Vibration Pressure.— The effect of vibration of soil on the amount of vibration pressure transmitted through the soil was investigated by a method very similar to the previous settlement procedure. The mold was filled with the soil and a soil pressure cell was placed in the soil near the bottom of the mold. The entire system of the mold, pressure cell, and surcharge weight was then attached to the vibrating table. The vertical pressure of the soil and surcharge weights was recorded continuously as the frequency of the horizontally vibrating table was varied. The frequency was recorded on the soil pressure chart so that a permanent record of the soil pressure oscillations vs soil vibration frequency was obtained.

Soil Investigated

The soil investigated was a clean, medium sand and therefore, the results represent the effect of vibration on a granular type of soil. This sand is a natural, subrounded, washed material and represents an ideal granular soil. The gradation curve for the natural sand used is indicated by curve 6 on the gradation curve (Fig. 7). The other soil gradations used in this study were obtained by artificially grading this natural sand. These other gradations (curves 1 through 5) were used to determine the effect of gradation on the susceptibility of granular soils to vibration.

TEST RESULTS

Apparent Cohesion and Internal Friction

A good indication of the susceptibility of soil to vibration can be obtained by determining any change in the basic soil properties which occurs when the soil is tested under vibratory conditions. However, the nature of the vibratory condition may affect the results. Therefore, a study of the dynamic response of the "apparent cohesion" and the "angle of internal friction" was made by two different methods. These two methods (the triaxial and the direct shear tests) are commonly used to obtain the basic properties of soils for the usual static condition.

Triaxial Test.— In the triaxial test, samples of soil gradation No. 6 were compacted in a Harvard miniature compaction mold at the optimum moisture content and then stored in a 100 percent humidity moisture room for 24 hr.

The specimens were then placed in the triaxial chamber and tested in a wet but yet unsaturated condition. The tests were conducted by the usual triaxial procedure (9). The only variation required to conduct the dynamic tests was the addition of a vibratory pressure which was applied to both the lateral and normal stresses by means of pulsing the hydrostatic pressure fluid. This vibratory pressure was approximately ± 0.050 psi at a frequency of 50 cps.

Figure 8 shows the results of the triaxial test. The failure envelope for the static tests is indicated by the solid line, and the failure envelope for the dynamic tests is indicated by the dotted line. The dynamic tests were not continued beyond the pressure shown due to the limitations of the underwater loudspeaker. The angle of internal friction determined from the static test results was 30.5^0 and the effective cohesion 5.25 psi. In the dynamic test the effective cohesion was reduced to zero and the angle of internal friction increased to 37.5^0.

Direct Shear Test.—The direct shear test was conducted on soil samples of gradation No. 6 compacted in the direct shear box at an optimum moisture content of approximately 6 percent. The compaction was obtained by a static force of 1,000 lb applied as a normal force on the soil sample. The same degree of compaction was therefore not

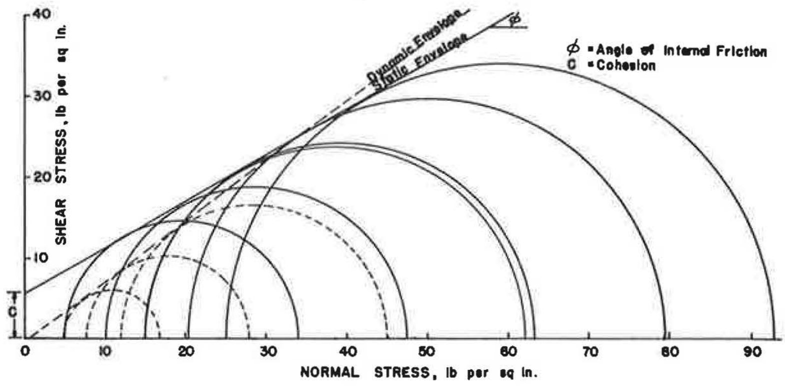


Figure 8. Failure envelope for triaxial test.

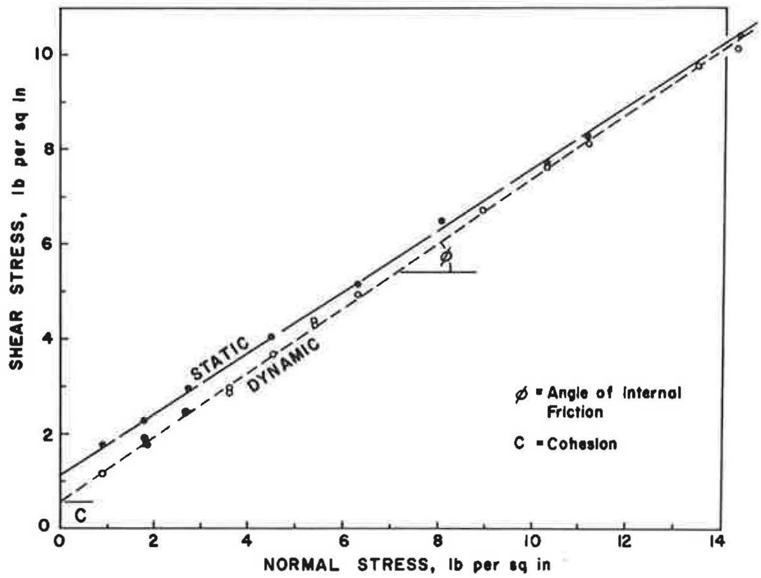


Figure 9. Failure envelope for direct shear test.

necessarily obtained on the direct shear and triaxial samples. Moreover, the direct shear specimens were used immediately after compaction. The testing of the specimens for the direct shear tests was done by the usual procedures except in the case of the dynamic tests where a vibratory pressure was applied to the normal load by means of a small motor with an eccentric weight.

Figure 9 shows the results of the direct shear tests. The vibratory pressure used in this test was approximately ± 0.150 psi at a frequency of 58 cps. The resulting angle of internal friction and effective cohesion determined by the dynamic test was 35.6^0 and 0.55 psi, respectively. The corresponding static tests resulted in an angle of internal friction of 33.4^0 and an effective cohesion of 1.1 psi. The dynamic test results, indicated by the dotted line, show the increase in the angle of internal friction and the decrease in the effective cohesion which resulted from the direct shear test.

Settlement and Density

The effect of vibration on the settlement and compaction of soils has resulted in a large amount of interest in soil vibration. This portion of the study was made to help

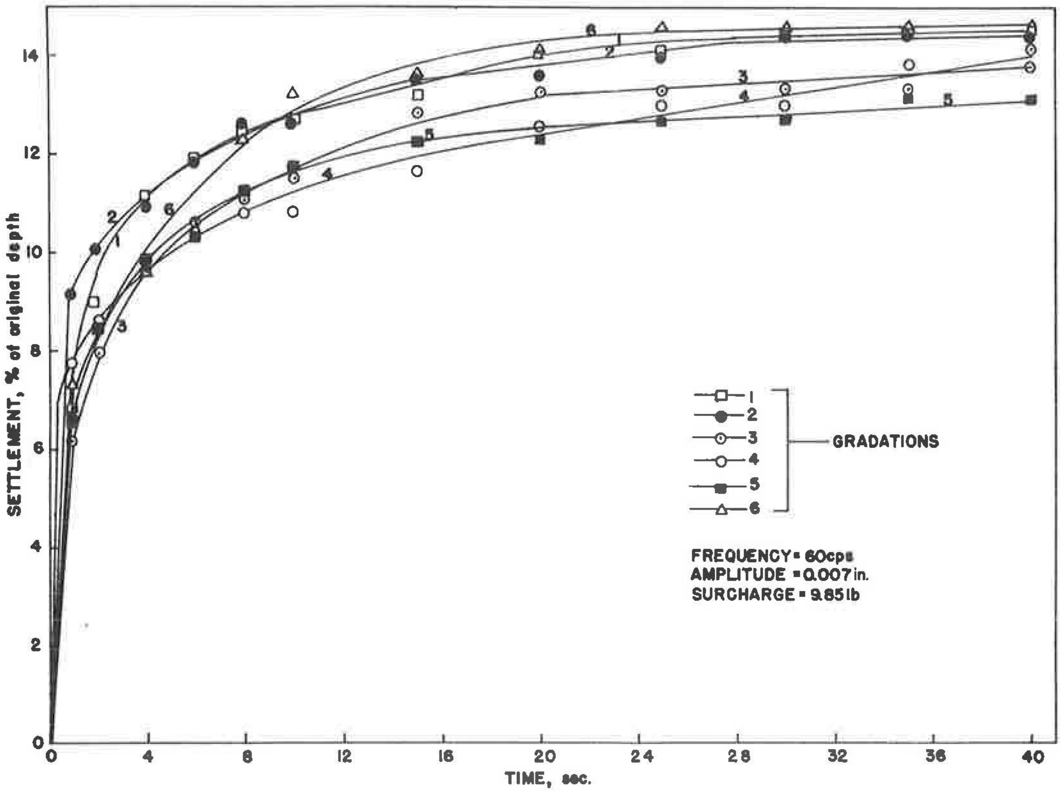


Figure 10. Time-settlement relationship for each soil gradation.

isolate the basic parameters that might affect soil settlement and compaction by vibration.

The susceptibility of a soil to vibratory loads is exemplified by the volume change relationships of the soil. These relationships can be determined in a number of ways. In this research the volume change was measured by the change in height of the sample as a percentage of the original height and also in absolute terms, as the dry density and the change in dry density of the soil. Different gradations of soil were studied to determine the effect of soil gradation, frequency of vibration, amplitude of vibration, time, and moisture content on the dynamic volume change relationships of soils.

Vibration.— The effect of vibration on soil settlement was determined by both the percentage settlement and the change in dry density relationships previously discussed. The soil was vibrated by a horizontal motion and the resulting vertical settlement of the soil recorded continuously during the vibration. The resulting time-settlement and time-dry density curves are shown in Figures 10, 11, 12, and 13. The largest settlement and change in density occurred during the first 10 sec of vibration. After 30 to 40 sec of vibration, the settlement and density reached their maximum values. This was true of all frequencies, amplitudes of vibration, and soil gradations.

Gradation.— To isolate the effect of soil gradation on the settlement response of soils to vibration, four gradations of soil (1 through 4 in Fig. 7) were investigated. The gradation curves for these soils are parallel, and for any given percent passing the particle size is double that of the next smaller gradation. The uniformity coefficient for these gradations are the same (1.88). The effective soil particle size is 1.80, 0.90, 0.46, and 0.23 mm for gradations 1, 2, 3, and 4, respectively.

These four gradations were studied to obtain the characteristics of each when subjected to a vibratory motion. The variations in density of each gradation present at the

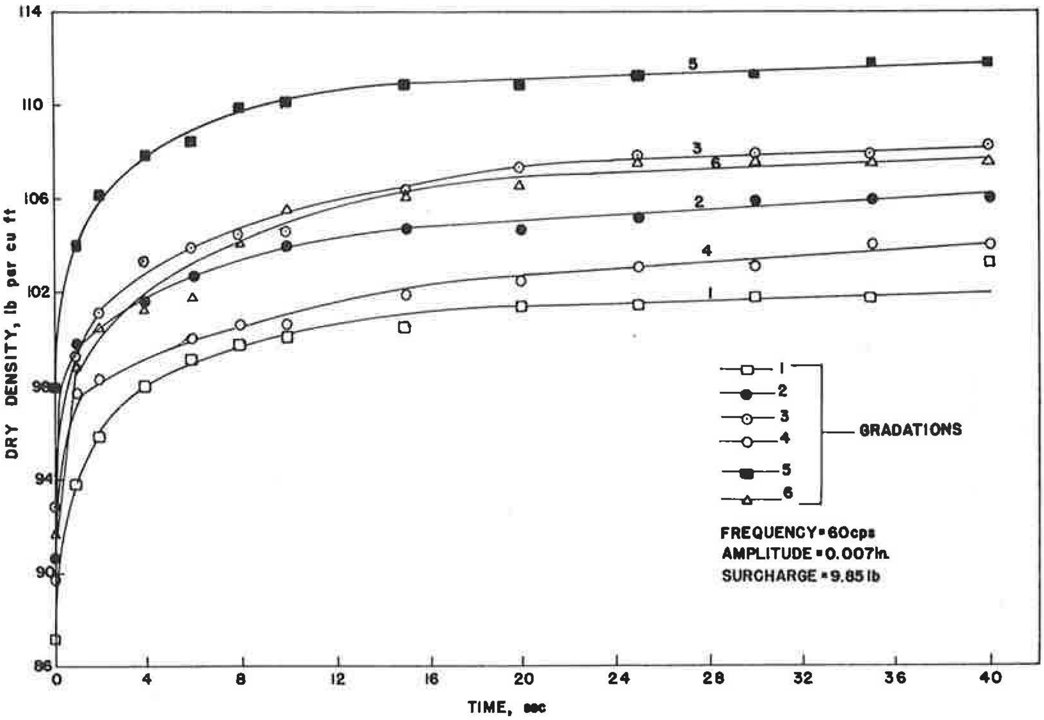


Figure 11. Time-dry density relationship for each soil gradation.

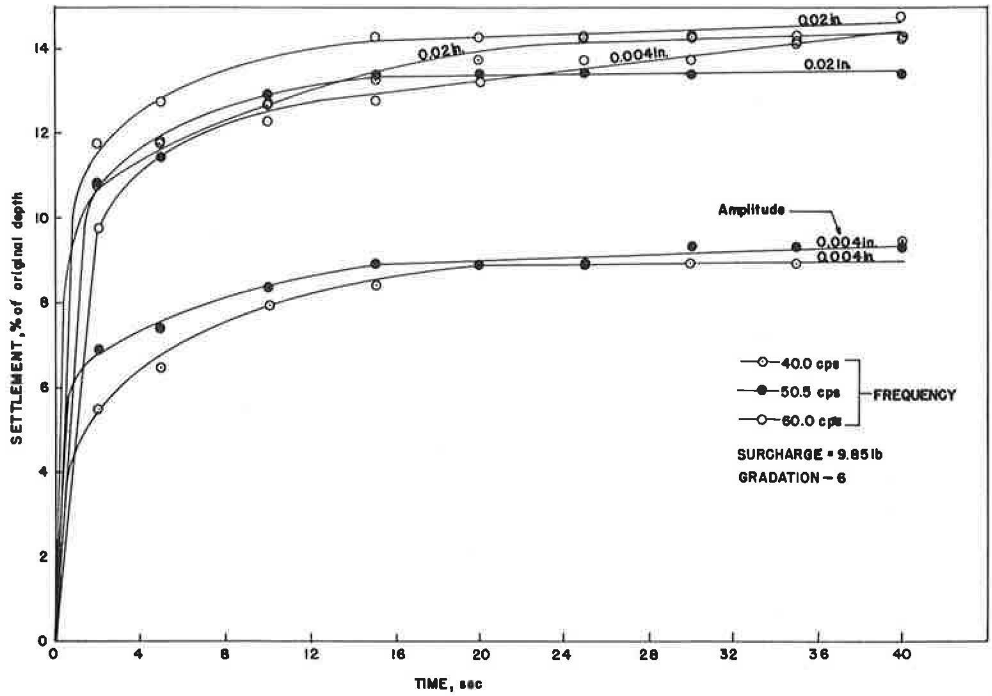


Figure 12. Time-settlement relationship for different amplitudes and frequencies.

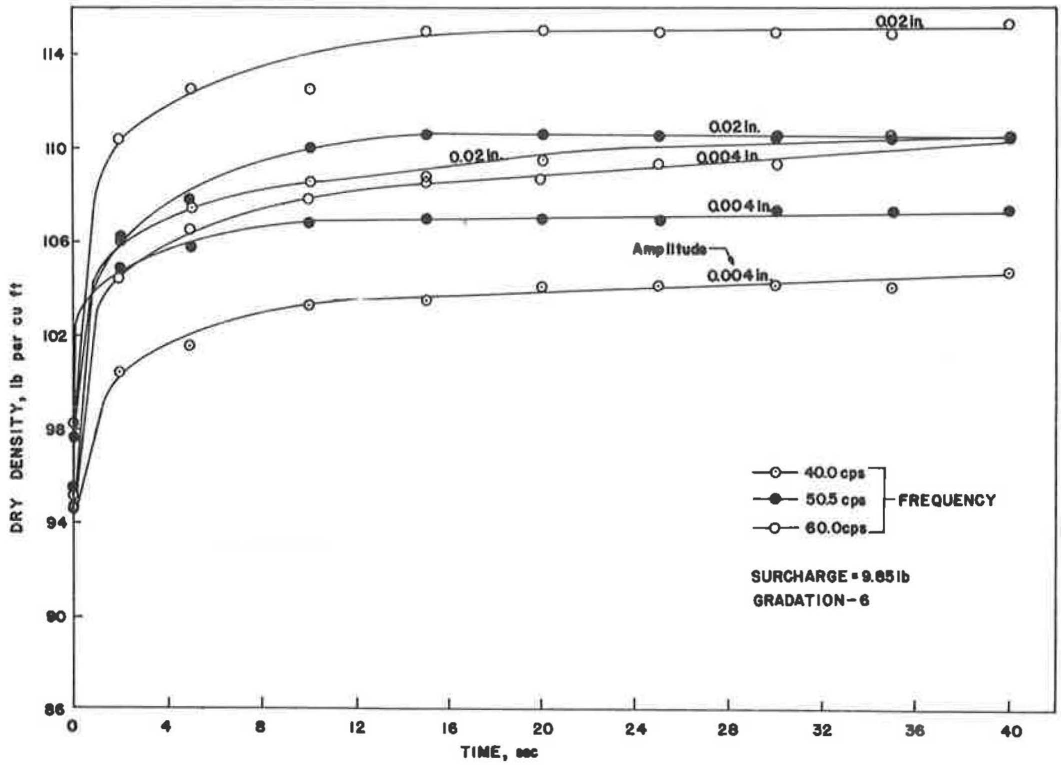


Figure 13. Time-dry density relationship for different amplitudes and frequencies.

beginning of the vibration were also present during and following the vibration. The solid line curves in Figure 14 show the dry densities for each gradation at the beginning of the test and at the end of 10, 20, 30, and 40 sec. The percent change in dry density is shown by dotted lines and indicates that the coarsest material had the greatest change in density.

The effect of the uniformity of gradation of soil could not be established because there was not enough variation in the uniformity coefficients of the soils studied. However, the variation of the uniformity coefficients of gradations 5 and 6 (5.55 and 2.03, respectively) is large enough to obtain an indication of this effect. The response of these two gradations indicates that the gradation with the largest uniformity coefficient (gradation 5) obtained the greatest density, as might be expected. This is shown in Figure 11 by the position of curves 5 and 6. However, the greatest change in density or percent settlement occurred in soil 6, as shown by the position of curves 5 and 6 in Figure 10.

Frequency and Amplitude.—The results of the second series of tests to study the effect of frequency and amplitude are shown in Figures 12 and 13. In this study, only the natural gradation was studied (gradation 6). The effect of frequency and amplitude is very apparent in these results. Figure 12 shows that significant changes in settlement occurred as the amplitude of vibration was increased. However, it is apparent that both amplitude and frequency affected the settlement and density. The 60-cps vibration was much more effective in settling the soil, even at the small amplitude, than was either the 40- or 50-cps vibration. This significant difference, as shown later, is the result of a resonance in the soil at this frequency. This indicates that even small amplitudes of vibration can produce large settlements if the frequency of vibration is near a resonant frequency of the soil.

Frequency-Dry Density Relationship.—The effect of the frequency of vibration on the dry density was studied for the natural soil gradation (gradation 6). The effect of

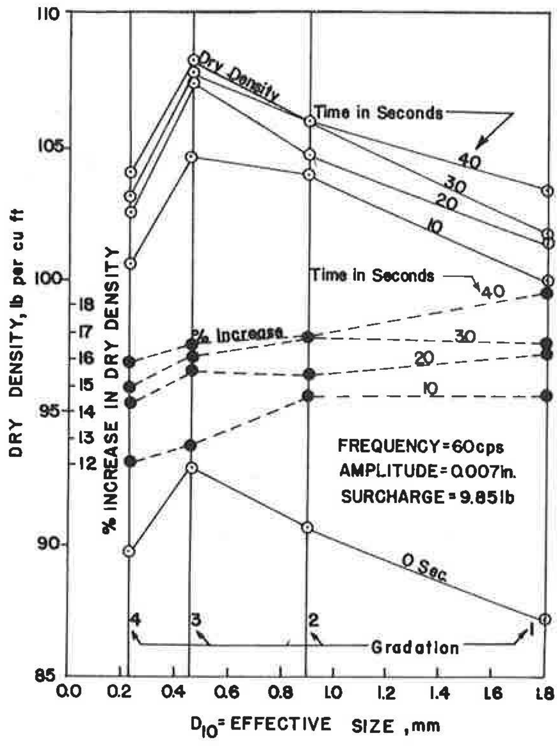


Figure 14. Soil size-density relationship for each gradation including effect of time.

the frequency of vibration on the resulting dry density of the soil is shown in Figure 15. These results indicate the increased effectiveness of the critical frequencies in compacting soil. The critical frequency of the soil, or the resonance condition, is indicated by an abrupt change in density. Moreover, the smaller amplitudes of vibration were affected more by the critical frequency but less abruptly than the larger amplitudes.

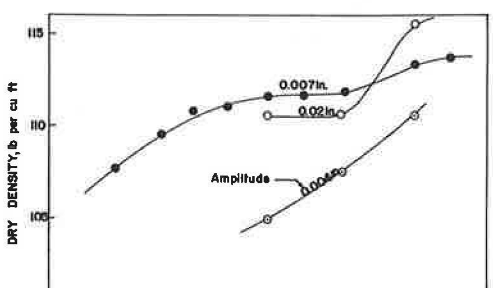


Figure 15. Effect of vibratory frequency on soil density for various amplitudes.

Moisture Content.— The previous studies of the effect of vibration on density were conducted with the soil in a dry condition. The moisture content of the soil was approximately 0.5 percent during all the previous tests. As a result, a study was made to determine the relationship between moisture content and density for a soil compacted by vibration. This relationship was obtained by placing the loose moist soil, at each moisture content, in a $\frac{1}{30}$-cu ft mold without any surcharge weight. The mold and soil were then vibrated at a frequency of 60 cps and an amplitude of 0.003 in. for 40 sec. This test was run in conjunction with a Harvard miniature compaction test (16) and the results are shown in Figure 16. The Harvard miniature apparatus had been previously correlated with the standard AASHO compaction procedure for this soil. Therefore, a standard AASHO moisture-density curve is also shown for this soil (Fig. 16). The maximum dry density resulting from the vibratory compaction is 100 pcf at a moisture content of 5.2 percent. The Harvard miniature curve is very

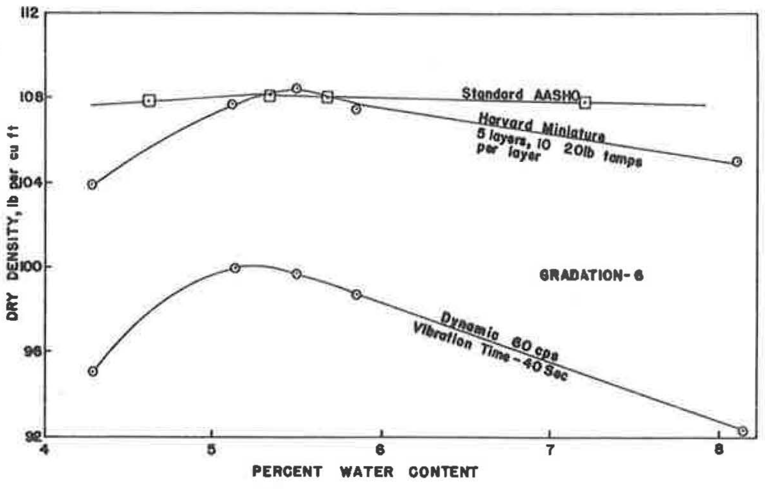


Figure 16. Dynamic and static moisture-density relationships.

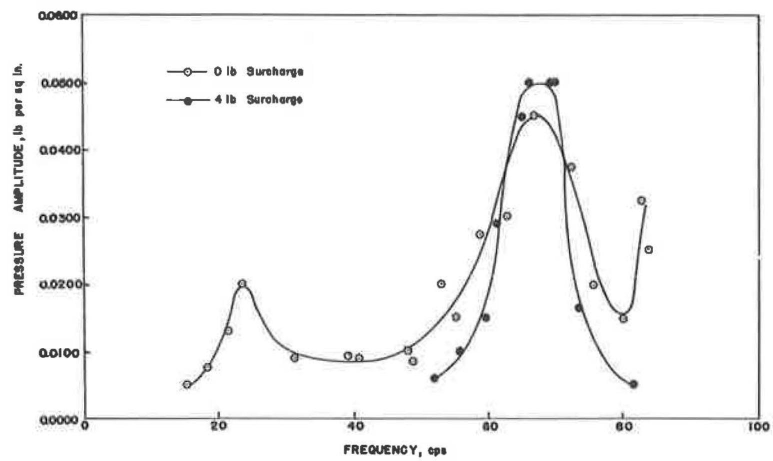


Figure 17. Vibratory pressure characteristics of natural soil (gradation 6).

similar to the vibratory compaction curve, except that a maximum dry density of 108.3 pcf was obtained at a moisture content of 5.5 percent. The standard AASHO moisture-density curve has a similar maximum dry density; however, the curve is much flatter and is almost horizontal for this soil for a moisture content of 5 to 8 percent. These results, primarily the form of the curve, indicate a correlation between vibratory compaction and the Harvard miniature compaction results.

Vibration Pressure

Many investigators have studied the resonance condition in soils by the use of vibrators placed on the surface of the soil. The response of the soil, in this type of analysis, is dependent on the kind of soil, as well as the weight and dimensions of the vibrator. However, it is reasonable to assume that soil particles have a resonance condition that is a function of the gradation, effective size, and type of soil. Therefore, a study was made of the effect of vibration on a soil that would yield results dependent only on the properties of the soil. The resulting procedure was an attempt to isolate this effect by a vibration pressure study of the soil. In this procedure the soil was placed in the $^1/_{30}$-cu ft mold and vibrated horizontally while the pressure cell measured the vertical pressure of the soil. Figure 6 shows the direction of vibratory motion of the mold and soil and the type of soil pressure measured by the pressure cell. The amplitude of the soil vibration pressure is shown in Figure 17 for various frequencies of vibration. Maximum vibration pressures occurred at two frequencies, and possibly a third. However, above 80 cps the vibrating table had a large amount of random vibration and, therefore, frequencies near and above 80 cps are very inconclusive.

The addition of surcharge weights to the vibrating soil system resulted in an increase in the maximum pressure amplitude at the resonant frequencies with very little, if any, change in the resonant frequency. This indicates the independence of the resonance or critical frequency from the loading conditions.

SUMMARY AND CONCLUSIONS

The application of any results obtained from this research must be tempered by the knowledge that the material used in these experiments was a clean granular sand. In addition to this, these experiments and tests were performed to obtain a comprehensive qualitative indication of the response of soils to vibratory pressure, and are not nearly complete enough to yield quantitative results. A complete investigation of the variables considered in this study would entail a much larger program than the current project. However, the complexity of the problem indicates that there is a necessity for basic research in the area of soil vibration. It is also very important to determine the type of vibration that results from vehicular traffic loadings to apply any results obtained from a basic research program properly.

Apparent Cohesion and Angle of Internal Friction

Two experimental procedures were used to determine the effect of vibratory loads on the basic constants of cohesion and friction angle, which make up Coulomb's Law. These two properties are basic in evaluating a soil.

The vibratory normal load in the direct shear test resulted in an increase in the angle of internal friction and a decrease in the apparent cohesion, similar to the effect of the vibratory pressure in the triaxial test results. The vibratory pressure caused a premature failure in both cases, with a decreasing effect as the normal and applied pressure increased. This probably resulted from the ratio of dynamic to applied pressure decreasing and therefore reducing the significance of the superimposed vibratory pressure.

These results are indicative of the variation in effect which different types of vibratory loadings had on the basic soil properties. From these results it is apparent that the type of vibratory pressure must be known before its effect can be predicted. This will be especially important in the case of vehicular traffic vibrations.

Vibratory Volume Change

The effect of a vibratory motion on the volume change characteristics of a granular soil has been studied for various frequencies, amplitudes, moisture contents, and soil gradations. These results indicate that a relatively insignificant vibratory motion can cause a large volume change in granular soils.

The maximum density occurred in the soil gradation with the largest uniformity coefficient, although the largest change in density occurred in the soils with the smaller coefficient. For the soil gradations with the same uniformity coefficient, the largest increase in density occurred in the soil with the largest grain size.

An increase in the amplitude of vibratory motion increased the settlement and maximum density of the granular soil, although the increase in density was small compared to the increase in the amplitude used. These results seem to indicate that soils will respond to very small amplitudes of vibration with changes in density almost as large as those resulting from larger amplitudes of vibration.

The vibration frequency affected the soil to a much greater extent than the vibration amplitude. The susceptibility of soils to certain frequencies is very evident and indicates the importance of determining the critical frequency of the soil per se. The resonant condition of a certain loading cannot be construed to be the critical frequency of the soil. Moreover, when the soil was vibrated at its critical frequency, the effect of moisture was consistent with the usual moisture-density relationship. However, this relationship cannot be compared with the vibration of soil in the quick condition because no correlation of this type was made.

Soil Vibration Pressures

A knowledge of soil vibration pressures could be very important in the study of active soil pressures in highway soil structures. The pressure that results from the vibratory motion of the soil might provide an important insight into the transmission of live load soil pressure through a soil mass.

The variation in the magnitude of the vibratory pressure of the granular soil which occurred for different frequencies of soil vibration indicates a critical frequency or resonance of the soil particles. The natural soil gradation used indicated two well-defined critical frequencies at 24 and 67 cps. The resonance at 67 cps produced a vibratory pressure approximately double the pressure at the lower critical frequency. Moreover, the soil at its lower critical frequency exhibited a "beat-frequency" type of maximum pressure. This might indicate that the lower critical frequency is not the fundamental frequency, or it might be the result of a resonance based on a particular soil particle size. Additional research on the fundamental frequency of soil particles, and on the cause and effect of soil resonances is necessary for an evaluation of this phenomenon.

REFERENCES

1. "Symposium on Dynamic Testing on Soils." ASTM Special Tech. Report 156 (1953). (Complete bibliography of publications on soil dynamics.)
2. Alai, J. A., "Testing Procedures for Model Footings and Presentation of Tradex Data." Symposium on Soil Dynamics, ASTM, 64th Annual Meeting (June 1961).
3. Bernhard, R. K., "On Biaxial Stress Field in Noncohesive Soils Subjected to Vibratory Loads." Symposium on Soil Dynamics, ASTM, 64th Annual Meeting, (June 1961).
4. Casagrande, A., and Shannon, W. L., "Strength of Soils Under Dynamic Loads." Proc., ASCE, 74:No. 4 (April 1948).
5. Converse, F. J., "Stress-Deformation Relations for Soft Saturated Silt Under Low-Frequency Oscillating Direct Shear Forces." Symposium on Soil Dynamics, ASTM, 64th Annual Meeting (June 1961).
6. Cunny, R. W., and Sloan, R. C., "Dynamic Loading Device and Results of Preliminary Small-Scale Footing Tests." Symposium on Soil Dynamics, ASTM, 64th Annual Meeting (June 1961).
7. Durelli, A. J., and Riley, W. F., "Performance of Embedded Pressure Gages Under Static and Dynamic Loadings." Symposium on Soil Dynamics, ASTM, 64th Annual Meeting (June 1961).
8. "Factors That Influence Field Compaction of Soils." HRB Bull. 272 (1960).
9. Lambe, T. W., "Soil Testing for Engineers." Wiley (1951).
10. Shenkman, S., and McKee, K. E., "Bearing Capacities of Dynamically Loaded Footings." Symposium on Soil Dynamics, ASTM, 64th Annual Meeting (June 1961).
11. "Non-Destructive Dynamic Testing of Soils and Highway Pavements." HRB Bull. 277 (1960).
12. Sinnamon, G. K., and Newmark, N. M., "Facilities for Dynamic Testing of Soils." Symposium on Soil Dynamics, ASTM, 64th Annual Meeting (June 1961).
13. "Soils." ASTM Special Tech. Bull. 206 (1957).
14. Tschebotarioff, C. P., "Tschebotarioff on Panama Canal." Proc., ASCE, 74: No. 7 (Sept. 1948).
15. Weissmann, G. F., "The Damping Capacity of Some Granular Soils." Symposium on Soil Dynamics, ASTM, 64th Annual Meeting (June 1961).
16. Wilson, S. D., "Small Soil Compaction Apparatus Duplicates Field Results Closely." Engineering News-Record (Nov. 1950).

Compaction and Compression Characteristics Of Micaceous Fine Sands and Silts

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Residual soils developed from metamorphic and igneous rocks frequently contain large amounts of mica, which is considered detrimental, because such may be excessively compressive. However, the critical percentages of mica for various soil types have not been determined.

Laboratory tests on synthetic and natural micaceous soils indicate that soils containing less than 10 percent mica and compacted to modified Proctor densities show no significant changes in dry density or compressibility. With increasing mica content, the dry density decreases and the compressibility increases. The presence of fine mica has a greater influence on compressibility than coarse mica. Correlation of results between synthetic and natural soils appears to be good.

• PLACEMENT of pavement sections and foundation elements on compacted local soils is common practice; however, in certain areas the local soils are considered unsuitable for this purpose. Many engineers consider micaceous soils to be unsuitable. Much of their distrust can be traced to the few articles on micaceous soils that have appeared in the literature. In discussing the influence of particle shape on compressibility, Terzaghi and Peck (11) presented compression curves for sands containing 10 and 20 percent mica which indicated a highly compressible material. Although the compression curves were useful to demonstrate a point, the results have been misinterpreted by many. The relative densities of the micaceous soils were not given, but presumably they were low. Rengmark (6) attributed the poor conditions of road surfaces in Sweden to the inferior performance of base materials which contained substantial quantities of mica. Nevertheless, not all the literature is of a negative nature. Sowers (10) indicated that well-compacted fills of micaceous sandy silts and silty sands make satisfactory foundation material, but cautioned that high densities must be obtained.

Micas are members of the phyllosilicate family of minerals, and as such they display a typical platey shape. The mica group contains many species, but muscovite and biotite are the most common varieties and have the widest distribution. Muscovites are most abundant in metamorphic rocks, whereas biotites occur in igneous, metamorphic, and sedimentary rocks. Soils formed during weathering of these rocks still contain mica bands and sheets, but they are ultimately broken down into smaller units. Mica contents of 20 to 30 percent are not uncommon, and bands of 100 percent mica have been observed. Mica flakes, or plates, in soil are usually small and of fine sand and silt sizes. Reported mixtures have generally been classified as nonplastic micaceous sandy silts or micaceous silty sands.

This investigation was undertaken to understand better the influence of mica content on the compression characteristics of nonplastic sandy silts and silty sands. Changes in compressibility caused by increasing mica contents were examined, and critical contents below which effects were slight were determined. Differences caused by substi-

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TABLE 1

DESCRIPTION OF MATERIALS USED

Sample	Source	Specific Gravity	Classification and Description
Sandy silt	Little Gap, Pa.	2.69	Sandy silt with some clay; liquid limit approximately 20, determined by one blow-count method; nonplastic; low dry strength
Silty sand	Wallington, N. J.	2.66	Red silty sand; capable of being compacted to high densities; nonplastic; slight amount of dry strength
Fine sand	Paramus, N. J.	2.70	Fine sand with trace of silt; nonplastic; no dry strength
Coarse mica	A & T Mineral Services, Springfield Gardens, N. Y.	2.84	Muscovite; effective grain size from sand to clay, but predominantly of sand and silt sizes; coarse textures
Fine mica	Charles Wagner, Inc., Phila., Pa.	2.89	Muscovite (Concord mica); effective sizes predominantly in low silt range; extremely fluffy when dry; greasy to the touch; adhesive when wet
Micaceous silty sand (natural)	Longwood Gardens Estate, Kennett Sq., Pa.	2.76	Nonplastic micaceous silty sand; residual soil formed from a micaceous schist; mineralogical composition primarily quartz, feldspar, and mica; slightly dry strength; mica content estimated at 30%, principally among fine sand and silt sizes

tuting differing mica sizes, and variation of density and of moisture content were also studied. The influence of mica content on the moisture density relationship was also studied.

MATERIALS

Synthetic micaceous soils were used for the major portion of this study because naturally occurring soils of varying mica content, equal mineral composition, and equal grain-size characteristics would be difficult, if not impossible, to obtain. Thus, to investigate the influence of mica content alone, soils with known particle gradations and mica contents were created by combining nonmicaceous soils and mica. Descriptions and sources of materials used are given in Table 1.

Three nonmicaceous soils were selected on the basis of their similarity to the granular portion of known micaceous soils. The grain-size distribution curves of the selected soils are shown in Figure 1. All three soils are in the fine sand to silt range. Nonplastic soils were chosen because the observed natural micaceous soils were predominately nonplastic, and further it was considered inadvisable to include the influence of clay in this study because of the anticipated difficulty in evaluating the influence of mica in plastic soils. The X-ray diffraction patterns for the three soils are shown in Figure 2.

Micas to be mixed with the nonmicaceous soils were chosen on the basis of grain size and mineral purity. A coarse and a fine mica were selected to determine the effect of mica size. Gradation curves for these materials are shown in Figure 3. Descriptions and sources of the mica used are given in Table 1. The X-ray diffraction patterns for the two micas are given in Figure 4.

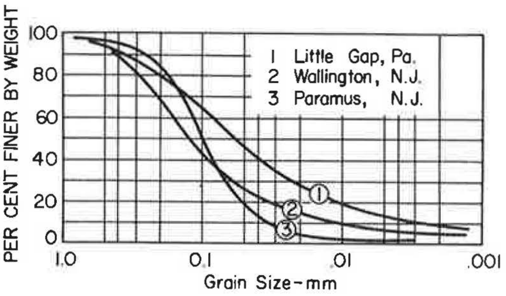

Figure 1. Gradation curve, nonmicaceous soils.

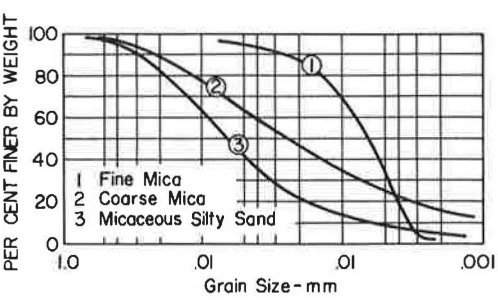

Figure 2. Gradation curve, micas and micaceous soil.

Natural micaceous soils were used to compare the compression characteristics of these materials with those of the synthetic samples. The gradation curve for the natural soil is shown in Figure 3, with material description and source included in Table 1, and X-ray pattern in Figure 5. X-ray patterns of synthetic soils are given on Figure 5 for comparison. The mica peaks can be located by comparing these patterns with those in Figure 4.

The average specific gravities of the materials used in the study are given in Table 1.

TEST PROCEDURE

To provide synthetic micaceous soils of known mica contents, proportioned weights of representative materials were mixed manually until a uniform texture was obtained. Soils and mica were mixed to yield test samples in the proportions shown in Table 2.

The mica content of natural micaceous soils was determined by the point count method (2). The type of mica was checked by X-ray diffraction procedures and the characteristics of the mica were noted by optical observations. The moisture density relationship was obtained for all samples using the modified Proctor test (AASHO T-180-57).

Samples were compacted and testing in floating ring consolidometers. For most tests polished zinc chromate plated steel rings 1.75 in. high and 4.0 in. in diameter were used, but a few tests were performed using Teflon-lined stainless steel rings. Porous stones were provided at upper and lower boundaries to insure proper drainage during compression.

The major portion of the testing program was concerned with the evaluation of compressibility of each material at its maximum density as determined by the modified Proctor test. To determine the effect of moisture on a sample of given mica content and maximum density, compression tests were run on samples at three different moisture contents: (a) at optimum moisture, (b) significantly below optimum moisture, and (c) above optimum moisture. The dry density was maintained within 1 percent for all tests of a given material.

TABLE 2

SYNTHETIC MICACEOUS SOILS

Soil	Source	Coarse Mica[a] (%)	Fine Mica[a] (%)
Sandy silt	Little Gap, Pa.	0, 3, 6, 12, 25, 50, 100	0, 6, 12, 25, 50
Silty sand	Wallington, N. J.	0, 6, 12, 25, 100	—
Fine sand	Paramus, N. J.	0, 6, 12, 25, 50, 100	—

[a]Percentage of mica content by weight in total sample.

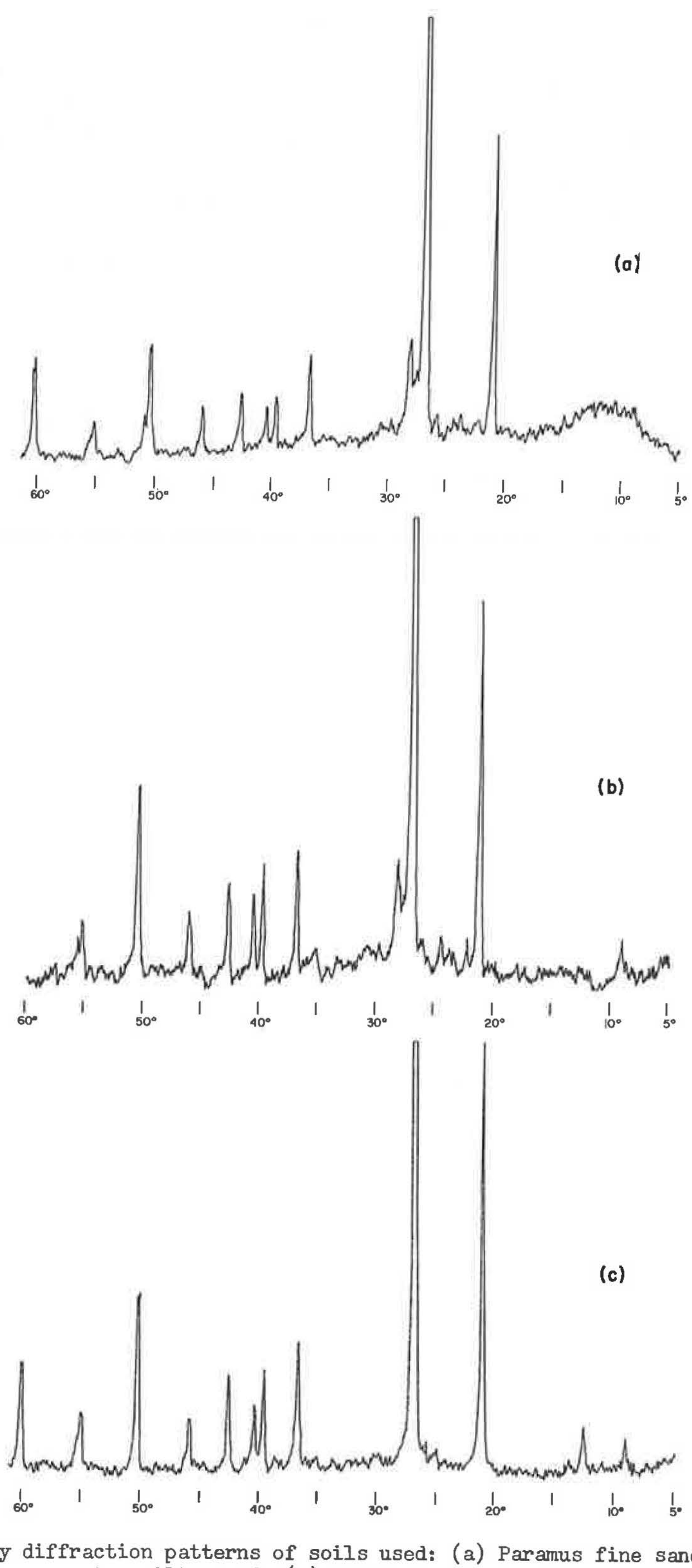

Figure 3. X-ray diffraction patterns of soils used: (a) Paramus fine sand; (b) Wallington silty sand; (c) Little Gap sandy silt.

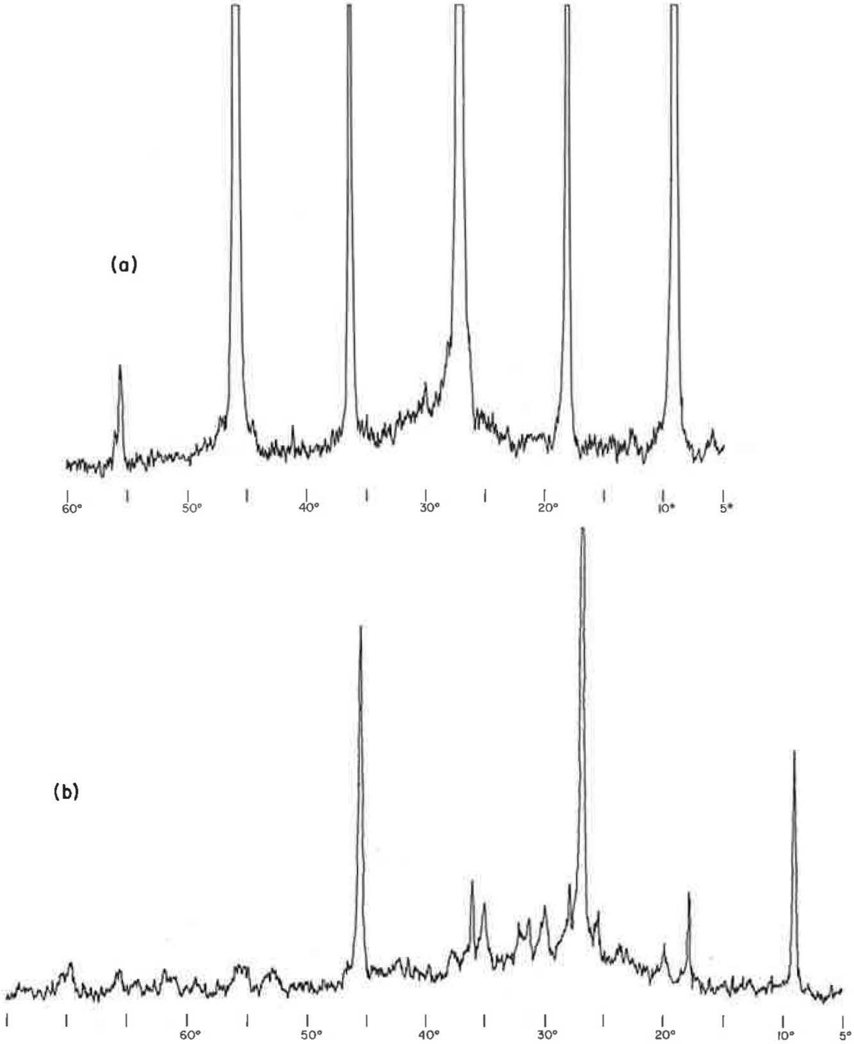

Figure 4. X-ray diffraction patterns of micas used: (a) fine mica; (b) coarse mica.

A minor portion of the testing program dealt with the influence of density on the compression characteristics of the more micaceous mixtures. Minimum densities were determined using the methods suggested by Yemmington (1) and Burmeister (1). Standard Proctor densities were determined using the AASHO T-99-57 procedure. Compression tests were performed on selected soils with mica contents of 25 and 50 percent at their minimum and standard Proctor densities.

RESULTS

The major portion of this study was concerned with the compression characteristics of micaceous soils compacted at modified Proctor density. Modified Proctor density was chosen as a standard because it represents a density commonly required, or used as a reference, on many projects. A soil that is excessively compressive after being compacted to modified Proctor density would normally be rejected. To ascertain the influence of density on the compression characteristics of the highly micaceous soils, a portion of the study was used to investigate this.

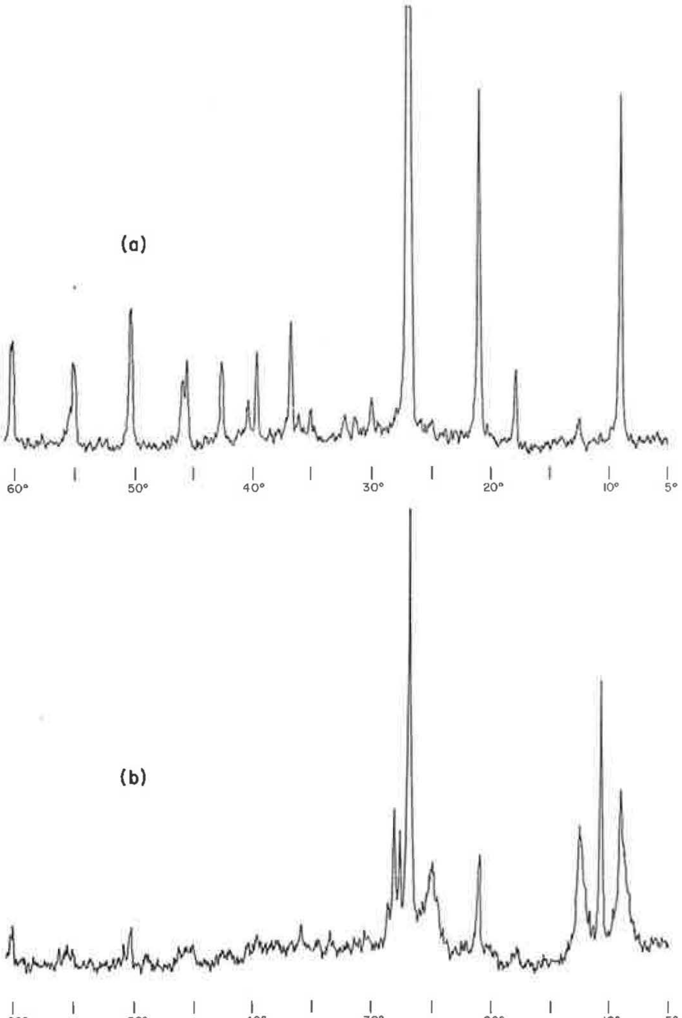

Figure 5. X-ray diffraction patterns of micaceous soils: (a) Little Gap sandy silt plus 25 percent coarse mica; (b) Longwood Gardens micaceous soil.

Moisture-Density Relationships

The influence of coarse mica content on the maximum modified Proctor densities and optimum water contents is significant (Figs. 6, 7, and 8). The trend is for the density and moisture contents to remain essentially constant for coarse mica contents of less than 10 percent. A variation of density of not more than 1 percent was obtained for all soils regardless of the soil type. As mica content is increased beyond 10 percent, decreases in maximum density and increases in optimum water content occur, the changes becoming greater with high mica content. The greatest change in density occurred with the silty sand soil mixture. The maximum densities of all three soils underwent an average reduction of 6 percent by the addition of 25 percent coarse mica.

Fine mica has a slightly greater effect than coarse mica on both dentisy and optimum moisture content, as shown in Figure 8. The decrease in dry density brought about by the addition of 10 percent mica was about 4 percent. As mica content is increased beyond 10 percent, maximum density decreases and optimum moisture content increases, but to a greater extent than with coarse mica.

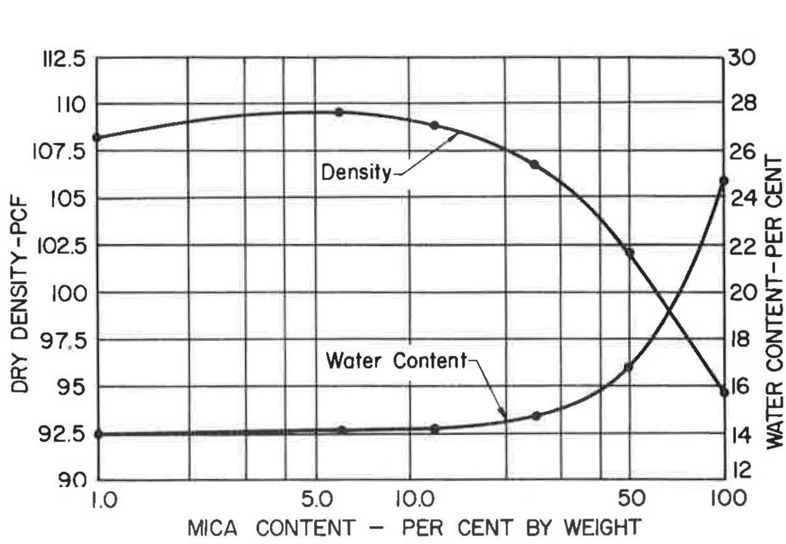

Figure 6. Density and water content vs mica content, Paramus fine sand.

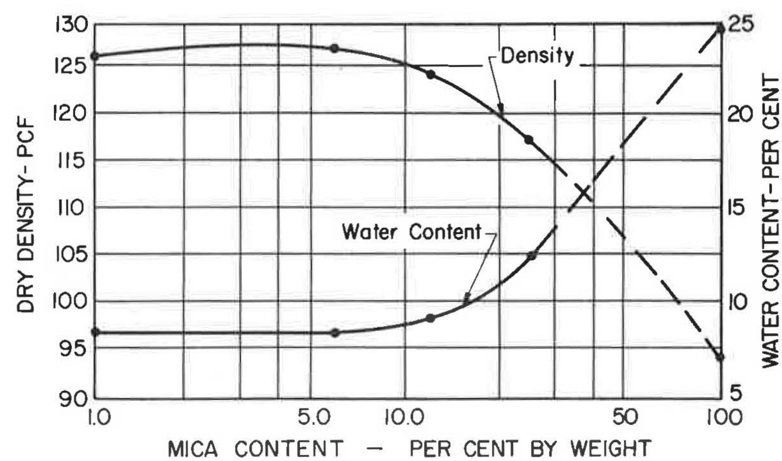

Figure 7. Density and water content vs mica content, Wallington silty sand.

To explain the variations of density and optimum moisture, the specific gravity and shape of the nonmicaceous and micaceous materials were considered. In each case, the specific gravity of the mica was greater than for the nonmica (Table 1). The nonmica particles are approximately equidimensional, whereas the mica particles are plate-like with high surface area to volume ratios.

At low mica contents, the mica flakes are sparsely distributed and do not interact to any degree. For the coarse mica-soil mixtures, a mica particle may simply replace an individual granular particle or fill an existing void, thus contributing to a higher density. As the quantity of mica flakes increases, so does the possibility of creating more void spaces, thus decreasing the dry density of the soil. Individual mica particles are capable of spanning over voids instead of filling them. If mica flakes abound in sufficient number to interact, the bridging phenomenon is further augmented.

When fine mica is considered, the number of flakes for a given weight is much greater than for coarse mica. Thus, contact between flakes is increased with corresponding increases in void ratio. Soil particles are too large to fill the small but numerous openings. Hence, when fine mica is added to a fine sand or silt, a decrease in

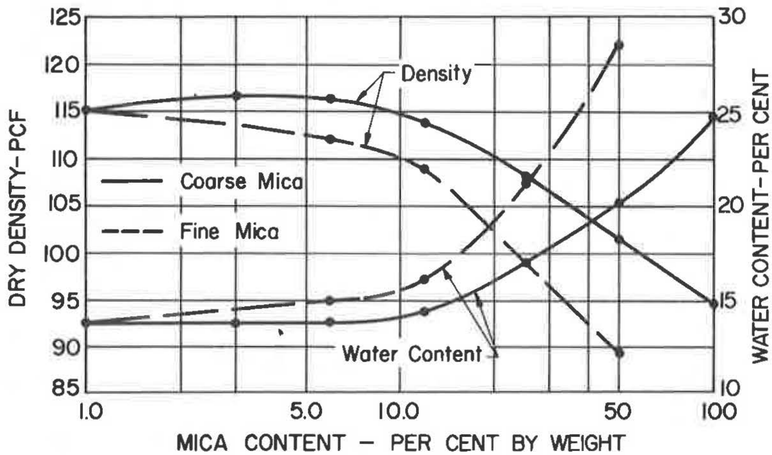

Figure 8. Density and water content vs mica content, Little Gap sandy silt with some clay.

density occurs even at low mica contents. Changes in void ratio with variation in mica for the soils used are given in Table 3.

Optimum water content increases with increasing mica content and the amount of optimum moisture is greater for the fine mica mixtures. Thus, the optimum moisture is a function of the specific surface of the micas.

Compression Tests

For the series of compression tests, densities at the start of the test conformed to the maximum Proctor densities. The soils were dynamically compacted in the compression rings, because Schultze and Moussa (7) found that sands compacted dynamically are more compressible than statically compressed sands. Thus the compression values obtained in this study are probably the greatest that can be anticipated. The results of the compression tests are given in Figures 9, 10, 11, 12, and 13.

As previously noted, mix densities start to fall below mica-free densities after the coarse mica content exceeds 10 percent. For these materials, the initially large void

TABLE 3

CHANGES IN VOID RATIO WITH MICA CONTENT[a]

Weight	Coarse Mica						Fine Mica, Sandy Silt	
	Fine Sand		Silty Sand		Sandy Silt			
	e_o	$\%\Delta e$	e_o	$\%\Delta e$	e_o	$\%\Delta e$	e_o	$\%\Delta e$
0	0.556	x	0.312	x	0.453	x	0.453	x
3	x	x	x	x	0.448	-1	x	x
6	0.557	x	0.330	+6	0.478	+5	0.510	+13
12	0.565	+2	0.355	+14	0.505	+11	0.554	+22
25	0.612	+10	0.439	+41	0.501	+31	0.686	+55
50	0.710	+28	x	x	0.702	+55	0.960	+212

[a] e_o= void ratio at modified Proctor density; and
$\%\Delta e$=percent change in void ratio due to mica content.

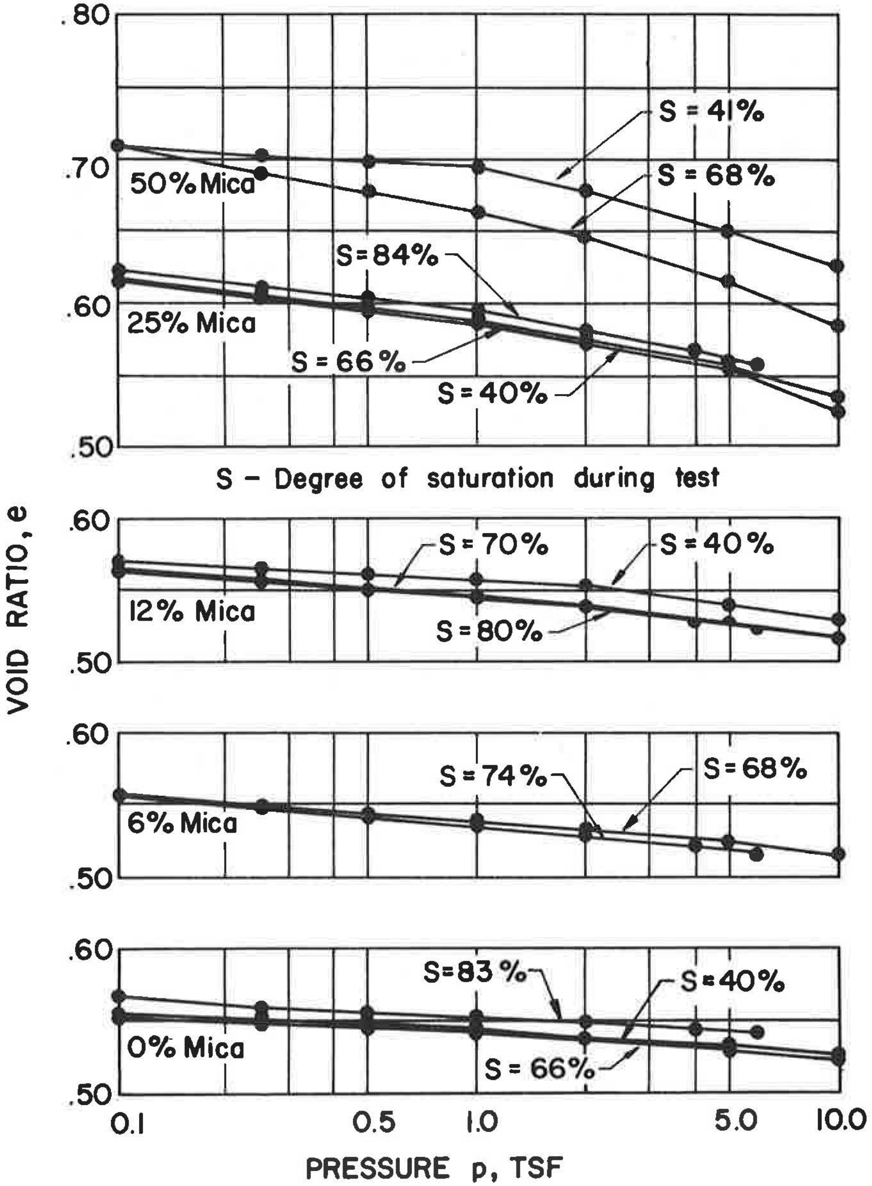

Figure 9. Compression test, Paramus fine sand.

ratios are indicative of greater compressibility. However, even for low mica contents, where densities remain approximately constant, compressibility was observed to increase with mica content.

To facilitate a comparative study, some curves representing the test results of samples not initially at, but very near, maximum density were transposed vertically to start at the void ratio corresponding to the maximum density. Such curves are shown as dashed lines. Actual results are included, but points have not been connected.

For all the samples tested, a break was observed in the e-log p-curve, this leads to the speculation that the compaction effort had the effect of a precompression load. Two tendencies were noted: (a) with a given soil, the break occurs at lower pressures for higher mica contents; and (b) in comparing soils at a given mica content, the break occurs at lower loads for soils with higher contents of fines.

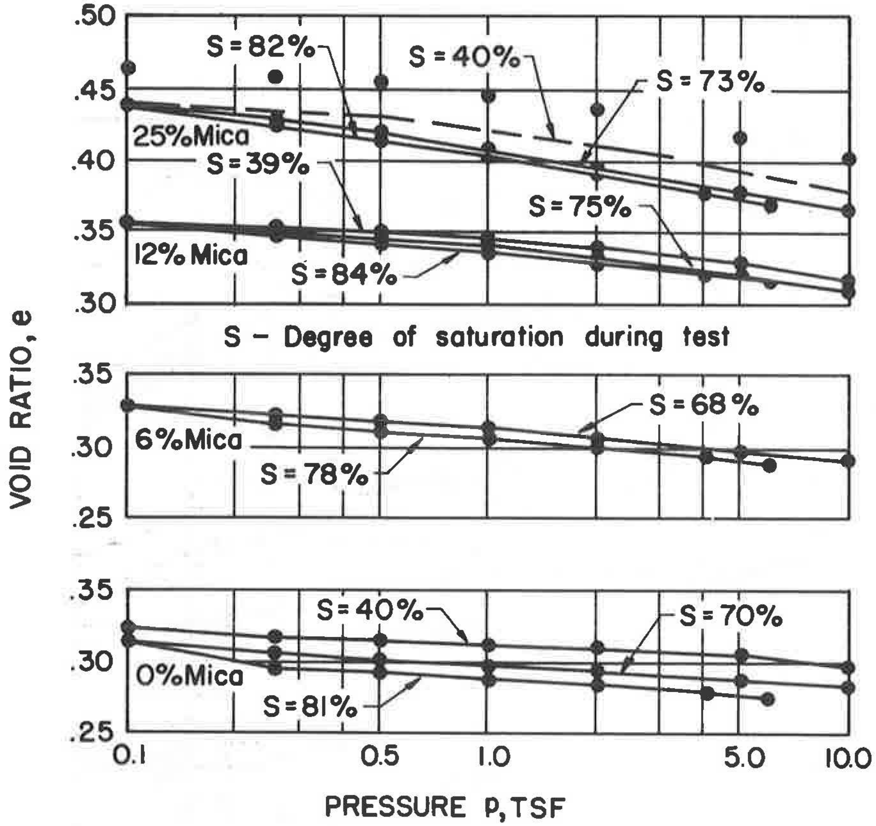

Figure 10. Compression test, Wallington silty sand.

The e-log p-curve is approximately linear in the 2- to 5-tsf range for all samples. In Figure 13, the numerical value of the slope for this loading range is shown as it varies with mica content. For the three soil types, the slope increases with mica content, but the variation between soil types is minor if the same size mica is the additive. Thus, the mica particles have a dominating effect on the compression characteristics, with variations in nonmica particle sizes being of limited influence. This is made more evident when comparing results obtained for the same soil mixed with different sized micas.

Minor variations in compressibility were noted for differing degrees of saturation, the trend was for the compressibility to increase with increasing water content. Probably the addition of water facilitates reorientation of particles under loading.

It may be argued that frictional resistance developed along the consolidation ring may be changed as water content is varied for a given sample. However, there was little evidence of adhesion, and inasmuch as solid friction depends on normal pressures only, ring friction in each case would remain approximately constant because samples were identical except for water content. Further, Leonards and Girault (4) have given evidence that ring friction may be of more importance to the time rate of compression than to the ultimate compression value, and is more pronounced for very low ranges of loading. Nevertheless, it is felt that on a comparative basis, consideration of friction is of little consequence.

Expansion of the samples when unloaded was minor. Volume increases were of the same magnitude for zero and high percentages of coarse mica. Only for soil with 50 percent fine mica was rebound considered notable, an unloading curve for this mixture is shown in Figure 12.

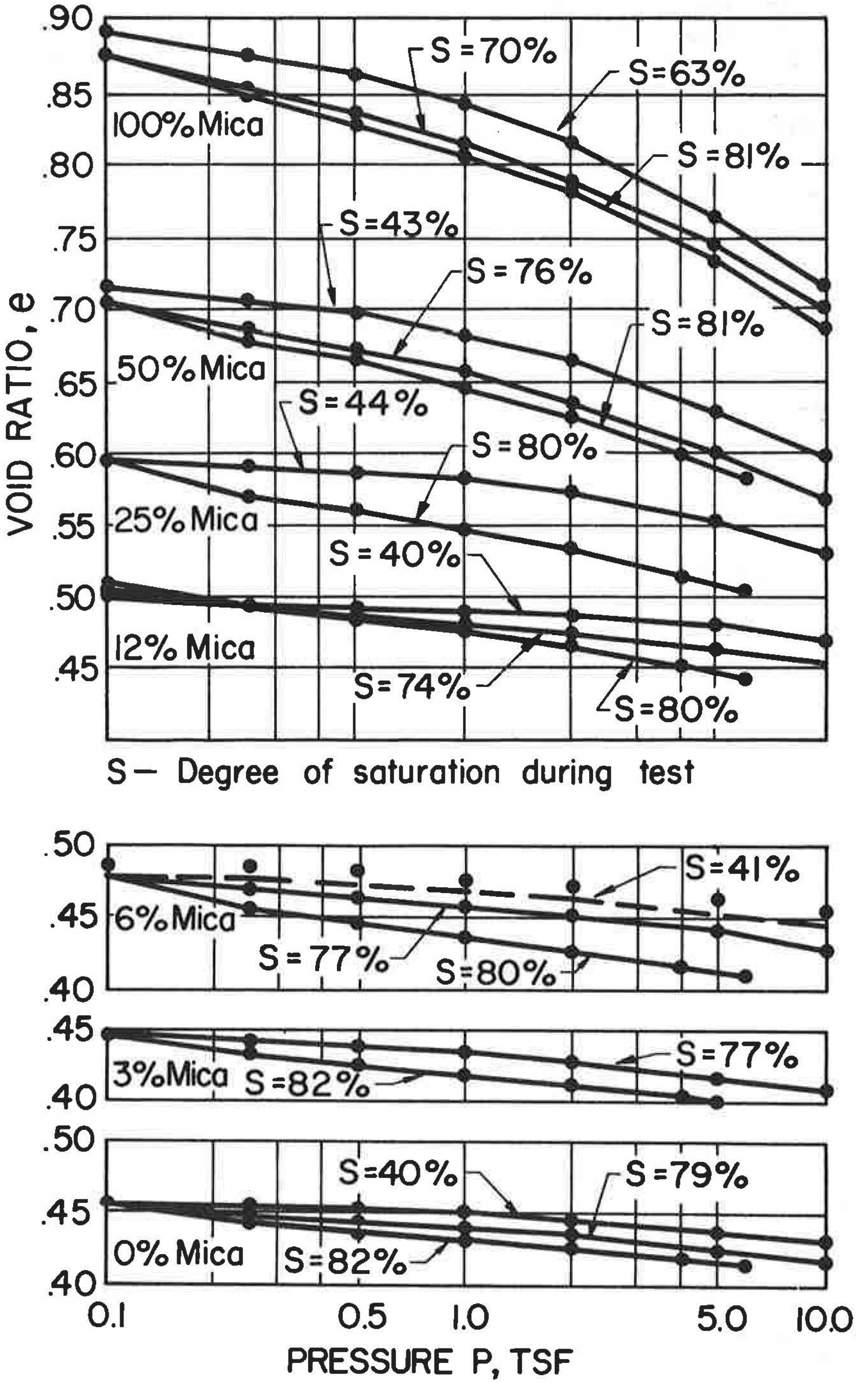

Figure 11. Compression test, Little Gap sandy silt (coarse mica).

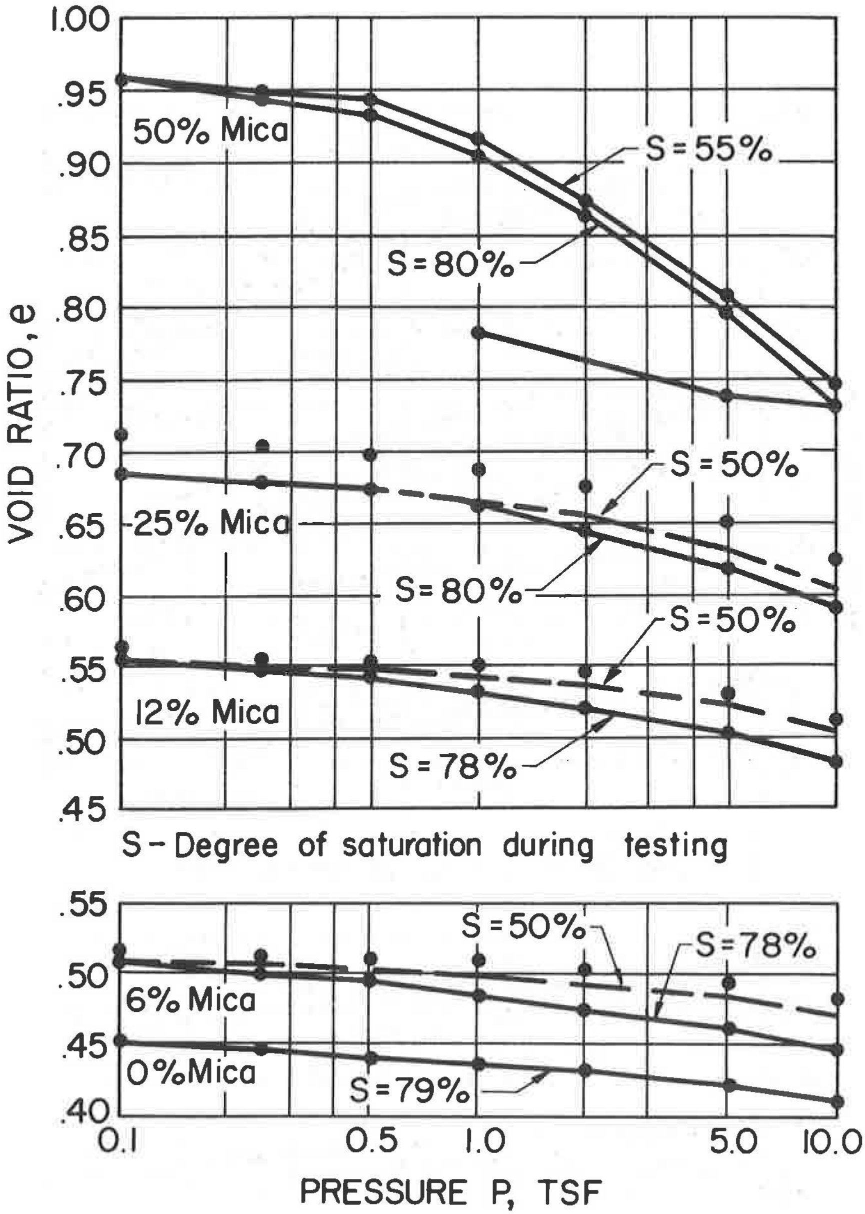

Figure 12. Compression test, Little Gap sandy silt (fine mica).

A natural micaceous soil was used to check the trends observed and the values obtained for the synthetic soils. The e-log p-curve for the natural micaceous soil sample is shown in Figure 14. The value of the slope of the compression curve (2 to 5 tsf) vs mica content is shown in Figure 13. The mica particles are slightly smaller than the coarse mica used, but larger than the fine mica. Consequently, this point would be expected to fall between the extremes determined with the synthetic soils. The close agreement observed leads to the following conclusion: it may be possible to predict soil compression characteristics if mica content, mica sizes, nonmica sizes, and amount of compaction are known.

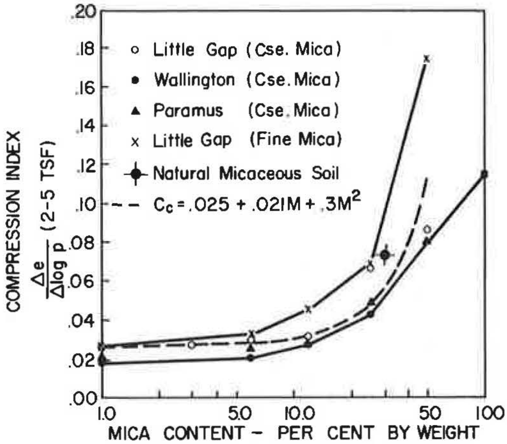

Figure 13. $\Delta e/\Delta$ log p vs mica content (2 to 5 tsf).

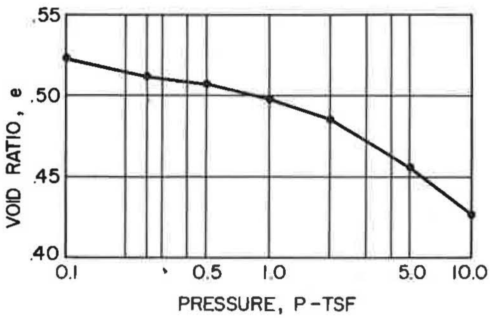

Figure 14. Compression test, Longwood Gardens micaceous soil.

For mica contents up to 50 percent, the compression index in the 2- to 5- tsf range for micaceous sands and silts compacted to maximum Proctor densities may be estimated from:

$$C_c = 0.025 + 0.021M + 0.3M^2 \qquad (1)$$

in which M is the mica content expressed as a fraction. For mica contents of 5 to 50 percent, the compression index may be approximated more easily, but with accuracy, by

$$C_c = 0.01 + 0.2M \qquad (2)$$

Minimum and Standard Densities

Because the higher mica contents had a significant influence on the modified Proctor densities, a limited study of the soils was undertaken. For determining the influence of the type and amount of mica alone, only one type of nonmicaceous soil was used in the synthetic mixtures. Two procedures were used to determine minimum density because of the lack of a standard procedure for this test. The results of these tests are given in Table 4. In all cases the minimum density decreased with increasing mica content. Exceedingly low minimum densities were obtained using high percentages of fine mica of uniform size. The minimum density of the 50 percent fine mica soil was about three-tenths of the modified Proctor density. The effect

TABLE 4

DRY DENSITIES OF MICACEOUS SOILS

Sample	Mica		Min. Density (pcf)		Proctor Density (pcf)	
	%	Type	Bur. (1) Method	Yem. (1) Method	Standard	Modified
Little Gap sandy sandy silt	T25	Coarse	60.1	62.2	107.1	108.4
		Fine	51.8	52.5	96.6	98.7
	T50	Coarse	56.9	60.5	98.8	101.2
		Fine	25.5	26.9	88.8	90.2
Longwood Gardens micaceous soil	—	—	70.2	69.2	104.6	114.6

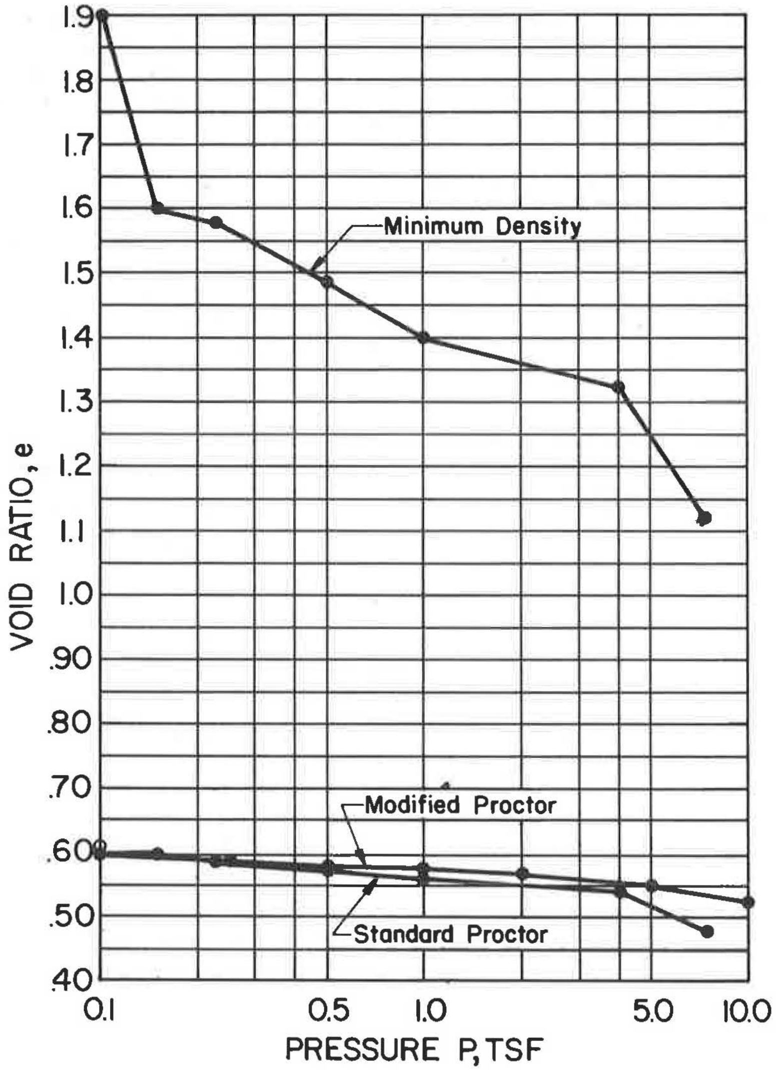

Figure 15. Compression test, Little Gap sandy silt plus 25 percent coarse mica.

of coarse mica on the minimum density values was not as great as the fine mica, but it was still significant. The influence of 25 percent fine mica was essentially the same as 50 percent coarse mica. A 50 percent coarse mica content resulted in a minimum density of approximately one-half the modified Proctor density. The influence of mica on the minimum density of the natural micaceous soil was essentially the same as that exhibited by the synthetic micaceous soils.

The standard Proctor densities for the synthetic micaceous soils were slightly less than their modified Proctor densities. This was not the case for the natural soil. No reason can be given for this fact, other than a difference in sample. A second sample, taken at the same time and location as the sample used in the minimum and modified Proctor densities, was used for the standard Proctor density. The grain-size distributions were similar.

Compression at Lower Densities

A minor testing program to illustrate the influence of the density of micaceous soils on their compression characteristics was undertaken with one micaceous mixture. The results of this testing are shown in Figure 15. Because the modified and the standard Proctor densities of this soil are nearly the same, their compression characteristics are very similar and may be taken as being equal. The slight difference in amount of compression at high loads is probably due to the higher moisture content of the standard Proctor sample. The high compressibility of the low density sample illustrates the need for density control with micaceous soils.

CONCLUSIONS

The presence of mica in a nonplastic soil can have a pronounced effect on its density, this is due principally to the particle shape of the micas. With a given compactive effort, increasing mica content in excess of about 10 percent causes dry densities to decrease and optimum moisture contents to increase. Differences between dry densities for soils with less than 10 percent mica are very minor—about 1 percent for coarse mica or only slightly greater; about 4 percent for fine mica. Minimum density also decreases with increasing mica content. Very low minimum densities may be obtained when fine mica is present in quantity. The effect of coarse mica is still significant. Minimum density values from 0.3 to 0.6 of the modified density values may be obtained when the mica content exceeds 25 percent.

The compressibility of nonplastic micaceous soils increase with increasing mica content. Soils containing fine mica are more compressible than those with coarse mica. Moisture has a slight effect on ultimate compression of micaceous soils; however, compression increases with the degree of saturation. Compressibility of well-compacted soils with up to 50 percent coarse mica and up to 30 percent fine mica may be tolerable. The compression index of nonplastic micaceous soils with mica contents up to 50 percent and compacted to the modified Proctor density may be estimated by Eq. 1. Close control over density is needed to limit the amount of compression of micaceous soils.

REFERENCES

1. "Procedures for Testing Soils." ASTM (1958).
2. Chayes, F., "Petrographic Modal Analysis." Wiley (1956).
3. Grim, R. E., "Clay Mineralogy." McGraw-Hill (1953).
4. Leonards, G. A., and Girault, P., "A Study of the One-Dimensional Consolidation Test." Proc., 5th Internat. Conf. on Soil Mechanics and Foundation Engineering. Paris (1961).
5. Pettijohn, F. J., "Sedimentary Rocks." Harper (1956).
6. Rengmark, V. F., "The Significance of Mineralogical Composition for the Conditions of Bearing Capacity of (Grand) Roads." Svenska Vagforen. Tidskr. (1947).
7. Schultz, E., and Moussa, A., "Factors Affecting the Compressibility of Sand." Proc., 5th Internat. Conf. on Soil Mechanics and Foundation Engineering, Paris (1961).
8. Skempton, A. W., "Notes on the Compressibility of Clays." Quart. Jour. of Geolog. Soc. (London), Vol. 100 (1944).
9. Sowers, G. B., and Sowers, G. F., "Introductory Soil Mechanics and Foundations." MacMillan (1961).
10. Sowers, G. F., "Soil Problems in the Southern Piedmont Region." Proc., ASTM, Soil Mechanics and Found. Div., Vol. 80, Separate 416 (1954).
11. Terzaghi, K., "Influence of Geological Factors on the Engineering Properties of Sediments." Economic Geology, 50th Anniversary Vol. (1955).
12. Terzaghi, F., and Peck, R. B., "Soil Mechanics in Engineering Practice." Wiley (1948).
13. Twenhofel, W. H., "Principles of Sedimentation." McGraw-Hill (1950).

Effect of Geometric Characteristics of Coarse Aggregates on Compaction Characteristics Of Soil-Aggregate Mixtures

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This paper reports the results of a laboratory study to investigate the effect of the geometric characteristics of coarse aggregate particles on the compaction characteristics of soil-aggregate mixtures. Six coarse aggregate materials with discernible geometric characteristics, including both pit-run gravel and crushed stone materials, were used in the study. The geometric characteristics of these materials were determined by the "particle index" test, a procedure developed at the University of Illinois particularly for the quantitative evaluation of these characteristics.

The results of this investigation show that the volume of the voids in a soil-aggregate mixture of a given gradation under a standard laboratory compactive condition decreases more or less linearly with decreasing values of the particle index of the coarse aggregates; that is, as the coarse aggregate particles become more spherical, rounded, and smoothly surfaced. The conclusion that the density of a soil-aggregate mixture varies not only with the gradation of the mixture but also with the geometric characteristics of the coarse aggregate fraction appears to be deserving of some consideration in the construction control of soil-aggregate roads.

• THE SERVICE behavior of a soil-aggregate material for roads and pavements is dependent on many factors. One of the important influencing factors is that of density or the void characteristics of the compacted soil-aggregate material. Experimentation both in the field and in the laboratory has indicated that the stability of a soil-aggregate material increases with an increase in density (1), so long as excessive pore water pressures are not developed (2). Moreover, dense soil-aggregate mixtures afford high abrasive resistance, shed the greater portion of rain water, and maintain a more uniform moisture content through the replacement (by means of capillary action) of the moisture lost through evaporation (3). A desirable density may be achieved through the use of that gradation of the combined aggregate that has the least voids. Thus, by careful gradation control an aggregate skeleton of high stability could be obtained from materials that otherwise would possess a much lower stability.

Almost every State has specifications covering suitable gradations and materials for soil-aggregate roads. The AASHO and the ASTM also have standard specifications for materials and mixtures suitable for uses as surfaces, bases, and subbases, according to the gradation, as well as the Atterberg limits, of the soil-aggregate mixtures. Many of these grading specifications cover materials of various kinds, including stone, gravel, or slag with natural or crushed sand and fine mineral particles passing a No. 200 sieve (4). Because the gradations of soil-aggregate mixtures are always determined by a sieve analysis which classifies the sizes of aggregate on the basis of its least section area, and inasmuch as long, pencil-shaped or other irregularly shaped particles may pass a sieve and be weighed with others of lesser volume and hence of smaller size,

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Paper sponsored by Committee on Compaction of Embankments, Subgrades and Bases.

it is evident that aggregate materials with particles of different geometric characteristics may yield appreciably different density values even though they are identical in gradation.

In this paper the term "geometric characteristics" is used to include shape, angularity, and surface texture of aggregate particles. The term "shape" is used to refer to the form of an aggregate particle, whereas the term "angularity" is applied to the sharpness of the corners and edges of the particle. Thus, a cube and a tetrahedron are geometric solids of different shapes but, because the radius of curvature of their edges or corners is zero, they have equal degree of angularity. The term "surface texture" refers to the intimate details of the particle surface independent of shape and angularity. It is the property that measures the relative degree of smoothness or roughness of the particle surface.

OBJECTIVE AND SCOPE OF STUDY

In this investigation, a laboratory study was made to determine the effect of the geometric characteristics of coarse aggregates on the compaction characteristics of soil-aggregate mixtures. Six coarse aggregate samples with discernible geometric characteristics, including both pit-run gravel and crushed stone materials, were used in the study. The geometric characteristics of these materials were determined by the "particle index" test, a procedure developed at the University of Illinois particularly for the quantitative evaluation of these characteristics (5).

Each coarse aggregate was combined with various percentages of finer aggregate and soil materials to form seven different gradations. The compaction characteristics of these soil-aggregate mixtures were determined essentially according to AASHO Designation T-99-57. In this report the test data are analyzed and the effect of the geometric characteristics, as well as the gradation, of these materials on their void characteristics after compaction tests are presented.

TABLE 1

GENERAL PHYSICAL CHARACTERISTICS OF AGGREGATE MATERIALS

Characteristic	Gravel 61	Crushed Stone 178
Apparent specific gravity (AASHO T-85-60)	2.49	2.66
Absorption (AASHO T-85-45)	3.8	1.0
% wear in abrasion test (AASHO T-96-56)	31.5	28.7
% loss in soundness test (AASHO T-104-57)	13.0	14.1

TABLE 2

PARTICLE INDEX OF VARIOUS COARSE AGGREGATES

Sample Designation	Particle Index for Particle Size of		
	$\frac{3}{4}$- to $\frac{1}{2}$-In.	$\frac{1}{2}$- to $\frac{3}{8}$-In.	$\frac{3}{8}$-In. to No. 4
Gravel:			
61(a)	7.1	8.0	6.9
61(b)	9.8	9.3	8.4
61(c)	10.7	10.6	10.6
Crushed stone:			
178(a)	13.4	12.4	12.6
178(b)	16.0	15.9	14.9
178(c)	16.9	17.4	16.4

MATERIALS TESTED

Coarse aggregate materials used in this investigation were obtained from two sources: a pit-run gravel, designated No. 61, from Greenup, and a crushed stone, designated No. 178, from Casey, both in central Illinois. The gravel was produced from outwash deposits of the Wisconsian stage of glaciation. It was composed of a large variety of materials, including limestone and dolomite, dark colored igneous rocks, quartzite, sandstone, and some chert. The crushed stone was obtained from

Figure 1. Coarse aggregate samples.

formations of Pennsylvanian age. The rock was a compact, fine-grained, hard, gray limestone. The general physical characteristics of these two materials are summarized in Table 1.

The preceding materials were typical road aggregates conforming to the specifications of the Illinois Division of Highways for soil-aggregate roads. Though both materials contained varying amounts of particles of different shapes, the crushed stone was characteristically more angular and rougher in surface texture than the gravel material. To provide materials with a considerable range of variation in their geometric characteristics, each material was separated into three distinct classes by visual examination according to the shape of particles. In this connection, the sample containing only bulky particles or grains of which the length, width, and height were about the same was designated shape a, that containing elongated pieces was designated shape b, and that containing flat particles was designated shape c. During the separation process, the soft, unstable particles in the gravel material, largely consisting of sandstone and limestone, were also removed to prevent aggregate breakage during the compaction test.

The preceding classification of aggregates with respect to shape was entirely arbitrary, and was made merely to provide coarse aggregate samples that were individually distinctive in their particle shape, angularity, and surface texture. The visual classification was performed by one operator and checked by another in order to maintain a reasonable degree of consistency. Typical examples of these two aggregates of various shapes are shown in Figure 1.

The total geometric characteristics, embracing shape, angularity, and surface texture, of these coarse aggregate materials were determined by the "particle index test." A detailed description of this test has been presented elsewhere (5). Essentially, the

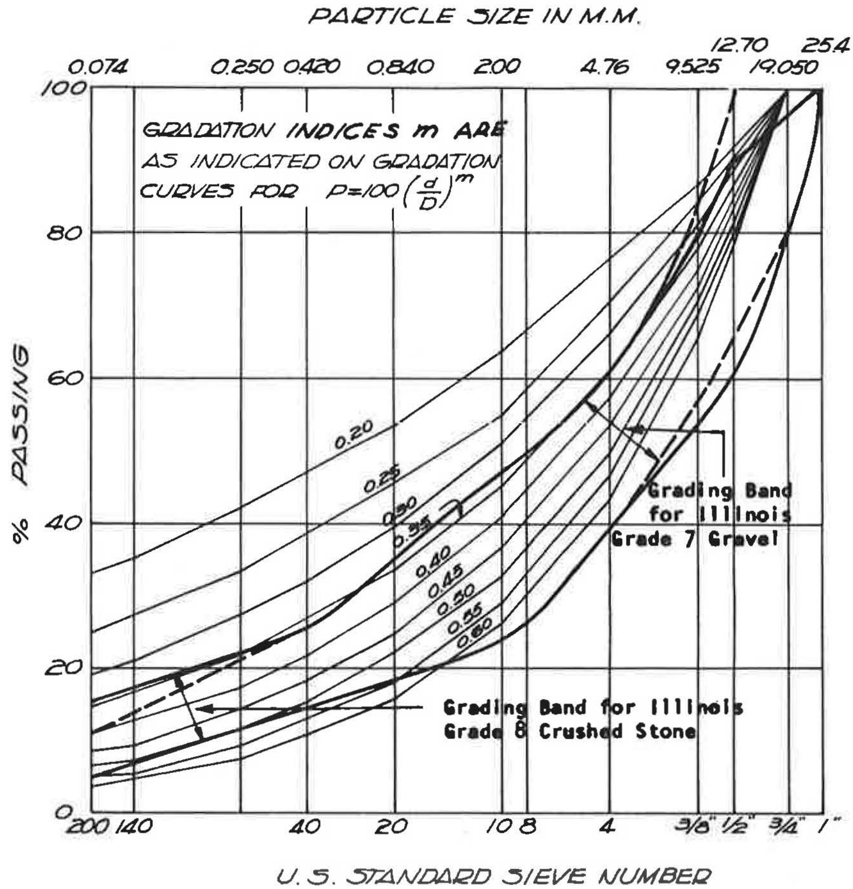

Figure 2. Particle size distribution of soil-aggregate mixtures.

test is based on the concept that the void conditions in a uniform-sized coarse aggregate when rodded in a standard rhombohedron mold show the combined features of shape, angularity, and surface texture of the aggregate. The result of this test is expressed as the particle index of the aggregate, for which a mass of single-sized, highly polished aluminum spheres is taken as zero. Typical index values of the aggregates that have been tested in the aggregate laboratory at the University of Illinois range from about 4 for a gravel composed of rather spherical particles with rounded corners and smooth surface to about 20 for a crushed limestone of flakey particles with angular corners and edges and a very rough surface.

The results of the particle index for the six aggregate samples are given in Table 2. The tests were performed according to the standard procedure in which each sample was separated by sieves into the following sizes: passing the 3/4-in. and retained on 1/2-in. sieve, passing the 1/2-in. and retained on 3/8-in. sieve, and passing the 3/8-in. and retained on No. 4 sieve. These values were used later in determining the particle index of the coarse aggregates in a soil-aggregate mixture containing particles of all the three sizes. The particle index of such a mixture was the weighted average of the particle index for each size group based on the grading of the mixture.

Particle index values in Table 2 show that, for the crushed stone or the gravel of the same nominal size, the particle index is greatest for the flat particles (shape c) and smallest for the bulky particles (shape a). For the crushed stone or gravel samples belonging to the same size and shape group, the general fact that the crushed stone exhibited a much greater value of particle index than the gravel material was attributable

TABLE 3

ATTERBERG LIMITS OF SOIL-AGGREGATE MIXTURES

Gradation Index	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
0.20	31.0	19.3	11.7
0.25	28.6	18.4	10.2
0.30	26.8	18.0	8.8
0.35	24.0	14.6	9.4
0.40	21.9	15.3	6.6
0.45	19.7	12.0	7.7
0.50	18.9	14.0	4.9
0.55	17.6	14.2	3.4
0.60	16.9	14.3	2.6

to its greater particle angularity and rougher surface texture. The different particle index values of the three particle sizes of an aggregate sample belonging to the same arbitrary shape group indicate that the total geometric characteristics of these arbitrarily selected particles were not unvarying.

The portion of the material passing the No. 4 sieve for this investigation was obtained from several sources. The material for the No. 4 to No. 140 sieve range was obtained from a stream gravel deposit located in the Bloomington moraine area near Penfield, Ill. For the No. 140 to No. 200 sieve material, Ottawa sand was obtained from a commercial source. The minus No. 200 binder material used was a greenish-brown silty clay from the B-horizon of the Flanagan, Catlin, Drummer soil association area in Urbana, Ill.

To control the gradation of the materials fully, the soil-aggregate mixtures to be used in the compaction tests were artificially prepared from gradation curves based on the mathematical expression for grain-size distribution developed by Talbot and Richart (6):

$$p = 100\,(d/D)^m \tag{1}$$

in which

p = percentage of material passing sieve with opening of d;
D = maximum size of particles of the given soil-aggregate mixture; and
m = variable exponent, termed "gradation index."

In this investigation, the maximum size D was limited to the ¾-in. size, and the gradation indexes included 0.20, 0.25, 0.30, 0.35, 0.40, 0.45, 0.50, 0.55, and 0.60 The particle size distribution curves for various values of gradation index used in this investigation are shown in Figure 2. Also presented in the figure are the grading bands specified by the Illinois Division of Highways for grade 7 gravel and grade 8 crushed stone, which give the grading limits that have been found in Illinois to give satisfactory results in practice for the types of aggregate materials used in this investigation. Also, the curve with a gradation index of 0.50 represents the ideal grading curve for maximum density developed by Fuller. Although it was originally developed for concrete aggregates, nearly all the gradation specifications now in use approximate, with variable tolerances, this curve.

Table 3 gives the liquid limit, plastic limit, and the plasticity index of soil-aggregate mixtures having different gradation indexes. Because these physical constants were determined on the portion of the mixtures passing the No. 40 sieve, these values were the same for all mixtures having the same gradation index. This similarity of the liquid limit and the plasticity index of the soil fines for the soil-aggregate mixtures involving six different coarse aggregates, but having the same gradation, permitted direct comparison of the test results in terms of the particle index or the geometric characteristics of the coarse aggregate particles.

COMPACTION TEST

The compaction test was conducted on the soil-aggregate mixtures essentially according to Method C of AASHO Designation T-99-57. The standard mold used in this test is 4.0 in. in diameter and 4.59 in. in height, with a volume of 1/30 cu ft. The rammer is 2 in. in diameter having a flat circular face and weighing 5.5 lb. In the standard procedure, the sample is compacted in three equal layers, each by 25 uniformly distributed

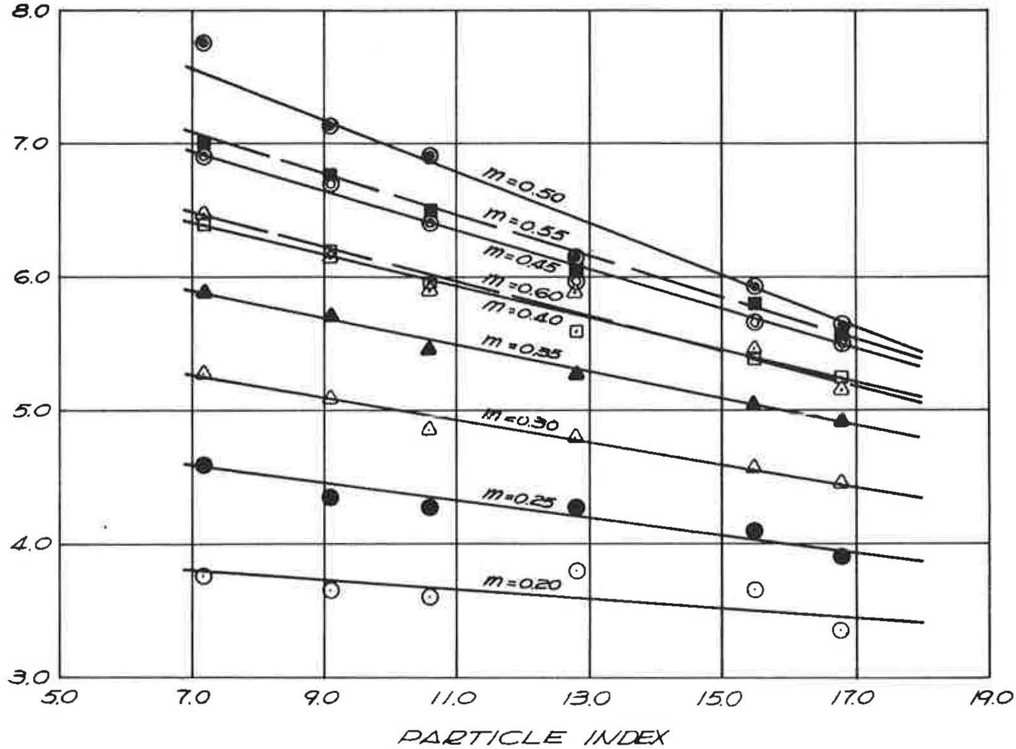

Figure 3. Relation between compaction characteristics of soil-aggregate mixtures and particle index of coarse aggregates for various mixture gradations.

blows from the rammer, dropping free from a height of 12 in. above the elevation of the material to be compacted. The moisture content of the specimen is determined by taking a small moisture sample from the center of the compacted material, after the specimen has been weighed and removed from the mold, and then drying it in an oven to constant weight.

Owing to the large quantity of aggregate materials in most soil-aggregate mixtures in this investigation, a variation from the standard method of determining moisture content was found necessary. To determine the moisture content more accurately, the entire soil-aggregate mixture, rather than a small moisture sample, was used for its determination. In this connection, the soil-aggregate sample was oven dried before mixing with water. By knowing the oven-dry weight of the soil-aggregate sample and being careful to prevent evaporation and not to lose any material during the compaction process, the moisture content of the soil-aggregate mixture was determined before each compaction trial simply by dividing the amount of water that had been added by the original oven-dry material expressed as a percentage.

The preceding procedure was first suggested by Ziegler (7) and was used with considerable success in this investigation. It permitted the use of a relatively small quantity (3,000 g) of soil-aggregate sample for a complete test. In addition, the moisture-density relationship curve was determined within the comparatively short period of time of approximately 3 hr.

In the determination of the weight-volume relationships of the soil-aggregate mixtures, corrections were made for the volume of water that was absorbed by the aggregate particles during the compaction test. This portion of water varied with the kind of aggregate material as well as the general shape of the particles. Generally speaking, the absorption of the three crushed stone samples was more or less the same (1.0 percent) but was much lower than that of the gravel samples. Among the gravel samples,

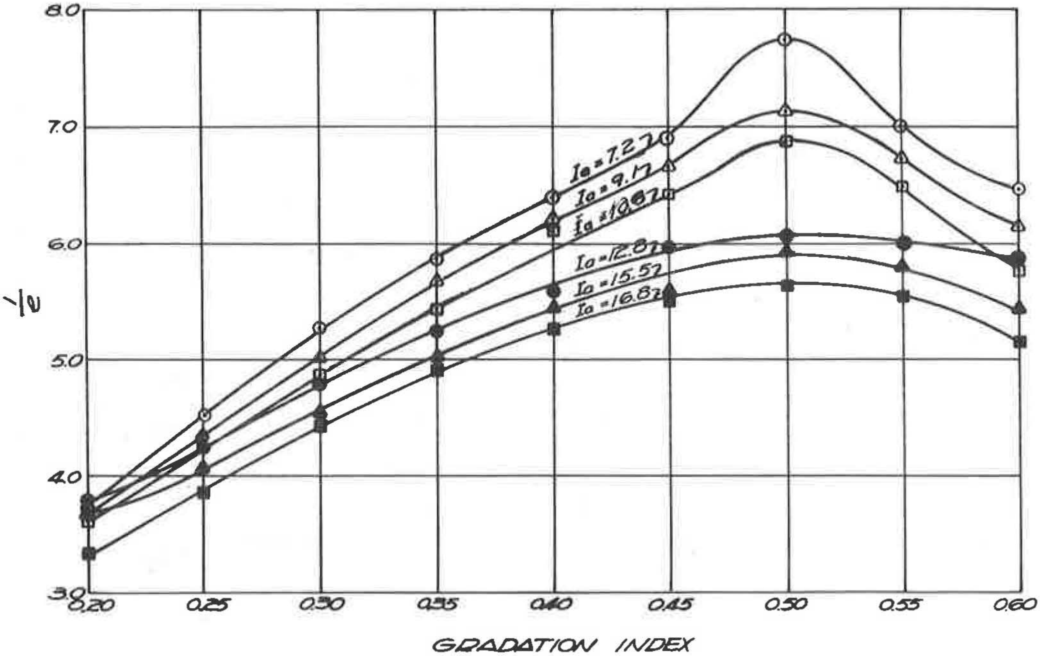

Figure 4. Relation between compaction characteristics and mixture gradation for soil-aggregate mixtures containing coarse aggregates of varying particle index.

the one with flat particles had the highest absorption (5.0 percent) and that with the bulky particles the lowest absorption (3.0 percent). Because only the volume of water in the void space of a soil-aggregate mixture was used in the volumetric analysis, the amount of water absorbed by aggregates, which had been predetermined in the laboratory, was excluded from the total water added in the determination of the void characteristics of the mixtures.

RESULTS

From the results of the compaction tests, the void characteristics of each mixture were analyzed at maximum dry density and optimum moisture content. In the presentation of these data, use has been made of the reciprocal value of the void ratio:

$$e = V_v/V_s \tag{2a}$$

in which

V_v = total volume of voids; and
V_s = the volume of solids.

Hence,

$$1/e = V_s/V_v \tag{2b}$$

The reciprocal of the void ratio affords a convenient basis for comparing the compaction characteristics of soil-aggregate mixtures containing particles of different specific gravities. A low reciprocal value of the void ratio indicates a low solids content per unit volume of compacted soil-aggregate mixture.

Figure 3 gives the relation between the reciprocal voids ratio of various soil-aggregate mixtures and the particle index of their coarse aggregates. The particle index for each mixture is the weighted average of the values for the three sizes of aggregate particles given in Table 2 on the basis of their weight percentages. The plot clearly demonstrates that the particle index or the geometric characteristics of the coarse aggregates

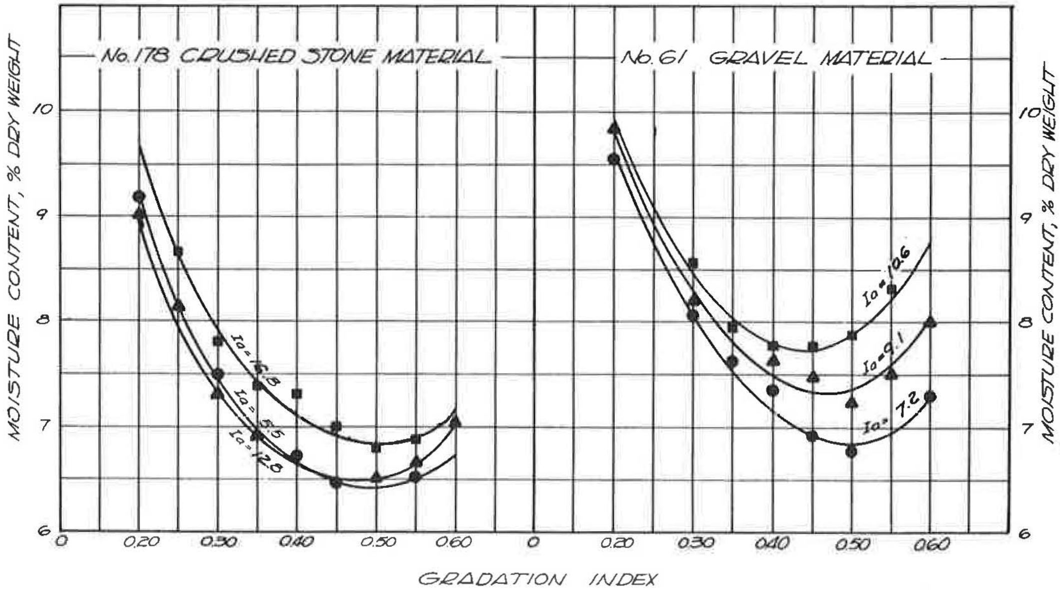

Figure 5. Relation between moisture content and mixture gradation for soil-aggregate mixtures containing coarse aggregates of varying particle index.

in the mixture was an influencing factor on the resultant void characteristics of the compacted sample. More specifically, the volume of the voids in the soil-aggregate mixtures of a given gradation under a standard laboratory compaction condition increased more or less linearly with increasing values of the particle index of the coarse aggregates. It must be noted that the particle index reflects the combined features of shape, angularity, and surface texture of an aggregate mixture, and that the value becomes progressively smaller as the aggregate particles become more spherical, rounded, and smoothly surfaced. It is obvious that aggregate particles with a small particle index would slip more readily over one another when exposed to a compactive effort than would the aggregates with a larger particle index. Hence, the former would form a more closely knit aggregate pattern and thus contain less void space than the latter.

Also, the significance of the particle index of the coarse aggregates on the resultant void characteristics of the compacted samples appears to increase with increasing values of gradation index until the optimum value of 0.50 is reached. After this point is reached, the significance becomes less apparent with further increases in gradation index values. This relationship is indicated by the change in slope of the lines in Figure 3 for increasing values of gradation index.

Figure 4 shows the reciprocal void ratios of various soil-aggregate mixtures are plotted against their corresponding gradation indexes. The data indicate that, for the soil-aggregate mixtures containing coarse aggregates of given geometric characteristics, there was one gradation value at which minimum void space in the compacted soil-aggregate sample was consistently obtained. This particular value of gradation index as indicated from this series was equal to 0.50. This gradation seems to represent the most desirable number of particles of any particular size in the mixture such that the voids within any particle size were filled by those of a smaller size, regardless of the geometric characteristics of these particles. This gradation index represents the ideal grading curve for maximum density developed by Fuller. Also, similar results have been shown by previous investigators (8).

Figures 3 and 4 also show that the minimum void space for a given soil-aggregate mixture under a particular compactive condition may be obtained by adjusting the gradation of the mixture. However, this minimum void space obtainable from a soil-aggregate mixture containing coarse aggregates of a given particle index is definitely limited as indicated by the straight line representing the gradation index of 0.50 in Figure 3. Further, the lines in that figure representing the various gradations tend to converge toward the higher values of particle index. Any adjustment in gradation in this range of higher particle index values will, therefore, have little influence on the resultant void characteristics of the compacted samples.

Figure 5 shows the influences that the particle index and the gradation had on the optimum moisture content of various soil-aggregate mixtures. There is a general trend in this plot indicating that soil-aggregate mixtures containing coarse aggregates with a smaller particle index also required a smaller optimum moisture content for effective laboratory compaction. Although Figure 4 shows that the void space is a minimum for a gradation index of 0.5 for all soil-aggregate mixtures, the curves in Figure 5 indicate that optimum moisture content for this gradation is also a minimum and that, with an increase in void space on either side of the optimum gradation index 0.50, there is a corresponding increase in optimum moisture content for effective compaction. Because the presence of moisture in the sample tends to facilitate the rearrangement of the soil and aggregate particles into a condition of minimum void space, it would be expected that mixtures containing particles that were more spherical, rounded, and smoothly surfaced required less moisture to reach this condition. It may also be realized that at a gradation index of 0.50, the particular combination of aggregate and soil particles is such that a minimum of void space is inherent in the composition of the sample. If the void space in the sample is thus naturally minimized at a gradation index of 0.50, it stands to reason that a minimum of water will be present in the sample at this gradation as compared to other gradations.

SUMMARY AND CONCLUSIONS

In this investigation, use has been made of standard laboratory compaction procedure with certain modifications in an attempt to gain information concerning the effect of the geometric characteristics of coarse aggregate particles, as indicated by the "particle index," on the void characteristics of compacted soil-aggregate mixtures. The coarse aggregate materials employed in this study included both pit-run gravel and crushed stone materials. The soil-aggregate materials were artificially prepared according to a mathematical expression to yield nine different and fully controlled gradations. From the results of the testing, the following conclusions have been drawn:

1. The geometric characteristics of the coarse aggregates, as indicated by the "particle index," have a definite bearing on the resultant void characteristics of a compacted soil-aggregate mixture. There appears to be an almost linear relationship between void content in a compacted sample and the particle index of the coarse aggregate. The percentage of voids in a compacted sample increased with increasing values of particle index.
2. For the soil-aggregate mixture containing coarse aggregates of given geometric characteristics, there is an optimum gradation index at which maximum dry density is achieved under a given compactive effort. For a mixture with a ¾-in. maximum aggregate size, this gradation index is equal to 0.50. The optimum moisture content is also a minimum for this gradation.

The data in this investigation indicate convincingly that there is more to consider than gradation in the evaluation of void characteristics and that a definite relationship exists between the particle index of the coarse aggregates and the void characteristics of the compacted soil-aggregate mixture. For a soil-aggregate mixture containing coarse aggregates of given geometric characteristics under a particular compactive effort, it is possible to increase its maximum dry density by varying its gradation. However, the maximum density obtainable is much limited for a mixture containing coarse aggregates having a very high particle index.

ACKNOWLEDGMENT

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Technical advice was provided by a project advisory committee consisting of the following members: W. E. Chastain, Sr., Engineer of Physical Research; Eddy Lund, Soils Engineer, District 1; and C. J. Vranek, Field Engineer, Bureau of Local Roads and Streets, for the Illinois Division of Highways—Norman H. Gundrum, District Engineer; and Arthur F. Haelig, Construction and Maintenance Engineer, for the Bureau of Public Roads—William W. Hay, Professor of Railway Civil Engineering; and Moreland Herrin, Associate Professor of Civil Engineering, for the University of Illinois.

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REFERENCES

1. Huang, E. Y., "In-Situ Stability of Soil-Aggregate Road Materials." Proc., ASCE, Jour. Highway Div., 88: No. HW 1, pp. 43-70.
2. Foster, C. R., "Reduction in Soil Strength with Increase in Density." ASCE Trans., 120:803-822 (1955).
3. Willis, E. A., "Design Requirements for Graded Mixtures Suitable for Road Surfaces and Base Courses." HRB Proc., 18: pt. 2, pp. 206-208 (1938).
4. "Tentative Specifications for Materials for Soil-Aggregate Subbase, Base, and Surface Courses." ASTM Standards 1961, p. 1249 (1961).
5. Huang, E. Y., "A Test for Evaluating the Geometric Characteristics of Coarse Aggregate Particles." ASTM (in press).
6. Talbot, A. N., and Richart, F. E., "The Strength of Concrete— Its Relation to the Cement Aggregates and Water." Univ. of Ill. Engineering Experiment Station Bull. 137 (1923).
7. Ziegler, E. J., "Effect of Material Retained on the No. 4 Sieve on the Compaction Test of Soils." HRB Proc., 28:409-414 (1948).
8. Chamberlin, W. P., and Yoder, E. J., "Effect of Base Course Gradation on Results of Laboratory Pumping Tests." HRB Bull. 202, 59-79 (1958).

Iowa State Compaction Apparatus for Measurement of Small Soil Samples

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A method has been developed for preparing specimens either for use in evaluating additives in soil stabilization studies or for controlling density in field construction. This method, in use at Iowa State University for several years, employs specimens 2 in. in diameter by 2 in. (approximately) high and requires only about one-tenth the material and one-third the time needed for making the standard AASHO-ASTM (or Proctor) specimens.

The density results obtained with this apparatus very closely correlate with the results obtained with the standard method. The results obtained with 17 raw soils and 10 soil-cement mixtures are presented. In addition, the effects of applied compactive energy on the densities of the different types of specimens are given.

• IN 1933, Proctor (1) described one of the first scientific approaches towards the study of soil compaction. He showed that there is a definite relationship between the maximum density to which a soil may be compacted, the amount of energy applied in the compaction process and the moisture content of the soil during compaction. Thus, for a given soil there is a moisture content which, with a given compactive effort, will give a maximum density.

Proctor devised a laboratory test for obtaining the optimum moisture content and maximum density. The compactive energy used in this test was equivalent to that produced by field compaction equipment. Standardized laboratory procedures, equipment, and ways for reporting results were developed. The ASTM Committee E-10 on Standards (2) gave it tentative standard status and the present designation is D 698 - 58 T. The American Association of State Highway Officials (3) accepted this test in 1938 and have listed it in their standards.

The development of heavier compaction equipment, particularly during and after World War II resulted in the creation of a modified laboratory test procedure by the Corps of Engineers. This modified test—commonly referred to as the modified Proctor or modified AASHO moisture-density test—is also standardized by both AASHO and ASTM. Their present designations are AASHO Designation: T 180 - 57, and ASTM Designation: D 1557 - 58 T, respectively.

These standard methods are well known to soil and highway engineers. They are now among the most widely used methods in control and design of highway and airport construction.

Recent expansion in soil stabilization research has increased the demand for soil moisture-density and strength studies. This increased activity has again pointed out drawbacks in these standard tests. Investigations involving these tests require large volumes of soil and relatively long periods of time in which to prepare specimens. Thus, any reliable moisture-density test that would reduce sample and performance

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time requirements would greatly aid soil stabilization research and would also be helpful in construction control. This problem has been studied by many investigators.

OTHER MOISTURE-DENSITY TESTS

In the late 1940's, British engineers studied the Dietert test (4) which was in use in foundry-sand testing. This was adapted to become a laboratory compaction technique (5). The Dietert apparatus consists of a 2-in. diameter mold supported on a metal base by two vertical pegs. About 150 g of soil material passing a ⅛-in. sieve are put into the mold and compacted by dropping an 18-lb weight through 2 in. onto a steel plate covering the soil. Each end of the soil in the mold is compacted with 10 blows. The Dietert apparatus, though more convenient than the AASHO-ASTM tests, has certain disadvantages. The apparatus is rather cumbersome, the specimens for all soils are not the same size and, generally, have higher densities than those obtained with the standard AASHO-ASTM apparatus. No attempt has been made to correlate this apparatus with the modified AASHO-ASTM apparatus.

The Harvard test (6), developed in 1950, employs a miniature compaction apparatus which consists of a cylindrical mold 1.3125 in. in diameter and 2.816 in. long. Soil passing the No. 4 sieve is compacted in this mold by means of a ½-in. diameter steel rod to which a prestressed spring is attached. Soil is added to the mold in five equal increments. The rod is forced into each layer of the soil until the tension on the spring is just released; then the rod is raised, and the cycle is repeated the desired number of times.

This apparatus has certain advantages in that small quantities of soil and little compactive energy are required to prepare each specimen. It is claimed that this apparatus gives moisture-density curves more closely duplicating field compaction curves than those from either the laboratory-dynamic or static methods of compaction. Its main disadvantage is that no single compactive effort or procedure adequately duplicates either field compaction or laboratory density for all types of soils. Thus, before any investigation, a correlation study has to be made. Furthermore, many investigators feel that the reproducibility results obtained with the Harvard apparatus have to be more firmly established.

Two other fairly popular moisture-density test methods have been developed in California. The older method, the California static load test (14), employs about 4,000 g of soil statically compressed in a mold 6 in. in diameter and 8 in. high. A newer method, the California impact apparatus test (2), uses a 2-in. diameter by 36-in. long hinged mold in which the added soil is compacted by the impact of a 10-lb, 2-in. diameter hammer falling through a distance of 18 in. About 2,300 g of air-dry sample are needed for each molding operation. However, both of these methods use as much or more sample than the standard methods—and take as much time or more to perform.

All the preceding procedures have some good features, as well as some bad ones. None of them has all the requirements of an "ideal" compaction apparatus and procedure; that is, an apparatus easily constructed, soil needed in small amounts, little time required to prepare the specimen, and results with good reproducibility.

Accelerated research in soil stabilization led to an attempt to develop such an ideal apparatus by personnel at the Iowa State University Engineering Experiment Station. An apparatus to mold specimens 2 in. in diameter by 2 in. high was first conceived by Davidson and Chu (7) to give densities equivalent to those obtained by means of the standard Proctor technique.* Because preliminary studies (7, 8) showed this apparatus to be feasible, a more complete investigation was carried out to obtain more definite data regarding the use of this apparatus and method.

IOWA STATE COMPACTION TEST

Apparatus

The significant features of the Iowa State compaction apparatus are shown in Figure 1.

Mold.—A cylindrical metal mold having an internal diameter of 2.0 ± 0.001 in. and

*Editor's note: A similar device developed by PCA is described in HRB Proc., 20:824 (1940).

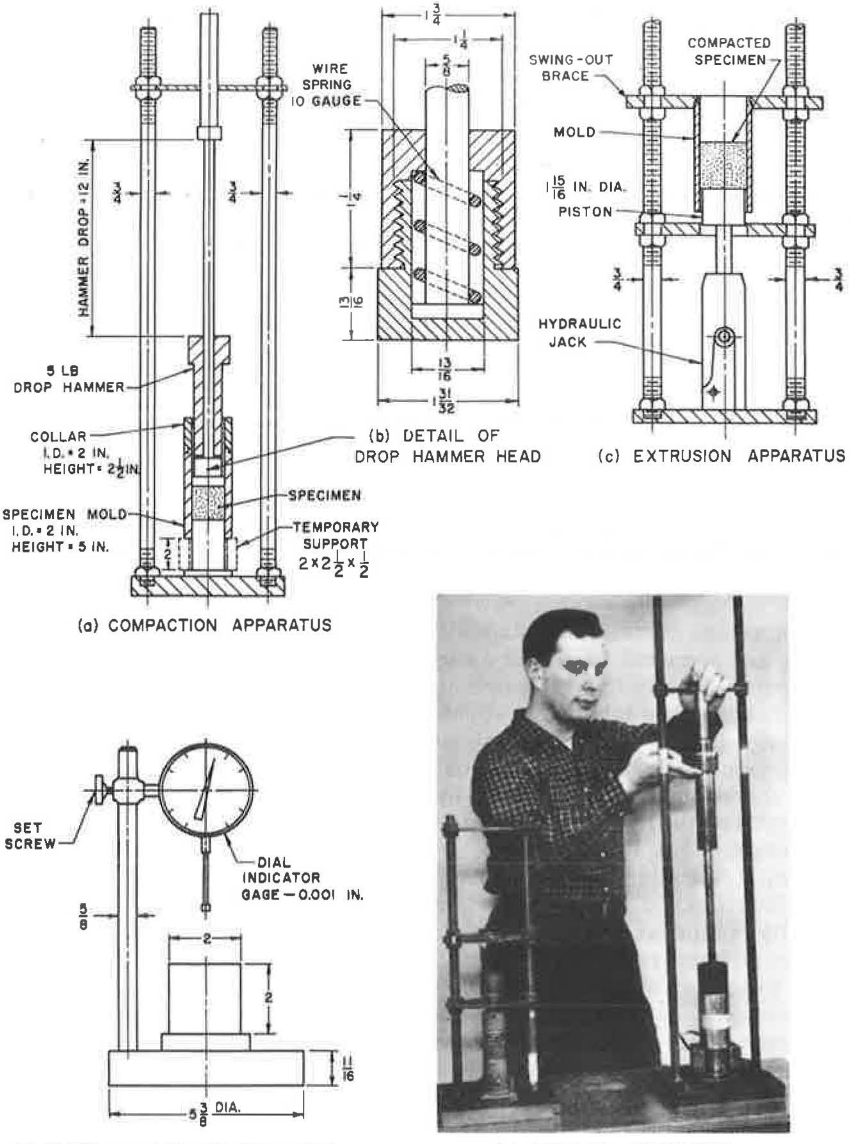

Figure 1. Iowa State compaction apparatus.

and a height of 5.0 in. is used. The mold is provided with a detachable collar that is approximately 2 in. high.

Base.—The cylindrical base has a diameter of $1^{15}/_{16}$ in. and a height of 3.0 in.

Temporary Supports.—The temporary supports are approximately 2 in. in height and are used to hold the mold above the bottom of the base until after the first blow with the hammer.

Frame.—In the frame are two steel rods, a base plate, and a cross-member having a semicircular notch which guides the downward movement of the hammer during compaction.

Hammer.—A 5-lb metal hammer that will drop 12 in. during compaction is used. The entire hammer assembly weighs approximately 10 lb 7 oz.

Extrusion Apparatus.—A hydraulic jack, with a $1^{15}/_{16}$-in. diameter piston, capable of extruding the compacted specimen from the mold without damaging the specimen is used.

Height-Measuring Apparatus.—A dial apparatus capable of measuring the heights of the extruded specimens to the nearest 0.001 in. is used.

Test Procedure

Step 1.—A predetermined amount of air-dry soil or soil-additive mixture is weighed, placed in a mixing bowl, and dry-mixed for 1 min, using a mechanical mixer set at low speed. A Hobart Model C-100, ¼-hp mixer has proved satisfactory for the mixing procedure. After dry mixing, the required amount of moisture is added and wet-mixed for 2 min, then the side of the bowl is scraped, the contents briefly hand-mixed and machine-mixed for another 1 min. With some heavy clays it is necessary to hand-mix entirely.

Step 2.—From this mixture, a predetermined amount of soil is taken sufficient to yield a compacted specimen 2.00 ± 0.05 in. high. This is easily determined by trial and error and is generally in the region of 190 g.

Step 3.—The temporary supports are placed about the cylindrical shaft on the compaction apparatus. The 2-in. diameter mold is set on top of these supports and the detachable collar affixed.

Step 4.—The soil is compacted by dropping the 5-lb hammer through a distance of 12 in. for the required number of times. The temporary supports should be removed after the first blow. For example, if a total of seven blows is required, the following procedure should be followed: one blow is added, the temporary supports removed, three more blows added, the detachable collar removed and the mold inverted, the three remaining blows are then added.

Step 5.—The specimen is extruded, its height is measured to the nearest 0.001 in., and weighed to the nearest 0.05 g. Where the height does not meet the 2 ± 0.05-in. criterion, the specimen should be discarded and another one compacted.

Standard Proctor Density Correlation Study

The standard Proctor[1] density correlation study was divided into two phases:

Phase 1.—The aim of phase 1 was to determine if there was a "true" correlation between the results obtained with the two methods of test. In an effort to minimize as many variables as possible, the following procedures were carried out.

Soils.—Five soils (a sand, a silty loam, a gravelly clay loam and two clays) were used in this study. Their properties are given in Table 1. The soils were chosen to cover a broad spectrum of soil types usable with the Iowa State compaction apparatus.

After each soil was transported to the laboratory, it was air-dried and thoroughly mixed. After being crushed with a rubber hammer, the soil was sieved to remove all particles retained on the No. 4 sieve. The soil was then remixed, after which, by repeated quartering it was divided into 20- to 30-lb batches; each batch then was stored in 2½-gal containers. Inasmuch as this study was expected to extend over a period of months, this method of storing would reduce to a minimum any differential changes in soil-moisture conditions and gradation effects (9).

Cement.—To determine if the Iowa State apparatus would give the same correlation with soil-cement mixtures compacted to standard Proctor densities as with raw soils, two different cement contents were added to each soil. A new batch of cement, sufficient for the entire study, was obtained and kept in a sealed container when not in use. The cement used was Type I. This type of cement is commonly used in soil-cement construction.

Apparatus.—The Iowa State apparatus, scales, etc., were used throughout the study. The Proctor apparatus and method of compaction used conformed to that specified by Method A of ASTM D 698 - 58 T. The same apparatus, mold, etc., were used throughout.

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[1] As used here and in the remainder of the text, the term "Proctor" refers to the test described by ASTM Designation: D 698 - 58 T.

TABLE 1

DESCRIPTION OF NATURAL SOILS

Property	Soil				
	1	2	3	4	5
Source	Harrison Co., Iowa	Story Co., Iowa (Cook's Quarry)	Durham Co., N. C.	Benton Co., Iowa	Livingston Co., Ill.
Textural composition (% by weight):					
Gravel (4.76 - 2 mm)	0	28	0	0	0
Sand (2 - 0.074 mm)	1	17	13	94	10
Silt (0.074 - 0.005 mm)	80	33	22	2	38
Clay (<0.005 mm)	19	22	65	4	52
Organic matter (%)	0.18	0.14	0.27	0.07	0.73
Cation exchange capacity, (minus No. 40 sieve fraction)(meq/100 g):	15.97	9.43	36.2	9.73	15.29
pH	8.51	8.61	5.56	7.50	8.74
Atterberg limits:					
Liquid limit (%)	34	24	74	19	36
Plastic limit (%)	27	13	26	NP	18
Plastic index	7	11	48	NP	18
Predominant clay minerals (X-ray diffraction)[a]	Mo	—	K, H	Mo, I	I
Classification:					
HRB or AASHO	A-4(8)	A-4(5)	A-7-6(20)	A-3(0)	A-6(11)
Unified	ML	CL-SC	CH	SP	CL
Textural	Silty loam	Gravelly clay loam	Clay	Sand	Clay

[a]Mo = montmorillonite; K = kaolinite; h = halloysite; I = illite.

Operator.—Because work done by more than one operator may give varying results, care was taken that all compaction was carried out by one operator (10). In addition, a system of operator controls was devised to evaluate the efficiency uniformity of the operator throughout the investigation.

Mixing and Molding.—After placing a predetermined amount of air-dry soil (and cement, when used) sufficient for three Iowa State specimens and one Proctor specimen in a bowl, it was mixed as specified earlier. While the specimens were being compacted, a damp cloth was placed over the mixing bowl to maintain constancy of moisture content.

Iowa specimens were compacted by means of 3, 6, 8, 10, 12, and 14 blows per side. At least five moisture contents were used to establish a moisture-density curve for each compactive effort. Each optimum moisture content-maximum density curve for the Proctor specimens utilized at least 30 specimens, and each Iowa curve, at a given compactive energy or number of blows, was obtained from at least 18 specimens.

Operator Controls.—After the preparation of each tenth batch a special batch was prepared and was used as a control batch. Each of these control batches contained exactly the same amount of one soil, cement, and water. The purpose of these controls was to measure operator efficiency. They also gave an indication of which was the more reproducible, the Proctor or the Iowa State specimen.

Test Results for Densities.—Figure 2 shows the maximum densities obtained with the Iowa specimens vs the number of blows per side necessary to obtain these densities. These graphs indicate very clearly the effect of increasing the compactive effort on the maximum densities. As the compactive energy is increased, the maximum densities, each obtained from a typical moisture-density curve for a particular number of blows, also increase. In addition for all soils within the range of compactive energies studied the rate of density increase appears to be constant for a given soil. Again, the rate of density increase is greatest with the fine-grained soils, and it lessens progressively with the coarser-grained soils.

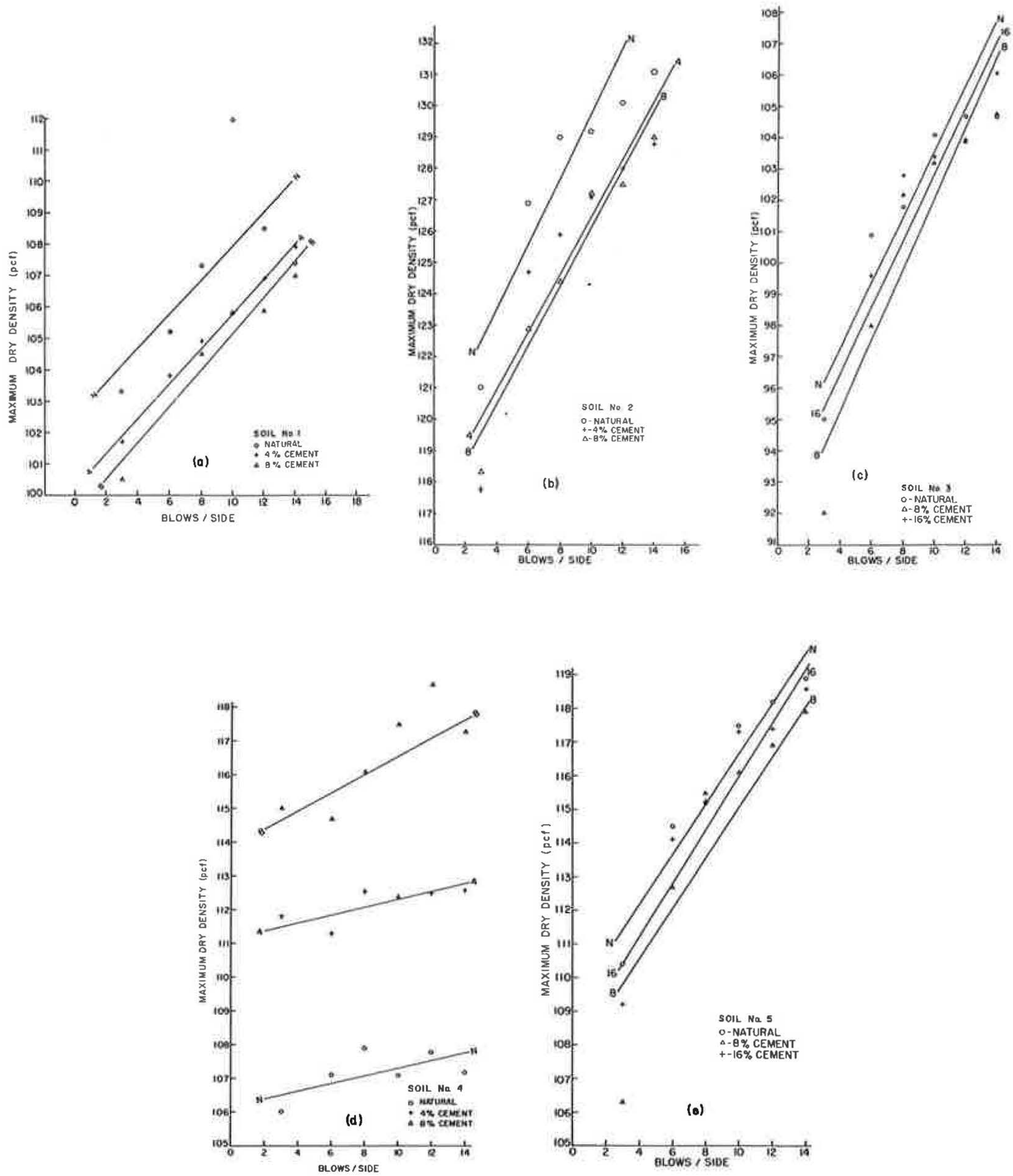

Figure 2. Maximum density-compactive energy relationships, obtained with Iowa State compaction apparatus, soils 1 through 5.

The lines indicated in these graphs were obtained by means of a "least squares" fit; their coefficients of correlation are given in Table 2.

The maximum densities obtained from the Proctor curves are given in Table 3. The number of blows of the Iowa State drop-hammer needed to attain each density is also shown. A problem is how valid is each Proctor maximum density. Davidson (11) has reported that the standard Proctor laboratory test can be performed with an accuracy of ± 4 pcf at least 99.7 percent of the time. A Wisconsin glacial till was used for his accuracy study. This observation would seem to be borne out by the results shown in Figure 3.

TABLE 2

COEFFICIENTS OF CORRELATION INDICATING DEGREE OF RELATIONSHIP BETWEEN MAXIMUM DRY DENSITIES AND COMPACTIVE ENERGIES

Soil No.	Cement Content	Coefficient of Correlation
	0	0.738
1	4[a]	0.997
	8[b]	0.980
	0[c]	0.946
2	4	0.936
	8	0.969
	0	0.958
3	8	0.941
	16	0.963
	0	0.704
4	4	0.841
	8[d]	0.863
	0	0.964
5	8	0.948
	16[e]	0.968

[a]Six blows per side eliminated.
[b]Ten blows per side eliminated.
[c]Fourteen blows per side eliminated.
[d]Twelve blows per side eliminated.
[e]Ten blows per side eliminated.

As mentioned before, control batches were prepared after each tenth regular batch. Each control batch contained exactly 2760 g of Iowa gravelly clay loam, 240 g of Type I cement, and 300 g of distilled water. From each mixture were molded three Iowa test specimens (each compacted with 10 blows per side) and one standard Proctor specimen. Figure 3 is, therefore, a plot of the control number vs dry density for both Iowa State and Proctor specimens. It is clear from these plots that the Iowa State apparatus gives more reproducible results than the Proctor apparatus. All 40 batches gave densities within a spread of 2.7 pcf with the Iowa State apparatus, but the 40 Proctor values enclosed a spread of 7.5 pcf—only 32 of them were within a spread of 3 pcf.

The question arises as to whether this direct comparison is a "fair" one inasmuch as one compactive effort gives densities of about 125 pcf, and the other 113 pcf—and it might be expected the higher density reproducibility results should be better. However, examination and comparison on the basis of the trends exhibited in Figure 3 again indicate that the Iowa State method gives the more consistent results.

To make some allowance for these possible variabilities, the number of blows necessary to attain each Proctor maximum density ± 1½ pcf are also given in the Table.

Test Results for Optimum Moisture Contents.—The optimum moisture contents for maximum Proctor densities are given in Table 3. In a manner similar to that described

TABLE 3

SUMMARY OF CORRELATION TEST RESULTS, PHASE 1

Soil No.	Cement Content	Proctor Test		Iowa State Test		Proctor Test	Iowa State Test		Soil Classification	
		O.M.C. (%)	Max. Density (%)	Total No. of Blows[a]	Equiv. Moist. Content[b] (%)	Max. Density (pcf)	Total No. of Blows[a] Max.	Min.	AASHO	Textural
1	0	18.6	105.8	12	19.2	105.8 ± 1.5	17	7	A-4(8)	Silty
	4	20.3	102.4	8	19.5	102.4 ± 1.5	13	3		loam
	8	19.3	101.8	8	19.9	101.8 ± 1.5	13	3		
2	0	12.2	122.3	6	12.8	122.3 ± 1.5	9	3	A-4(5)	Gravelly
	4	12.9	118.3	2	13.2	118.3 ± 1.5	6	1		clay
	8	14.0	117.4	2	14.1	117.4 ± 1.5	5	1		loam
3	0	24.8	92.5	1	25.0	92.5 ± 1.5	2	1	A-7-6(20)	Clay
	8	10.8	97.5	12	23.6	97.5 ± 1.5	14	9		
	16	9.2	102.3	19	22.3	102.3 ± 1.5	22	16		
4	0	12.9	108.7	42	12.0	108.7 ± 1.5	74	20	A-3(20)	Sand
	4	11.8	113.1	33	12.1	113.1 ± 1.5	62	8		
	8	10.8	115.8	15	11.9	115.8 ± 1.5	26	4		
5	0	16.1	110.6	3	19.1	110.6 ± 1.5	7	1	A-6(11)	Clay
	8	20.2	106.5	1	18.5	106.5 ± 1.5	2	1		
	16	19.4	108.6	3	19.4	108.6 ± 1.5	7	1		

[a]To achieve same density as with Proctor test.
[b]Optimum moisture content to give maximum density with given number of blows.

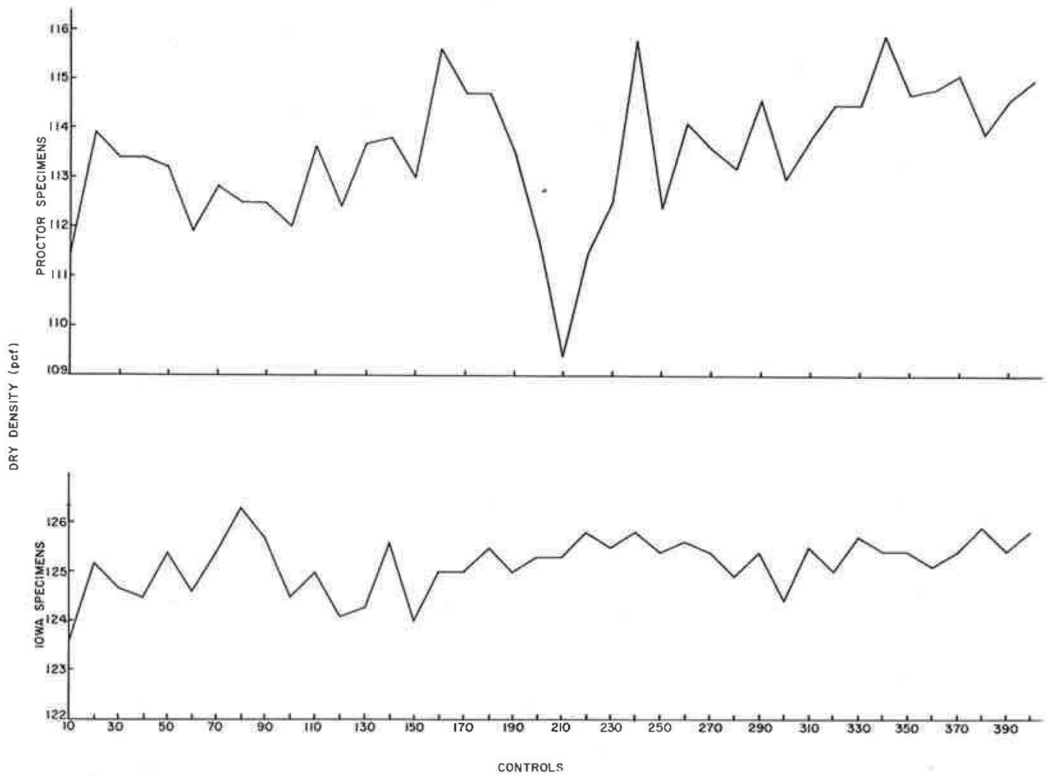

Figure 3. Density-control number relationships for Iowa State and standard Proctor tests.

for densities, the equivalent moisture contents obtained with the Iowa State apparatus were determined and are also shown in the Table.

Summary of Phase 1.—The results in Table 3 indicate quite clearly that the Iowa State compaction apparatus can be used to obtain maximum densities and optimum moisture contents which correspond with those obtained with the Proctor apparatus. However, unlike the Proctor apparatus, the compactive energy (as expressed by the number of blows) may vary from soil to soil.

Phase 2.—The aim of phase 2 was to corroborate the results obtained in phase 1, and at the same time to extend them. Because under normal working conditions, moisture-density relations would not be conducted under strict conditions (that is, by the one operator, batching, etc.), this phase was carried out under what might be considered more usual conditions (12).

Soils.—Eight natural soils (five clays, a gravelly clay loam, a silty loam and a sand) were used. Their properties are given in Table 4. These soils were chosen again so as to cover a broad spectrum of soil types and geographic locations. Soils 1a, 2a and 4a were used in phase 1 also. Soil 10 was originally believed to be sampled from the same source as soil 3, but on analysis its physical properties turned out to be different; therefore, it is treated as a separate soil.

In addition to these natural soils, four artificial soils were produced by means of controlled blending of the natural soils. The properties of these mixtures (a sand, a sandy loam, a gravelly sand, and a gravelly sandy loam) are also given in Table 4.

Preparation of these soils/mixtures was similar to that described for phase 1, except that after final mixing the materials were stored in 200-lb steel containers.

Only raw soils were used here. No cement was added to any mixture.

Apparatus.—Two Iowa State compaction apparatus were used. At different intervals, a different apparatus, scale, etc., were used. The Proctor apparatus and compaction

TABLE 4

DESCRIPTION OF NATURAL SOILS

Soil No.	Natural soil[a]					Combined Soil			
	6	7	8	9	10	12	13	14	15
Source	Harris Co., Texas	Monroe Co., Mich.	Orange Co., Va.	Ringold Co., Iowa	Durham Co., N. C.	—	—	—	—
Textural composition (% by weight):									
Gravel (4.76-2 mm)	0	0	0	0	0	0	0	21.7	35
Sand (2-0.0.74 mm)	3	7	21	21	45	85	54.5	66.7	35.4
Silt (0.074-0.005 mm)	36	36	37	41	18	5	36.9	3.9	24.0
Clay (0.005 mm)	61	57	42	38	37	10	8.6	7.7	5.6
Colloids (<0.001 mm)	37	—	—	—	—	—	—	—	—
Organic matter (%)	0.6	0.6	2.6	0.06	0.1	—	—	—	—
Atterberg limits:									
Liquid limit (%)	65	44	44	41	51	19	19	19	15
Plastic limit (%)	18	21	27	17	26	NP	NP	14	NP
Plasticity index	47	23	17	24	25	NP	NP	5	NP
Specific gravity	2.67	2.68	2.65	—	—	2.65	2.65	2.68	2.68
Predominant clay mineral (X-ray diffraction)[b]	Mo	I, Ch	H	Mo	Mi, K	—	—	—	—
Cation exchange capacity (minus No. 40 sieve fraction)(meq/100 g)	33.1	13.4	12.4	17.5	8.4	—	—	—	—
Classification:									
HRB or AASHO	A-7-6(20)	A-7-6(14)	A-7-6(12)	A-7-6(14)	A-7-6(11)	A-2-4	A-4(2)	A-1-b(0)	A-2-4(0)
Unified	CH	CL	ML-CL	CL	CH-NH, NL-CL	SC-SP	SM	SP-SC or SW-SC	SM-SC
Textural	Clay	Clay	Clay	Clay	Clay	Sand	Sandy loam	Gravelly sand	Gravelly sandy loam

[a]Soils 1a, 2a, and 4a same as soils 1, 2, and 4, respectively, described in Table 1.
[b]Mo = montmorillonite; I = illite; C = chlorite; H = halloysite; Mi = mica; K = kaolinite.

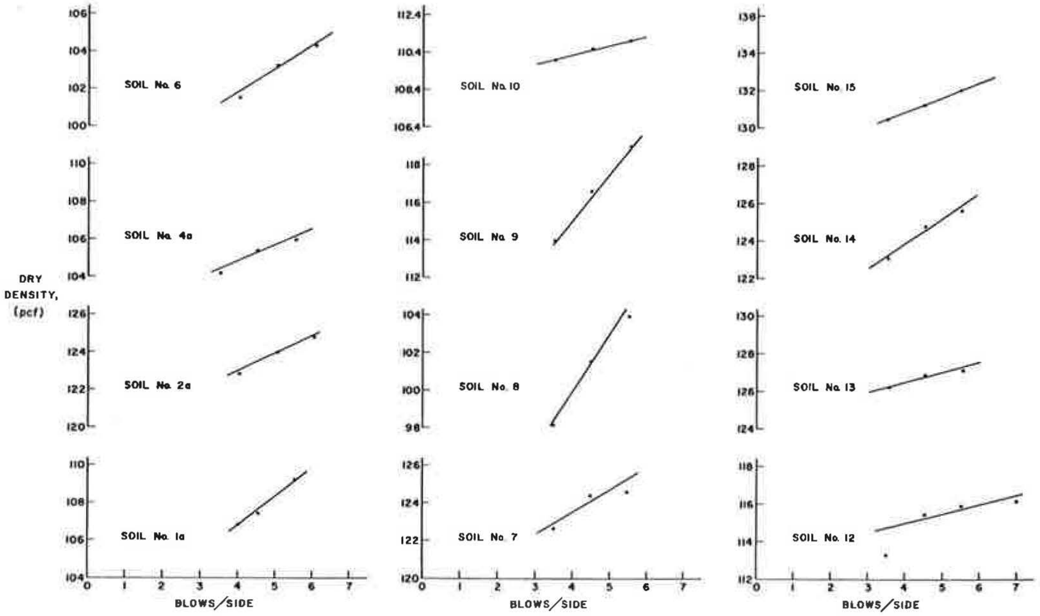

Figure 4. Maximum density-compactive energy relationships, obtained with Iowa State compaction apparatus, soils 6, 7, 8, 9, 10, 1a, 2a, 4a, 12, 13, 14, 15.

method conform to that specified by Method A of ASTM D 698 - 58 T. Three different sets of apparatus were used in this phase of the study.

Operator.—Four different operators were used to compact specimens at varying intervals.

Mixing and Molding.—The mixing and molding procedures were similar to those given earlier in this paper, with the following exceptions.

Proctor specimens were prepared separately from the Iowa State specimens. Each optimum moisture content-maximum density curve for the Proctor specimens used at least five specimens (one specimen giving one point on the curve), whereas each Iowa State curve, at a given compactive energy, was obtained from at least 15 specimens—each point being the average of three values.

Test Results for Densities.—Figure 4 shows the maximum densities from the Iowa State curves vs the number of blows per side necessary to obtain these densities.

These plots again show the effect on maximum densities of increasing the compactive effort. Again, within the range of compactive efforts studied, the rate of maximum density increase appears to be constant within a given soil. Also, the rate of change is greatest with the fine-grained soils and least with the coarse-grained ones. The lines in these graphs are subject to an "eye-fit." Due to the relative paucity of data, a more rigid statistical procedure was not justified.

The maximum densities obtained from the Proctor curves are given in Table 5, as are also the number of blows of the Iowa State compaction hammer needed to attain each density. The number of blows with the Iowa State apparatus needed to attain maximum Proctor densities ± $1\frac{1}{2}$ lb are also given in the Table.

Test Results for Optimum Moisture Contents.—The optimum moisture contents for maximum Proctor densities are given in Table 5. In a manner similar to that described before, the equivalent moisture contents obtained with the Iowa State apparatus were determined and are also given in the Table.

Summary of Phase 2.—The results shown in Table 5 also indicate that the Iowa State compaction apparatus can be used to obtain optimum moisture contents and maximum densities that correspond closely to those obtained with the standard Proctor apparatus and procedure. Again, it is clear that, unlike the Proctor test, the applied compactive energy may vary from soil to soil.

TABLE 5

SUMMARY OF CORRELATION TEST RESULTS, PHASE 2

Soil No.	Proctor Test		Iowa State Test		Proctor Test	Iowa State Test		Soil Classification	
	O.M.C. (%)	Max. Density (pcf)	Total No. of Blows[a]	Equiv. Moist. Content[b] (%)	Max. Density (pcf)	Total No. of Blows[a] Max.	Min.	AASHO	Textural
6	19.8	102.4	9	22.7	102.4 ± 1.5	11	7	A-7-6(20)	Clay
7	13.8	119.1	2	12.7	119.1 ± 1.5	4	1	A-7-6(14)	Clay
8	21.0	98.5	7	23.4	98.5 ± 1.5	8	6	A-7-6(12)	Clay
9	14.6	114.7	7	14.7	114.7 ± 1.5	9	6	A-7-6(14)	Clay
10	15.6	109.5	5	18.4	109.5 ± 1.5	11	1	A-7-6(11)	Clay
2a	11.8	121.3	4	14.6	121.3 ± 1.5	7	1	A-4(5)	Gravelly clay loam
1a	16.4	104.7	5	17.4	104.7 ± 1.5	7	3	A-4(8)	Silty loam
4a	12.9	111.1	22	12.0	111.1 ± 1.5	25	19	A-3(0)	Sand
12	11.0	116.8	16	10.0	116.8 ± 1.5	21	9	A-2-4	Sand
13	9.7	125.7	5	11.0	125.7 ± 1.5	10	1	A-4(2)	Sandy loam
14	8.9	127.1	13	9.4	127.1 ± 1.5	15	10	A-1-b(0)	Gravelly sand
15	7.8	132.5	12	7.6	132.5 ± 1.5	15	8	A-2-4(0)	Gravelly sandy loam

[a] To achieve same density as with Proctor test.
[b] Optimum moisture content to give maximum density with given number of blows.

Combination of Phases 1 and 2.—In combining phases 1 and 2, it must be kept in mind that phase 1 was a very carefully controlled experiment, and phase 2 was what might be considered a routine experiment. Also, three of the soils evaluated were used in both experimental phases.

Energy Recommendations.—If the Iowa State test is to be used to simulate the standard Proctor test, it can only do so if the compactive energy (as reflected by the number of blows) is varied. The amounts of variation depend on the characteristics of the soils being tested. The questions then arise as to how the compactive energy should be varied and what the soil characteristics are that cause these changes.

Examination of the more obvious chemical characteristics indicates no relationships. Most of the more common clay minerals are represented, but they appear to have little effect on compactive energies. Also, within the range studied, the amount of organic matter appears to have no influence.

There seems to be little doubt that the physical characteristics of the soil have the most influence. The question is therefore what the simplest breakdown of these characteristics is that justifies compactive energy changes. One of the most widely used soil classification systems is the AASHO system. Although this system has certain debatable features, it has the advantages of being easy to use and of having such wide acceptance that most soil and highway laboratories use it as an aid towards classifying their soil materials as a matter of routine.

Based on this system the following recommendations are given (Table 6) regarding the number of blows of the Iowa State drop-hammer to use with the various soil types.

Where an even number of blows is recommended for a particular soil, this means that one-half the number of blows should be applied to one end of the specimen and the other half should be applied to the other end after the mold is inverted.

TABLE 6

IOWA STATE COMPACTION TEST ENERGIES TO USE WITH VARIOUS SOIL TYPES

Total No. of Blows[a]	Soil Type[b]
6	A7, A6
7	A4
14	A3, A2, A1

[a] Drop-hammer weighs 5 lb and falls through height of 12 in.
[b] Based on AASHO system; soil classified after being passed through No. 4 sieve.

TABLE 7

SUMMARY OF CORRELATION TEST RESULTS

Soil No.	Cement Content (%)	Proctor Test O.M.C. (%)	Proctor Test Max. Density (pcf)	Iowa State Test Recommended No. of Blows[a]	Iowa State Test Equiv. Density[b] (pcf)	Iowa State Test Equiv. Moist. Content[c] (%)	Density Differences (pcf)	Moisture Content Differences (%)	Soil Classification, AASHO
3	0	24.8	92.5	6	96.2	24.4	3.7	-0.4	A-7-6(20)
	8	10.8	97.5	6	94.2	25.2	-3.3	14.4	
	16	9.2	102.3	6	95.4	24.7	-6.9	15.5	
5	0	16.1	110.6	6	111.4	18.5	0.8	2.4	A-6(11)
	8	20.2	106.5	6	109.8	17.9	3.3	-2.3	
	16	19.4	108.6	6	110.3	18.9	1.7	-0.5	
6	0	19.8	102.4	6	100.3	23.6	0.9	3.8	A-7-6(20)
7	0	13.8	119.1	6	122.0	12.8	2.9	-1.0	A-7-6(14)
8	0	21.0	98.5	6	97.0	23.0	-1.5	2.0	A-7-6(12)
9	0	14.6	114.7	6	112.8	14.9	-1.9	0.3	A-7-6(14)
10	0	15.6	109.5	6	109.8	18.0	0.3	2.4	A-7-6(11)
1	0	18.6	105.8	7	104.5	20.0	-1.3	1.4	A-4(8)
	4	20.3	102.4	7	102.2	19.0	-0.2	-1.3	
	8	19.3	101.8	7	101.3	20.0	-0.5	0.7	
2	0	12.2	122.3	7	123.0	12.7	0.7	0.5	A-4(5)
	4	12.9	118.3	7	120.5	12.8	2.2	-0.1	
	8	14.0	117.4	7	119.9	13.6	2.5	-0.4	
1a	0	16.4	104.7	7	106.1	17.2	1.4	0.8	A-4(8)
2a	0	11.8	121.3	7	122.5	13.1	1.2	1.3	A-4(5)
13	0	9.7	125.7	7	126.3	10.9	0.6	1.2	A-4(2)
4	0	12.9	108.7	14	107.5	14.5	-1.6	1.6	A-3(0)
	4	11.8	113.1	14	111.9	14.0	-1.2	2.2	
	8	10.8	115.8	14	115.7	12.0	-0.1	1.2	
4a	0	12.9	111.1	14	107.5	13.2	-3.6	0.3	A-3(0)
12	0	11.0	116.8	14	116.4	10.5	-0.4	-0.5	A-2-4
15	0	7.8	132.5	14	133.3	6.8	0.8	-1.0	A-2-4(0)
14	0	8.9	127.1	14	127.9	9.5	0.8	0.6	A-1-b(0)

[a]To achieve near maximum Proctor density.
[b]Maximum density attained at given number of blows.
[c]Optimum moisture content to attain maximum density with given number of blows.

Where an odd number of blows is recommended, the larger "half" of these blows should first be applied, then the mold inverted and the remaining blows applied.

The temporary supports (as specified under "apparatus") should always be removed after the very first blow is applied.

Analysis.—A summary of the data obtained when the recommended compactive energies are applied to the investigated soils is given in Table 7.

Close correlation between the densities obtained with the Iowa State and the Proctor procedures is found with most mixtures, excepting those containing soil 3. This soil was originally believed to have come from the same source as soil 10 which shows good correlation; however, the physical characteristics of the two soils show them to be quite apart. These differences are strongly reflected in the soil 3, cement mixtures; why the density and moisture differences should be so great is not known at this time.

Omitting the mixtures containing soil 3, 46 percent of the densities achieved with the Iowa State test are within 0.9 pcf, 75 percent are within 1.7 pcf, and 87.5 percent are within 2.5 pcf of those maximum densities achieved with the standard Proctor method of test. Similarly, 46 percent of the optimum moisture contents obtained with the Iowa State apparatus are within 1 percentage point, 75 percent are within 1.5 percentage points, and 96 percent are within 2.5 percentage points of the optimum moisture contents achieved with the standard Proctor test.

At this stage, the results obtained in this study are not being interpreted on a "pure" energy basis. Correlation on the basis of an exact energy comparison was not feasible at the time of the study due to the impracticalities involved in evaluating (a) the friction factor as reflected by the compaction hammer-soil-mold effect and (b) the exact effect of the spring being incorporated in the compaction hammerhead.

Reproducibility Studies.—Figure 3 shows control specimen numbers vs the attained densities. All these controls were compacted by the same operator using the same apparatus, and were prepared at regular intervals throughout phase 1 of the study which lasted over 4 months. It is obvious that when only one operator and one set of apparatus are involved, and if careful mixing and batching procedures are carried out, the Iowa State apparatus gives more precise results than the standard Proctor apparatus.

When different operators and different Iowa State apparatus sets are involved, and if there is less control over mixing, sampling, and batching, the reproducibility results are not so striking. However, comparison of the raw soil results for soils 1 and 1a, 2 and 2a, 4 and 4a indicate again the superior reproducibility of the Iowa State densities. This is most noticeable when comparing the results obtained with the most difficult of these three soils (No. 4), which is a sand.

Time and Sample Requirements

The Iowa State method takes less materials and is more rapid for studying moisture-density relations. In addition, there is less expenditure of operator energy and less operator fatigue when the Iowa State method is used. This results in more efficient operator output and greater value for the dollar spent.

The following observations were made for comparative purposes during the course of phase 2 of the study. The time required for two technicians, after mixing, to compact, weigh and record data for one standard Proctor specimen is 5.4 min and for one Iowa State specimen is 1.4 min. Between 1,800 and 2,500 g of soil are needed for each Proctor specimen at each moisture content; but only from 180 to 250 g are required for each Iowa State specimen. If, as was done in this study, three 2-in. diameter by 2-in. high specimens are prepared at each moisture content, and if 100 g are used for a moisture content determination, the sample requirement is still only 40 percent of that required for the standard Proctor test; and the time requirement is still more than 25 percent less.

Another factor is that, in moisture-density studies, the Iowa State specimen is small enough to be used for the moisture content sample. This increases investigation accuracy and efficiency under the following circumstances:

1. As a larger soil-moisture sample is taken.
2. As sampling error is eliminated (as is done in many cases) the soil-moisture sample is taken directly from the mixing bowl.
3. As one weighing procedure is eliminated, because the specimen has already been weighed for density purposes.

Use as Soil-Additive Evaluation Test

Although not investigated to a great extent in this study, it is obvious that the Iowa State compaction apparatus may be used to prepare specimens for soil-additive strength evaluation purposes.

Moisture-Density Test.—Figure 2 and Table 7 show that the addition of cement to soils has little effect on the compaction characteristics. Figure 2 shows that for a given soil, the effect on density of the raw soil/compactive energy relationship is the same as that caused by the soil-cement mixture/compactive energy relationship. Table 7 shows that, in general, the same compactive energies may be applied to the soil-cement mixtures as are applied to the raw soil mixtures to attain their equivalent standard Proctor maximum densities.

Strength Test.—The unconfined compression test, which is also simple in procedure and requires little special equipment, is used by many investigators in the study of stabilized soils. Though specimens having a height to diameter ratio of 2:1 are most desirable for this test, there is no reason why the Iowa State specimens may not be used in the preliminary evaluation of the effects of certain additives. After using these more easily prepared specimens for determining the more obvious detrimental or helpful additives or determining suitable percentages of a particular additive, limited numbers of the usual 2:1 cylindrical specimens need only be prepared for the final "finer" analysis.

Other Uses

The Iowa State compaction apparatus has proved its usefulness and convenience for over seven years at Iowa State University. This has led to extensive research regarding other uses for it, besides being used in moisture-density and unconfined compressive strength studies. The apparatus is being modified to give densities correlating with those obtained with the modified AASHO (Proctor) test procedure. A miniature bearing test, capable of simulating the California Bearing Ratio test, has been developed and is being evaluated. In addition, a practical freeze and thaw test, similar to that used in England and Belgium but using the 2-in. diameter by 2-in. high specimen, is being extensively studied (13).

SUMMARY AND CONCLUSIONS

1. The Iowa State compaction apparatus and procedure can be used within field- and laboratory-attainable accuracies to obtain the same maximum dry densities and optimum moisture contents of soils as determined by the standard AASHO-ASTM (or standard Proctor) test.
2. The Iowa State compaction test can be used within field- and laboratory-attainable accuracies to obtain the same maximum dry densities and optimum moisture contents of soil-cement mixtures as determined by the standard AASHO-ASTM (or standard Proctor) test. There appears to be no reason why it cannot be used in other soil-additive/moisture density studies.
3. Use of the Iowa State compaction test to obtain standard Proctor moisture-density relationships requires that the compactive effort be varied according to soil type. The Iowa State compaction test energies to use with various soil types are given in Table 6. The soil type is based on the AASHO system; the soil is classified after being passed through the No. 4 sieve.
4. The Iowa State compaction test is reliable; it is more precise, and it requires less time and materials than the standard Proctor test.
5. There is a straight-line relationship between compactive energies and the maximum densities attained at these energies, and between compactive energies and the optimum moisture contents attained at these energies. As the compactive energy increases, the maximum density also increases. As the compactive energy increases, the optimum moisture contents decrease.
6. The Iowa State apparatus and procedures can be used very readily and economically in soil stabilization strength studies involving the unconfined compressive strength test.

ACKNOWLEDGMENTS

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REFERENCES

1. Proctor, R. R., "Fundamental Principles of Soil Compaction." Eng. News-Record, 111:No. 9, pp. 245-248 (1933).
2. "1961 Book of ASTM Standards." Part 4, American Society for Testing and Materials (1961).
3. "Tests and Specifications." Part 3, American Society of State Highway Officials, 7th ed. (1958).
4. "Soil Mechanics for Road Engineers." Great Britain, Department of Scientific and Industrial Research, Road Research Laboratory (1952).

5. Little, A. L., "Laboratory Compaction Technique." Internat. Conf. on Soil Mechanics and Foundation Engineering, Proc., 2:224-226 (1948).
6. Wilson, S. D., "Small Soil Compaction Apparatus Duplicates Field Results Closely." Eng. News-Record, 145:No. 18, pp. 34-35 (1950).
7. Chu, T. Y., and Davidson, D. T., "Some Laboratory Tests for the Evaluation of Stabilized Soils." Iowa Engineering Experiment Station Bull. 192, 243-256 (1960).
8. Viskochil, R. H., "Effects of Density on Unconfined Compressive Strength, Absorption and Volume Change of Lime and Fly Ash Stabilized Soils." M. S. thesis, Iowa State University (1962).
9. O'Flaherty, C. A., "New Techniques for Data Evaluation and Control in Soil Engineering Investigations." Ph. D. thesis, Iowa State University (1962).
10. Edgar, C. E., "Use of the Iowa State Drop-Hammer Apparatus in the Study of the Moisture-Density Relations of Soils and Soil-Cement." M. S. thesis, Iowa State University (1962).
11. Davidson, D. T., and Chu, T. Y., "Calculations of Standard Proctor Density and Optimum Moisture Content from Mechanical Analysis, Shrinkage Factors, and Plasticity Index." HRB Proc., 29:480-481 (1949).
12. Austin, K. B., "Use of the Iowa Drop-Hammer Apparatus in Studying Moisture-Density Relations of Soils." M. S. thesis, Iowa State University (1961).
13. George, K. P., "Development of a Freeze-Thaw Test for Evaluating Stabilized Soil." M. S. thesis, Iowa State University (1961).
14. Porter, O. J., "The Preparation of Subgrades." HRB Proc., 18:324-331 (1938).
15. Catton, M. D., "Research on the Physical Relations of Soil and Soil-Cement Mixtures." HRB Proc., 20:821-855 (1940).

Discussion

W. H. CAMPEN and L. G. ERICKSON, Omaha Testing Laboratories, Omaha, Nebraska—The authors are to be complimented for doing a tremendous amount of research in an orderly manner. However, as often is the case, the end results are the opposite of what was anticipated. In other words, their data show that the proposed method does not duplicate the results obtained by the standard Proctor method.

To substantiate this contention the authors' data in phase 1 of the research are used because of the better-controlled conditions in that phase. Table 8 shows the relationship between soil type in respect to plasticity index and blows required in the Iowa State method to produce the same maximum density obtained by the standard Proctor method.

The data show that the energy required for maximum density varies inversely with the plasticity index. For instance, nonplastic sandy soil No. 4 requires 42 times more energy than very plastic No. 3. This finding is the reverse of what is known to be the

TABLE 8

RELATIONSHIP BETWEEN SOIL TYPE AND PLASTICITY INDEX AND NUMBER OF BLOWS

Soil No.	Plasticity Index	No. of Blows
4	0	42
1	7	12
2	11	6
5	18	3
3	48	1

TABLE 9

BRITISH FIELD COMPACTION TEST RESULTS

Soil No.	Plasticity Index	Passes Required to Develop 100 Psf Dry Wt.
4	8	1.4
5	19	2.5
6	52	10.5

fact. It is well known by both engineers and contractors that low PI and sandy soils can be compacted easily, whereas heavy clays offer high resistance to compaction.

In 1945, the writers presented a paper (16, Fig. 2) showing that to obtain a dry weight of 110 pcf soil 2 (PI = 10.5) required 235 foot-poundals, whereas soil 13 (PI = 21.1) required 1,170 foot-poundals.

Table 9 gives the results of field compaction tests conducted by the British, showing relationships between plasticity index and energy requirements. Compaction was done with a 9½-ton three-wheel roller.

REFERENCE

16. Campen, W. H., and Smith, J. R., "Bearing Index of Soil as a Criterion for the Maximum Density Requirement." HRB Proc., 25:96-99 (1945).

CHARLES E. EDGAR, III, Closure—The authors wish to thank Messers. Campen and Erickson for their review of this paper. We quite agree with their discussion that sands compact easily in the field. It is precisely for this reason that one would expect a higher laboratory compactive energy (blow) requirement for sands to achieve the duplication of field results. The findings, as presented, speak for themselves. Further, when using this apparatus, there is much more of a "rebound" effect when compacting sands; there is little or no rebound with clay soils. Also, the Iowa State compaction method does not attempt to simulate the method by which maximum density is obtained either by the standard Proctor apparatus or in the field itself—it only attempts to duplicate the end results.

The reviewers have not refuted these findings. Rather, to the contrary, they have added even more support to the conclusions that the Iowa State compaction method will duplicate standard Proctor results.