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Bulletin No. 42

Soil Compaction

1951

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1951

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1951

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COMPACTION OF UNSTABLE MATERIAL WITH HEAVY PNEUMATIC TIRED ROLLER

M. N. Sinacori, *New York State Department of Public Works*

SYNOPSIS

A portion of an arterial project in New York was planned to cross an existing city refuse fill. Because of the inherent nature of the material making up this refuse fill, it was felt that if the usual methods of design and construction were applied to this project, the completed highway would prove to be highly unsatisfactory and possibly quite costly. Of the many methods considered, for a proper solution, the one investigated in detail involved the use of a heavy pneumatic-tired roller to compact the refuse fill from its surface. The method is predicated on the belief that such a roller rolled over the area a sufficient number of times would furnish a compacted crust of such thickness to support a light fill and permit the construction of an adequate pavement.

The pneumatic-tired roller, variable in weight from 12 to 50 tons, was used to roll seven test strips under controlled conditions. In general, it was found that the method is practical. Best rolling results were obtained when the area was rolled in two stages, the first stage required no more than 8 passes of the roller weighing approximately 30 tons, the second stage required up to 30 passes of the 50 ton roller. An average of 2 to 2-1/2 ft. of settlement was observed in the areas rolled completely.

Novel methods were used to measure the effects of rolling on the density of the refuse fill material in place, and on the depth to which rolling was effective. Because of the nature of the material, the taking of samples to measure natural densities in place was not practicable. Two methods used as a substitute were: (a) the electrical resistivity method, and (b) the use of probing rods to measure change in resistance to penetration. The electrical resistivity apparatus showed that rolling with the 30-ton roller was felt to a depth of 6 ft., and that rolling with the 50-ton roller was felt to a depth of 9 to 10 ft. The results of the rod probings showed an increase in resistance to driving to an average depth of 8 ft.

The investigation was conducted at the site of a proposed arterial project in Binghamton, New York. The present foundation for approximately 4,500 feet of the length of this project consists of a city refuse dump fill, varying in age at the time of the investigation from approximately 1 to 15 years, and extending to a depth variable from 5 to 25 feet.

The tests were undertaken to verify the belief that the upper layer of such a heterogeneous, loose fill could be compacted sufficiently by heavy surface rolling to eliminate local weak spots, and to give a uniform bearing value capable of supporting the intended fill embankments without distortion. The data obtained were necessary to determine the feasibility of this procedure of field construction, to prepare estimates of quantities, and to formulate specifications covering the work.

DESCRIPTION OF SITE

The refuse dump-fill site within the limits of the arterial project extends for approximately 4,500 lineal feet along the proposed alinement. The fill is made up of the usual material found in such refuse areas: food refuse, paper and wood, glass, metals (such as tin cans and bed springs), ashes, rubber tires, rags, and other miscellaneous matter. The various percentages of these materials vary widely with depth and in horizontal directions, depending upon the season of the year placed and the rate of dumping. Following the usual practices of land fill construction, a light cover of earth blankets the entire fill. This was placed with no specific effort to obtain compaction. The area consequently is a random, irregular fill, with wide variations in density and load carrying capacities. No record is available of the exact ages of the sections of this fill. The ground water is generally found from 7 to 15 feet below the surface.

Prior to the construction of the refuse fill, this area was the location of a meandering creek in low, flat lands. The material underlying the fill includes layers of partially consolidated muck, silt and clay, over a deep bed of silt, which is interspersed with pockets and layers of sand and gravel. Figure 1 shows the thickness of the refuse fill, and the underlying soil profile within this area.

A review of the existing literature on the characteristics of land refuse fills indicates that the settlement expected in such fills is due to two separate factors. The first of these is the settlement due to physical factors and mechanical adjustments and accounts for the major portion of the settlement.

This physical settlement can be subdivided into two components. The first component is the settlement due to the weight of the fill itself. This value is approximately 30 percent of the height of fill, with 20 percent taking place during the first 6 months, and the remaining 10 percent during the following 18 months. The fill generally reaches stability under its own weight in approximately 2 years. The second component of the physical settlement is that which takes place due to a superimposed load on the fill, such as a highway fill or a structure. This settlement will continue for a period following construction, with the actual amount and rate being a function of the nature of the local fill material and the superimposed load.

The second factor contributing to settlement is that due to decomposition of refuse material in the fill. The amount of this settlement is relatively small, compared to the physical settlement, but it continues to take place for 30 or more years. However, experience has shown that, because of the minor value of this settlement, it is not necessary to wait until ultimate stability is reached before proceeding with the use of the filled area.

CONSTRUCTION METHODS CONSIDERED

A number of methods were considered to permit the construction of the proposed highway across this area. Most of these methods were thought to be too expensive, or inadequate in other respects. Among the methods considered in detail were the following:

1. Removal of the unsuitable material by excavation and backfilling with acceptable borrow. This method was considered very expensive and impractical due to: the large quantities involved; the problem of finding a suitable waste area well beyond the city limits, and its consequent long haul; and the sanitary precautions necessary for handling the material at the site and through the city streets.

2. Building the fill on the foundation in its present state, without any excavation, and untreated in any manner. It was felt that this procedure would result in a pavement reflecting considerable unevenness and local differential settlement, and which would require large expenditures for maintenance after construction. In addition the pavement would generally be rough riding and dangerous.

3. Stabilizing the foundation by the injection of chemicals or grouting mater-

ials. This method was considered to be uncertain and expensive due to the large number of close-spaced holes required for adequate coverage.

4. Surcharging the area with a temporary fill to obtain compression in the refuse material and increase its bearing capacity. Due to the lack of easily available borrow in the vicinity, this method was not considered feasible.

5. Compacting the foundation in place to a greater density and a higher bearing value to the maximum obtainable depth, by a heavy impact or rolling load. Of these, it was felt that a heavy rolling load would be the more practical and economical, and was the method developed and covered in this investigation.

SCOPE OF INVESTIGATION

.Preparations for the investigation were predicated on the belief that a heavy pneumatic tired roller rolled over the area a sufficient number of times would compact the upper portion of the refuse fill to furnish a satisfactory bearing value and designate the local weak spots, which might be treated separately. It was felt that this method would develop a compacted crust several feet in thickness as the upper layer of the foundation, pre-consolidated by the rolling to reduce the settlement expected due to the superimposed fill load. On this would be added several feet of well compacted fill. The combination of these would support satisfactorily a flexible pavement, so as to eliminate abrupt differential settlements and minimize the gradual settlement covering long stretches.

To determine the feasibility of such a method, and to obtain specific information with which to prepare the necessary plans and specifications, field data was required to cover: (1) The effective weight of roller needed to compact the foundation to a satisfactory density; (2) the order of increasing the roller weight to obtain satisfactory rolling without causing too much lateral displacement or deep ruts that would interfere with the rolling operations; (3) the number of roller passes required, (4) the effect of these operations on the refuse fill areas of different ages; (5) the depths to which the rolling was effective in contributing to a compacted crust, (6) the average amount of settlement expected.

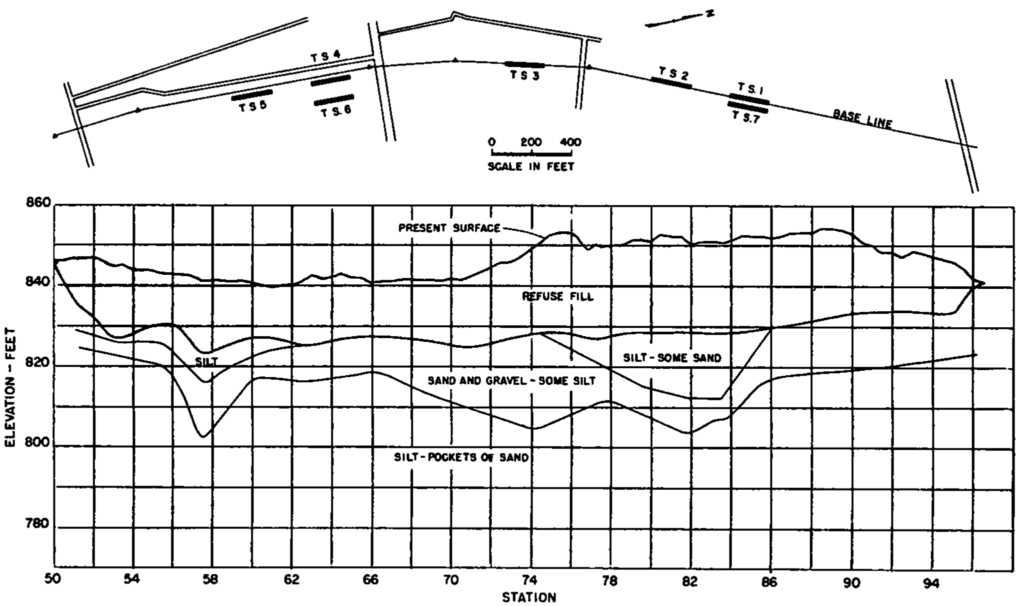

Figure 1. Plan and Base Line Profile

EQUIPMENT USED

Before the investigation was started, some thought was given to the possible type and weight of equipment which could be used to meet the developed requirements. At this time, the William Bros Boiler and Manufacturing Company of Minneapolis, Minnesota, offered the use of their new 50-ton compactor. It was decided to use this heavy pneumatic-tired roller for the investigation, as it was felt that it met the requirements.

The roller consisted of a large, metal box, approximately 17 ft. long by 8 ft. wide, under which were centered two axles in line, each with two rubber tires of size 18:00 x 24-24 ply. A tongue neck extended from the box to two front dolly wheels and a tie-bar connected the dolly wheels to the tractor. The empty weight of the roller was 12 tons, and the box was of such capacity that it could be loaded with saturated sand to a total weight of 50 tons. For this experiment the roller was used at three different weights: empty weighing 12 tons, loaded to half load at 31 tons, and finally loaded to full capacity at a total weight of 50 tons. During the tests, the roller was pulled first by a D-7 Caterpillar tractor, and later by an HD-19 Allis-Chalmers tractor.

TEST PROCEDURES

For the investigation, seven test strips were prepared in various locations on the refuse fill in such a manner that fills of different ages and composition would be included. Each test strip was 200 ft. long and 20 ft. wide. The relative locations of these test strips are shown in Figure 2. After they were laid out, the strips were cross-sectioned and referenced to established bench marks. During the rolling, additional cross-sections and profiles were taken periodically in every test strip to measure the amount of settlement or displacement taking place as a function of the number of roller passes and the weight of roller used.

In the first five test strips, the compaction tests were made in three separate stages. The first stage consisted of rolling the area with 12 full passes of the empty roller, which weighed 12 tons. In the second stage, the roller was loaded to a total weight of 31 tons, and the areas were rolled 12 additional times. Finally, with the roller loaded to its capacity of

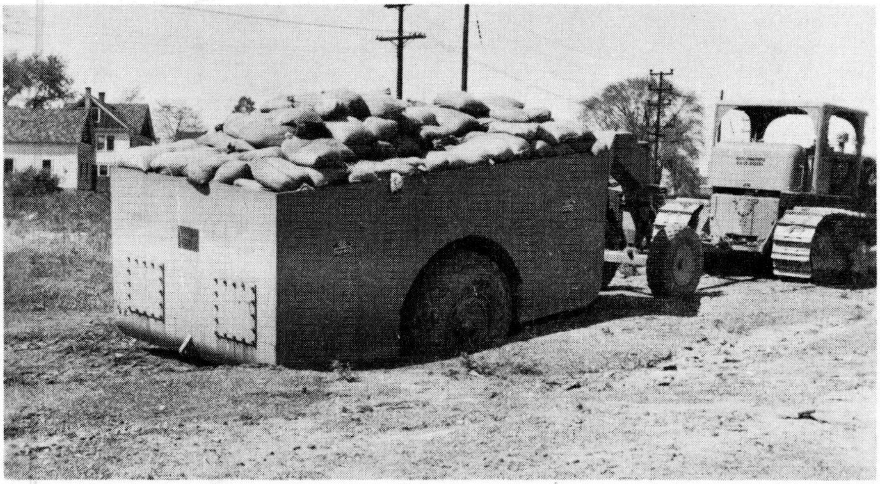

Figure 2. Roller Loaded to 50 Tons Sinking into a Soft Spot

Figure 3. Test Strip 2 Following Rolling with 31-Ton Roller Showing Surface Irregularities Developed

50 tons; this rolling was continued until no measurable settlement was noticed in these test strips. During these operations, local weak spots developed, into which the roller sank as much as 4 feet. These areas were backfilled and leveled off with a thin layer of sand and gravel before the rolling was continued.

The last two test strips were compacted only with the roller loaded to a total weight of 50 tons. It was thought advisable to compare the behavior of the roller fully loaded on material not previously compacted to that on material previously compacted by light rolling and to notice the number of depths of local depressions by a comparison of the two methods.

Figures 2, 3, and 4 show the roller used and the appearance of one of the test strips during the rolling operations.

TEST RESULTS

The test results obtained during the investigation were computed, analyzed, and plotted in several ways to show the required relationships. Each test strip included six control sections across which profiles were taken at 2-ft. intervals within the compacted width of 20 ft. These profiles were taken periodically during the rolling operations and were correlated with the number of passes and the various weights of roller used. Centerline profiles and additional sections at some of the weakest locations between these standard sections were taken as the investigation developed. To maintain control during the rolling operations, the average settlement across each of the six control sections for each test strip was plotted directly in the field against the number of passes given.

In the rolling operations one pass was taken to represent two trips of the roller, each trip offset from the other the width of the one tire to obtain complete area coverage.

Figure 5 shows typical curves of the average settlement obtained across each control section, plotted against the number of roller passes and the corresponding weights of roller. In general, the data show that the 12-ton roller gave an average settlement of 0.3 ft. after 12 passes. The major portion of this settlement developed during the first four to six passes. The

Figure 4. Test Strip 2 Following Rolling with 50-Ton Roller Showing Average Settlement obtained. Surface Irregularities Backfilled.

twelve additional passes of the 31-ton roller gave from 0.6 to 0.7 ft. of additional settlement, for an average total of approximately 1 ft. The major portion of the settlement developed with the 31-ton roller was obtained during the first eight passes. The 50-ton roller further increased the settlement to an average of 2 ft. The major portion of this settlement was realized during the first twenty to twenty five passes of the roller. Further rolling with the 50-ton roller resulted in additional settlement, but the actual amount was relatively small.

Because of the wide variability of the material making up this refuse fill, there developed with the rolling considerable differential settlement and displacement in relatively short distances longitudinally and transversely. The maximum depressions or local settlement obtained in the weak spots ranged from 50 to 100 percent greater than the average values for each section. Figure 6 represents a cross-sectional profile at a typical control section. The bulk of this variation in profile developed as the rolling progressed and was due to the variability of the material and the existance of many local weak spots. There was, however, a definite tendency to develop greater settlements along the centerline of the test strips than toward the outside edges. This accentuation may have been due to the overlapping effects of the rolling, since one coverage over the full width of the test strip was composed of two passes, one pass covering each side of the centerline.

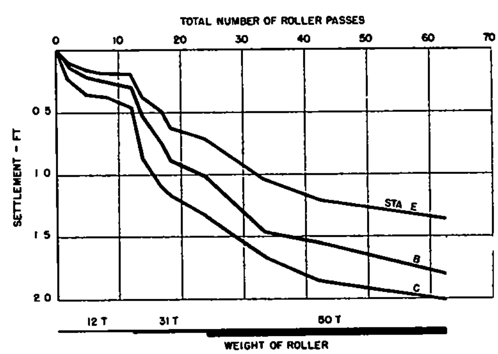

Figure 5. Effects of Roller Weights and Passes on Average Settlement - Test Strip 1

This variability of material encountered and the abrupt variation in settlement obtained during rolling is also represented by Figure 7. This figure shows a centerline profile of settlements for one of the test strips, which is typical of the others.

Two of the strips tested were rolled directly using only the roller loaded to 50 tons, without prior rolling with a lighter weight roller. In general, the amount of settlement obtained after rolling was of the same magnitude as for the strips initially prepared by lighter rolling. However, the direct use of the 50-ton roller on unrolled strips caused severe rutting in the weak spots which bogged down the roller. In these areas, rolling could only be continued under extreme difficulty. Figures 8, 9 and 10 represent three views of an area which was rolled only with the roller loaded to 50 tons.

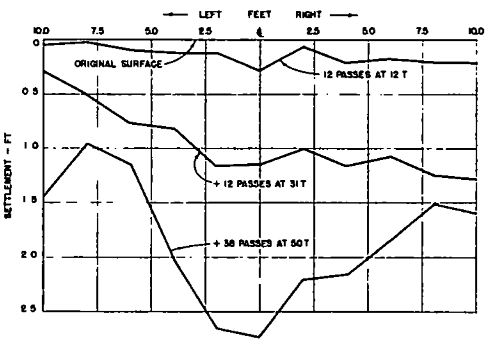

Figure 6. Transverse Settlement Profile Test Strip 4

RELATIVE MEASURE OF DENSITY

Attempts were made to measure the effects of rolling on the density of the refuse fill material in place, and on the depth to which the rolling was effective. The taking of samples to measure natural densities in place was not practicable because of the nature of the material. Two other methods, however, were used to obtain a measure of the relative change in density with depth. The first of these was the use of the electrical resistivity apparatus. The second method was the use of probing rods driven into the ground under standard conditions, and recording the number of blows for each foot of penetration.

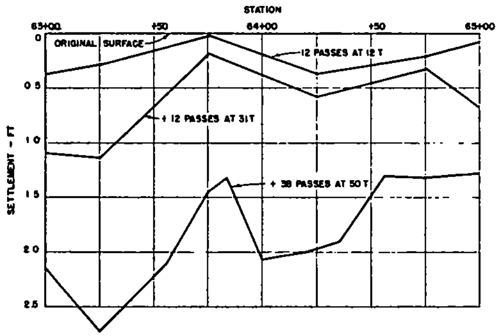

Figure 7. Longitudinal Settlement Profile Test Strip 4

Electrical Resistivity - The use of the electrical resistivity apparatus in itself was an experiment, insofar as it is not known if this method had been used before for such work. The theoretical aspects in the use of this apparatus have been covered in the existing literature and will not be mentioned here. However, the apparatus measures the resistance of an electrical current flowing through the soil material. It is known that, all other factors remaining constant, this resistance is lessened with an increase in the density of that material. In this experiment, two types of tests were used: The first was the point test, in which the electrode spacing at one location, which controls the depth to which measurable effects are produced, was changed progressively from 1 to 20 ft.; the second type of resistivity test made was the traverse test, in which the electrode interval was maintained constant and was progressed across the length of the test area. The point test gives a picture of changing resistance with depth, while the traverse test indicates the horizontal changes at a uniform depth.

Figure 11 is typical of the results obtained from the point tests. The various curves represent the resistance values obtained at the same location prior to rolling, and during the various phases of rolling. Typical results of the traverse tests, which indicate the horizontal changes in resistance at a uniform depth, are given in Figure 12 for an effective depth of 5 ft. and in Figure 13 for an effective depth of 10 ft. It should be pointed out that

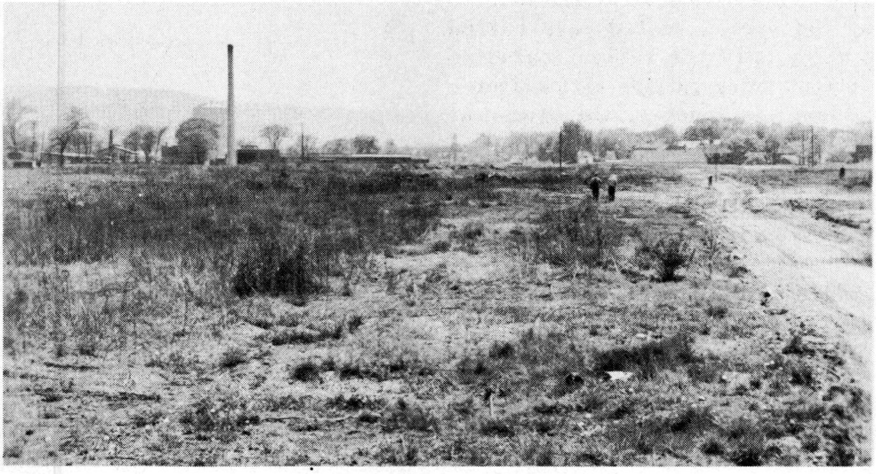

Figure 8. Test Strip 7 Before Rolling

Figure 9. Test Strip 7 Showing 50-Ton Roller Bogging Down During First Pass. No Prior Rolling given.

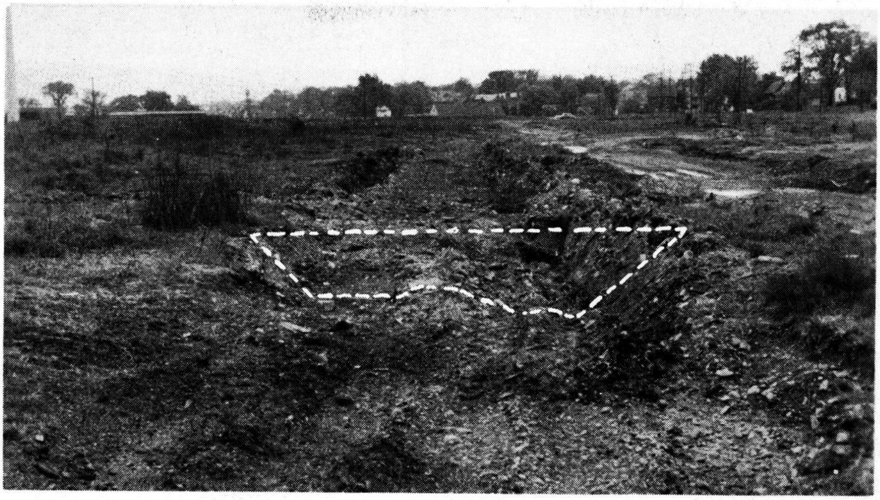

Figure 10. Test Strip 7 after 5 Passes with 50-Ton Roller. No other Rolling given.

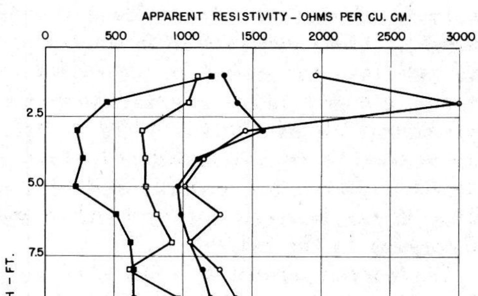

Figure 11. Resistivity Point Test - Test Strip 3 - Sta. 73+50

the results of this portion of the investigation are only qualitative, insofar as the relationship between the values of the resistance of the material to the passage of an electrical current and the corresponding density is not known. In general, the resistivity work was performed at the same locations within the test strips before and following each sequence of rolling. However, due to the lapse of time between these groups of tests, other variables may have come into being, such as rainfall which occurred during that period and which affected the resistance of the material.

Due to local variations, and the methods of analysis used, the data showed that the effects of the decreased resistance were felt to depths up to 14 to 18 ft. However, from a final review of the data, and the conditions under which they were obtained, it was concluded that rolling with the 12-ton roller was effective to an average depth of approximately 4 ft. Rolling with the 31-ton roller was effective to an average depth of approximately 6 ft., and rolling with a 50-ton roller was effective to an average depth of approximately 9 to 10 ft.

Probing Rods - The relative increase in density due to rolling, measured by the use of probing rods, was done by driving into the ground 1-5/8-in. rods with a 250-lb.

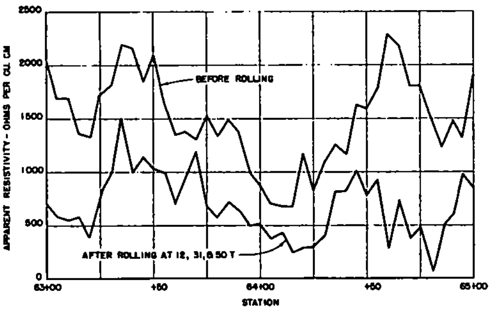

Figure 12. Resistivity Traverse Test At 5 Ft. Depth - Test Strip 4

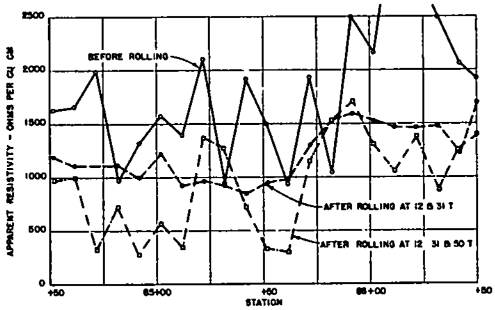

Figure 13 Resistivity Traverse Test At 10 Ft. Depth - Test Strip 1

hammer, dropped through a height of 6 in. The number of blows required to drive the rods for each foot of depth was recorded. The variation in the number of blows was considered a relative measure of the change in density of the material. The probing rods were driven at various locations to include the portions of the fill which had not been rolled, and also others which had been given complete rolling coverage. By comparing the results of these two areas, the increase in resistance to penetration was attributed to an increase in density due to the surface rolling operations. The values of resistance to penetration as a function of the depth below the surface were averaged for the points within each test strip, to obtain a more representative relationship. A group of such average values is shown plotted in Figure 14.

All the work done with probing rods was undertaken after the rolling operations were completed. Consequently, in order to correlate the effects of rolling on the density as measured by the increased resistance to penetration of the rods, two groups of holes were driven in the area tested. The first group of holes was located adjacent to the test strips on material that had not been rolled, and represents the area before rolling. The second group was located along the same stations, but within the rolled test strips, and represents the area after it was rolled. This difference in locations introduced additional variables due to the heterogeneous nature of the material, which were evident in the test results. However, an analysis of the test data showed that the number of blows required to drive the probing rods into the ground in the rolled test strips was generally greater than those driven outside of the rolled test strips. The variation in the number of blows required to drive the probing rods for the first 10 ft. has been averaged and is shown summarized in the table.

The curves generally show that an increase in resistance to penetration has been realized to a depth of approximately 8 ft. The number of blows required to drive the rods through the top 5 ft. in

VARIATION IN NUMBER OF DRIVING BLOWS REQUIRED

	In areas not rolled	In areas completely rolled
Cumulated number of blows to drive probing rods from 0 to 5 ft.	65	130
Average number of blows per ft to drive rods from 0 to 5 ft	13	26
Cumulated number of blows to drive probing rods from 5 to 10 ft.	45	73
Average number of blows per foot to drive rods from 5 to 10 ft.	9	15

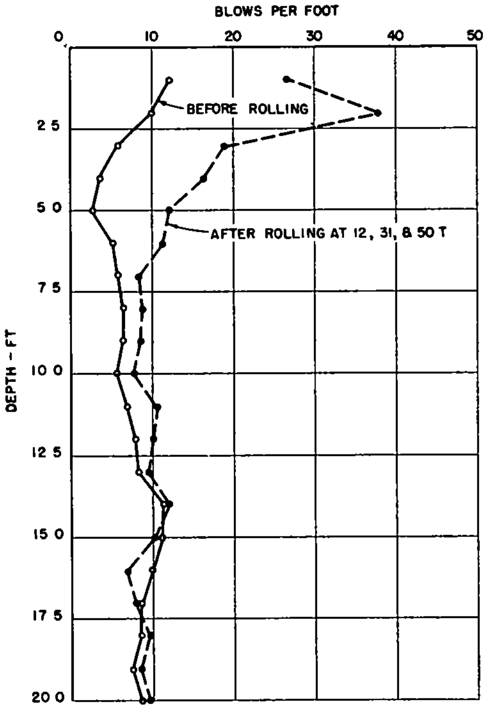

Figure 14. Probing Rod Average Penetration Resistance Test Strip 2

the areas completely rolled was approximately double the corresponding number of blows in the areas not rolled. Beyond a depth of 10 ft., the variations obtained were only within the limits of error assumed.

SUMMARY AND RECOMMENDATIONS

From the results of the investigation, the following summary and basic recommendations were made to aid in the detailed design, quantity estimates, and preparation of specifications for the portion ofthe arterial project within the limits of the refuse fill:

1. The refuse fill within the limits of the arterial project can be pre-consolidated by rolling with a heavy pneumatic tired roller. This rolling should develop a compacted crust sufficiently thick to offer relatively uniform support to a light fill and permit the construction of a servicable pavement.

2. The settlement obtained in the foundation due to rolling with the empty roller weighing 12 tons was relatively small. This value averaged approximately 0.3 ft. over the entire area. It is felt that the 12-ton roller is too light for effective rolling, and should not be used during construction.

3. Rolling the foundation directly with the roller fully loaded to 50 tons without prior rolling with a lighter roller developed many deep ruts and holes with the first few roller passes. In addition to tearing and loosening the material, this method slowed down operations considerably, insofar as too much time was spent in pulling the roller out of these depressions. The 30-ton roller, however, did not cause too much difficulty in this respect.

4. The major portion of the settlement due to the 31-ton roller was realized within the first six to eight passes. The few additional passes increased the amount of settlement very little.

5. Using the 50-ton roller, the major portion of the settlement was obtained in approximately twenty to twenty-five passes. Additional minor settlement was obtained with further rolling.

6. Consequently, for the actual construction, the foundation rolling operations should be carried on in two stages. The first stage should include complete coverage with an average of eight passes of the roller weighing approximately 30 tons. The second stage rolling should then follow using an estimated thirty additional passes of the roller weighing approximately 50 tons. The rolling should cover the area from toe to toe of slope.

7. The rolling operations proceeded most smoothly when all ruts and local depressions which developed during rolling were backfilled with granular material. It was observed during the investigation, however, that if too thick a layer of sand and gravel were spread, it reduced the effectiveness of rolling and prevented additional settlement.

8. Consequently, for the actual construction, the depressions developed in the foundation during rolling operations should be backfilled with selected granular borrow. The quantity should be the minimum required to level off the area and permit continued easy rolling. It is estimated that there may be needed approximately

12 in. of granular borrow over the entire area, with the actual quantity at any location varying with local conditions.

9. The electrical resistivity apparatus showed that rolling with the 30-ton roller was felt to a depth of approximately 6 ft. and that rolling with the 50-ton roller was felt to a depth of 9 to 10 ft. The results of the rod probings showed an increase in resistance to driving to an average depth of 8 ft. There were some areas which showed no change in resistance, while others showed change to a greater depth. Although it is not known what the direct relationship is between the change in resistance to the flow of an electric current and the change in density, nor the relationship between the number of blows on the rods and the density, it has been concluded that a crust has been formed on the surface of the refuse fill which has a greater density and bearing value than before rolling, and which is affected to a depth of at least 5 ft.

10. The results of the investigation show that an average area settlement of 2 to 2-1/2 ft. may be expected due to the foundation rolling. Consequently, sufficient additional common borrow should be provided in the quantities to compensate for this settlement.

11. The age of the refuse fill was not a factor in influencing the results of the rolling. The type and quality of refuse in the fill, on the other hand, controlled the amount of settlement, and the rutting and depressions developed.

12. Wherever local conditions permit, grade elevation should be established a minimum of 4 ft. above present ground line.

13. In cut sections, it was recommended that the area be excavated to 1 ft. below subgrade level before the foundation rolling was started.

14. The rolling operations will be more efficient and economical if the individual areas for rolling are planned of sufficient length to reduce to a minimum the relative time required for turning. The stretches laid out should be approved before rolling to obtain as long a section as field conditions permit. Turn-around areas at the ends of the rolling section should be wide and satisfactorily prepared to reduce the time for turning around, and eliminate the attending side shearing and rutting.

RECOMMENDED TENTATIVE SPECIFICATIONS
SPECIAL EMBANKMENT FOUNDATION ROLLING

Work - Under this item the contractor shall roll the embankment foundation with an approved pneumatic tired roller as directed by the engineer. Approved pneumatic tired equipment for rolling shall be of such capacity that the load may be varied from 30 to 50 tons. This load must be transmitted through two axles acting in a line perpendicular to the centerline of the roller to permit oscillating action. The total axle load shall be transmitted to the ground on four pneumatic tires. The pneumatic tires shall be evenly spaced across the entire width of the roller, and shall be attached two to each axle. The axles shall be so attached to the body of the roller that oscillation will be obtained in each set of two tires. Rollers which permit the individual oscillation of each tire under a proportionally maintained load will also be acceptable.

The rolling shall cover the entire foundation area between the toes of slope between the stations shown on the plans. These stations are approximate and may be varied in the field depending on local conditions. Preliminary rolling shall be done with a 30-ton axle load. It is expected that the amount of preliminary rolling required will be approximately 8 passes over the entire area. The axle load shall then be increased to a total load of 50 tons. Final rolling shall be done with the 50-ton axle load. This rolling shall be continued until the degree of stability of the foundation as required by the engineer has been obtained. It is expected that the amount of rolling required with the roller loaded to a total weight of 50 tons will be approximately 25 to 30 passes over the entire area to be covered.

In rolling, one pass shall be taken to represent two trips of the roller, each trip offset from the other the width of one tire to obtain complete area coverage.

As the rolling progresses, the irregularities between tire marks shall be leveled off to facilitate compaction and to permit complete area coverage by the tires. All local depressions which interfere with the rolling operations shall be backfilled with selected granular borrow. The amount of backfill to be used for these operations shall be kept to the minimum required to permit easier rolling operations. When the condition of the foundation is satisfactory for normal rolling, the speed of the roller shall be not less than 2-1/2 mi. per hr.

Payment - The quantity to be paid for under this item shall be the number of hours of rolling performed by the special rolling equipment. No payment will be made for idle equipment due to repairs, bad weather, wet subgrade, or for any other reason.

The time of rolling shall be recorded to the nearest minute by the contractor. This time shall be checked daily by the engineer.

The unit price bid for this item shall include the cost of furnishing all labor, materials, fuel, equipment, and repairs necessary to complete the work, except the selected granular borrow used during these rolling operations will be paid for under its respective item.

ACKNOWLEDGMENTS

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SOME COMMENTS ON EARTH COMPACTION

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SYNOPSIS

Compaction terminology is examined and revisions proposed. Examples of how to make a choice between the dry and the wet side of the optimum moisture content are given. The commonly used concept of the moisture content "by dry weight" is discussed with the conclusion that the use of this concept does not always indicate the moisture distribution in a subgrade. A British paper on experimental compaction is analyzed and the results of the analysis are illustrated graphically.

This paper summarizes views of the writer on various aspects of compaction and does not represent the results of a systematic study of this subject.

There is a tendency to call all rollers consisting of a drum with protruding feet "sheepsfoot rollers." The term "tamping roller" is also used and is preferable. There are various types of feet used on tamping rollers; the sheepsfoot being only one type. Figure 1 shows four types of roller feet. The cross section of a tapered foot (Fig. 1a) may be of three different shapes (round, rectangular, square). In a variation of the tapered foot one face may coincide with the radial plane of the drum, i.e., be vertical in the utmost lower position. The area of the cross section of a tapered foot is variable, whereas a pegfoot has a constant rectangular cross section (Fig. 1b). Both the clubfoot and sheepsfoot (Figs. 1c and 1d) have the tamping face larger than the cross section of the shank. The term "shank" is used to designate that portion of the foot other than the tamping face.

OPTIMUM MOISTURE CONTENT

Any horizontal line below the point of the optimum moisture content of a moisture-density curve intersects the latter at two points; one is located "on the dry side" and the other "on the wet side" of the optimum (Fig. 2). Though in a general case it is difficult to choose which moisture content to use in the field, in particular cases the answer is clear enough. The following three examples are taken from the practice of the California Division of Highways (1). In this particular case the impact method of compaction was used, which usually results in higher densities and hence lower optimum moisture contents than the AASHO method, T-99-38 (2).

In the construction of a freeway near Sacramento, California, very wet sand excavated in the borrow pit was compacted in hot weather in a single lift 16 in. thick using small pneumatic tired rollers. Placing and compacting the material in 4-in. lifts as required by the specifications, proved to be impossible because the material was drying out very rapidly and was gradually transformed into a mass of loose sand. The writer believes that this example indicates that it is convenient to work on the wet side of the optimum when compacting clean, cohesionless sand in hot weather.

On a project in Santa Barbara County brown, sandy clay was compacted by a heavy tamping roller and pneumatic-tired wobble-wheel roller at a rate of 230 cu.yd.per hr.

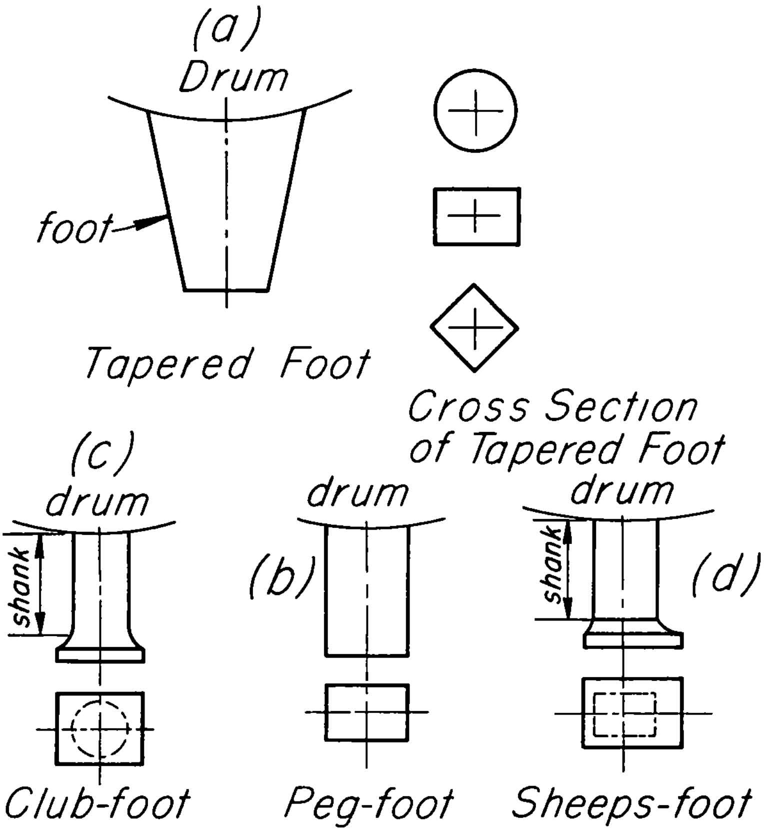

Figure 1. Sketches of Tamping-Roller Feet. (not to scale)

The moisture content was 16 percent, as compared with the laboratory optimum of 13 percent. The relative density obtained was 90 percent. Previous attempts to use lighter tamping rollers failed but removal of the material and its recompaction with heavier rollers gave satisfactory results. The writer believes that this example may be considered as evidence, perhaps to be checked and completed, in favor of using heavy tamping equipment and working on the wet side of the laboratory optimum, in the case of sandy clays or sandy loams.

Embankments in southern California up to 75 ft. high consisting mostly of decomposed granite, were compacted at a moisture content of 8 percent, as compared to the laboratory optimum of 11 percent. A relative density of 93 percent was obtained using a single-axle two-wheeled pneumatic-tired compactor with the tire inflation of 90 psi. The material was not extremely critical to water content in the same way as the old-fashioned water macadam. All such materials apparently can be compacted using the moisture content on the dry side of the optimum.

When a very high embankment is built, gradual addition of fill material may produce a compactive effort greater than the equipment. In such a case, the air from the lower layers of the embankment cannot readily escape. It will therefore, stand under high pressure and be gradually absorbed by the moisture. In fact, Henry's law states that for a given temperature the amount of gas which a liquid will absorb is directly proportional to the pressure of the gas. To avoid saturated condition as caused by that absorption, lower lifts of a high embankment should be built with enough air in them to stand the pressure (3). This may involve work "on the dry side" of the optimum moisture content.

WATER REQUIRED FOR COMPACTED SOIL

The common method of expressing the moisture content in percent of dry weight does not always give a true picture of moisture distribution in an earth mass, particularly in a subgrade. Figures 3 and 4 show moisture distribution in the subgrades of two actual California highways (4). In each of these cases, moisture content has been measured at the bottom of the four 6-in. layers, the moisture content by dry weight increasing downward in the case of Figure 3 and being erratic in the case of Figure 4. Judging from the moisture content by dry weight in both cases, the second 6-in. layer counting from the top is wetter than the uppermost layer. In reality, this is not the case since the amount of water per cu. ft. of earth material is practically the same in both the first and second layer, counting from the top. In fact, this amount as expressed in lb. per cu. ft. is: in the case of Figure 3

first layer 105.0x0.185=19.6 lb.per cu.ft.
second layer 91.0x0.218=19.8 lb.per cu. ft.

in the case of Figure 4

first layer 104.0x0.101=10.5 lb.per cu.ft.
second layer 97.2x0.114=10.1 lb.per cu.ft.

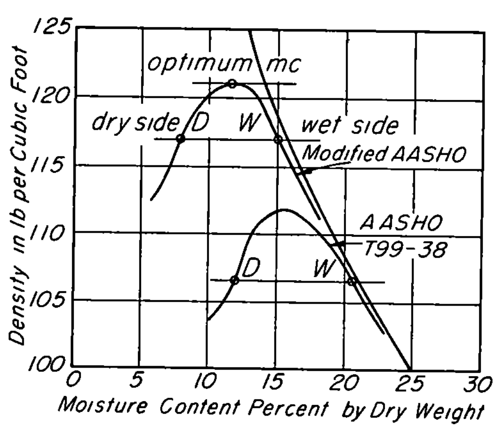

Figure 2. Moisture-Density Curves

Again, the optimum moisture content has different values for different kinds of compactive effort. The expression used by the writer hereafter "required weight of water in lb.per cu.ft." is a convenient abbreviation of the expression "weight of water in lb. required to produce one cu.ft. of earth material compacted to maximum density attainable with a specified compactive effort." This weight is the sum of the weights of water that the material contained before compaction plus water added during compaction less, of course, possible losses during the process of compaction.

ANALYSIS OF PAPER ON COMPACTION

In this paper (5), among other things, a table is given showing the results of the experiments on compaction using three laboratory methods, six field compactive efforts and five different soils. The writer analysed these results and represented the results of this analysis in the form of diagrams (Figs. 6, 7, 8, and 9). The laboratory methods and the field compaction efforts are numbered 1 to 9.

The laboratory methods of compaction are:

1. The British standard compaction test, which is practically identical to the AASHO method, T-99-38;
2. Modified AASHO test;
3. Dietert test (6) used in England and practically identical to a test used in the American ceramic industry.

The field compactive efforts were furnished by:

4. 2 3/4-ton smooth-wheel roller
5. 8-ton smooth-wheel roller
6. Pneumatic-tired roller (tire pressure 36 lb. per sq. in.)
7. Clubfoot tamping roller (foot pressure 115 lb. per sq. in.)
8. Tapered foot tamping roller (foot pressure 250 lb. per sq. in.)
9. 1/2-ton "frog rammer." In the absence of more detailed information on this type of compactive effort the writer believes that it is similar to the device locally known in America as Leaping Lena.

The soils used in the investigations are arranged hereafter in the order of the increasing average grain size that may readily be found from the grain-size distribution diagrams, Figure 5. For the sake of brevity this order is termed hereafter "order of increasing grain size." The opposite would be "order of decreasing grain size." The soils are designated by the symbols of the

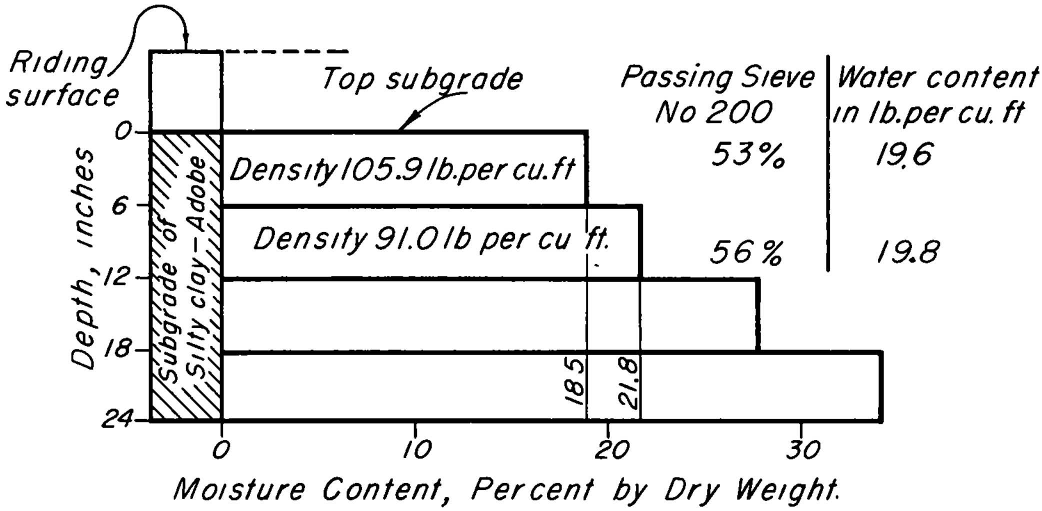

Figure 3. Amount of Water in 1 cu ft of Compacted Soil. (adobe). on a California Highway

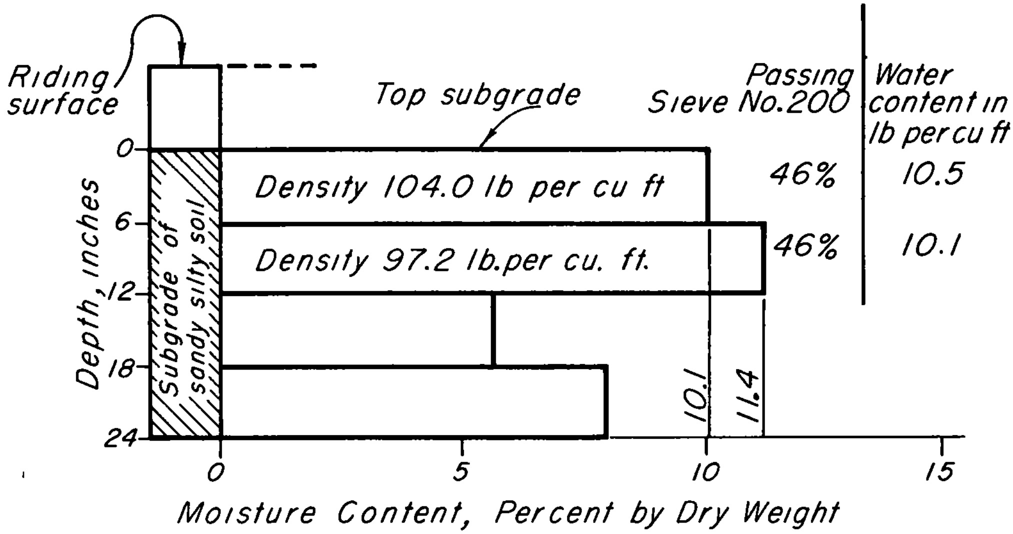

Figure 4. Amount of Water in 1 cu ft. of Compacted Sandy Silt on a California Highway

A-C classification (now replaced by another, also temporary soil classification), as follows: heavy clay CH; silty clay CL; sandy clay ML; sand SW; gravel-sand-clay mixture GW. Thus the average grain size increases gradually from the CH soil to the GW material.

The following table gives the values of the Atterberg limits of these soils, the SW and GW materials being non-plastic or practically non-plastic.

MATERIAL	LL	PL	PI
CH	75	28	47
CL	43	24	19
ML	27	19	8

Figure 6 illustrates the maximum densities that could be attained using different compactive efforts on the experimental soils can be subdivided into two groups: (a) gravel GW and sand SW; and (b) clays ML, CL, CH. The soils of group (a) attained 100 or more percent of the maximum density

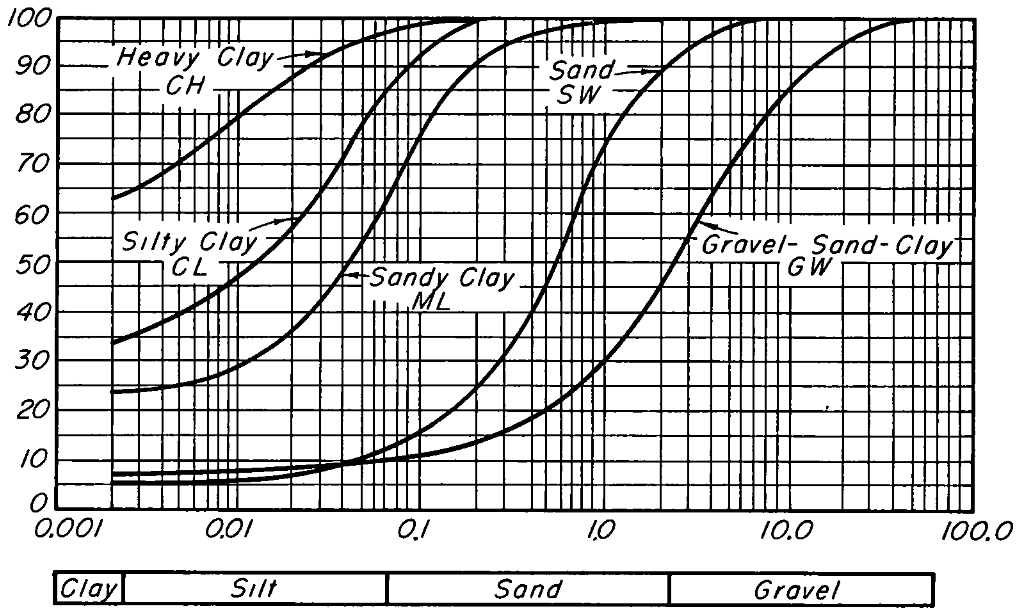

Figure 5. Size Distribution Curves for Materials of Reference 4

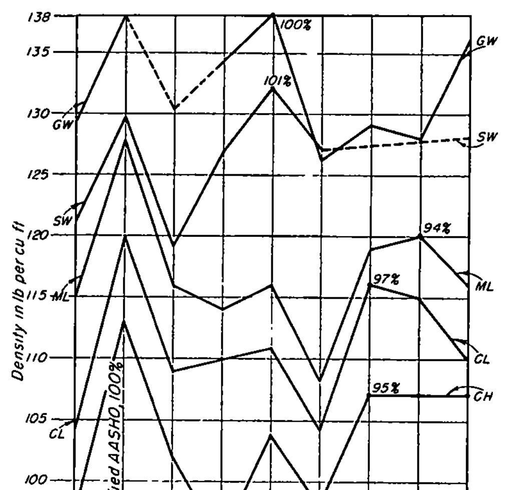

Figure 6. Maximum Densities for Different Compactive Efforts.

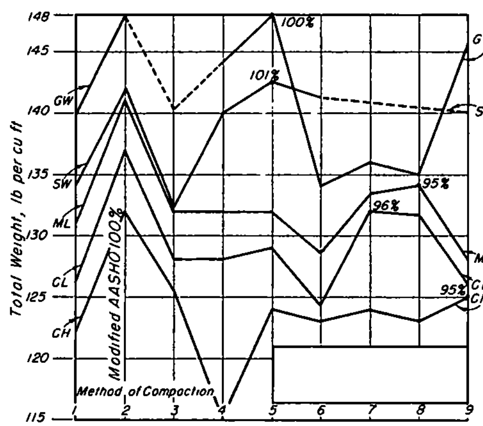

Figure 7. Total Unit Weight of Compacted Material at Maximum Density

as indicated by the modified AASHO test whereas the clays of group (b) could not attain this density though attained values were high (from 94 to 97 percent). It should be borne in mind, however, that in this case experimental compaction with all possible precautions is dealt with, and in a general case such high densities possibly cannot be attained in construction operations. For group (a), i.e., for gravel and sand, the maximum density values were furnished by heavy smooth-wheel roller (compactive effort 5) whereas the corresponding values for clay were attained using tamping rollers (compactive efforts 7 and 8). Apparently heavy, smooth-wheel rollers produce vibrations and wavy action to which sands and gravel are sensitive. This is not the case of cohesive soils, however, that require consolidating action of the roller and readjustment of flat particles for which purpose tamping rollers are more convenient than other types. The writer believes that a detailed research tending to explain the mechanism of earth compaction by various types of compactive efforts is highly desirable. Again, it should be noticed that if heavy modern models of pneumatic rollers were used in the British research under consideration (5) the results might have been different.

As to the performance of various types of roller feet very little difference between clubfeet (compactive effort 7) and tapered feet (compactive effort 8) was observed.

An examination of Figure 6 shows that the maximum densities that could be attained decrease systematically in the order of decreasing grain size. The maximum density attained in the case of the GW-material was 138 lb. per cu. ft. whereas the maximum density for heavy clay (CH) was 95 lb. per cu. ft. only, a range of 43 lb. per cu. ft.

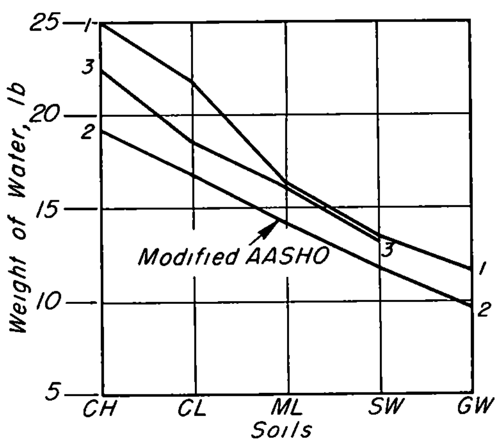

Figure 8. Amount of Water Required to Produce 1 cu. ft. of Compacted Soil by Different Laboratory Methods: (1) B. S. Compaction, (2) Modified AASHO, (3) Dietert.

The bearing value of a subgrade or an embankment is proportional or practically proportional to the unit weight of the earth material, i.e., to the density plus the water content in lb. per cu. ft. For the given series of experiments the total unit weights at maximum density for different soils and different compactive efforts are plotted in Figure 7. The curves in Figures 6 and 7 are in general of similar shape; the subdivision of all experimental soils into two groups holding in both cases. Again, soils of group (a) could attain 100 or more percent of the total unit weight indicated by the modified AASHO method, whereas clays of group (b) could not do it, though a high percentage (95 to 96 percent) of the laboratory total unit weight was at hand. The compactive efforts producing the maximum densities (Fig. 6) were also operative in attaining maximum unit weights of the compacted materials (Fig. 7). The total unit weights of compacted materials decreased systematically from GW-material (148 lb. per cu. ft.) to heavy CH-clay (115 lb. per cu. ft.). The smaller range (33 lb. per cu. ft. as compared with 43 lb. per cu. ft. in Figure 6) is explained by a larger water content

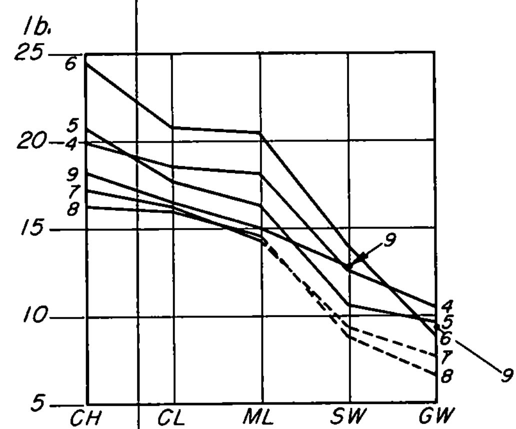

Figure 9. Amount of Water Required to Produce 1 cu. ft. of Compacted Soil by Different Field Compactive Efforts: (4) 2 3/4-ton Smooth-wheel roller, (5) 8-ton smooth-wheel roller, (6) pneumatic-tired roller, (7) club-foot tamping roller, (8) tapered-foot tamping roller, (9) frog rammer

at smaller densities as may be seen from Figure 2.

The required weight of water in lb. per cu. ft. is shown in Figure 8 for the laboratory tests and in Figure 9 for field compaction work. It should be again recalled that the B.S. compaction method is practically identical to the AASHO T-99-38 test method. The required weight of water was the largest for heavy CH-clays and the smallest for the GW-material and decreased systematically in the order of increasing grain size. It may be seen from Figure 9 that the pneumatic-tired rollers (compactive effort 6) required more water than other compactive efforts for all soils with exception of the GW-material. The tamping rollers (compactive efforts 7 and 8) required less water than any other compaction methods for all experimental soils the difference between the compactive efforts 7 and 8 being small.

In this investigation (5) the lift thickness (before compaction) was of 9 in. Dry density was the average dry density of the upper 6 in. generally and only 4 in. for the tamping rollers.

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