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Summer 1948

The Use and Treatment of Granular Backfill

By

ROY L. GREENMAN

Soils Engineer

Michigan State Highway Department

MICHIGAN ENGINEERING EXPERIMENT STATION

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Soils Engineer
Michigan State Highway Department
in cooperation with
The Michigan Engineering Experiment Station
Michigan State College

MICHIGAN STATE COLLEGE
MICHIGAN ENGINEERING EXPERIMENT STATION
EAST LANSING

CONTENTS

FOREWORD

The purpose of this bulletin is to describe the results of an investigation covering "The Use and Treatment of Granular Backfill." The study was undertaken to determine the causes of settlement, particularly in bridge and culvert approaches, and to find the most efficient method or methods of preventing it.

The principal subjects treated in this bulletin, that is, gradation and consolidation, may seem to the reader so unrelated as to justify the writing of two separate papers. However, in the course of the investigation it was found that to write on the study of gradation of granular material and say nothing about the consolidation of these materials would be to leave the job only half done. Likewise, to write on the subject of consolidation of all granular materials and to make no mention of gradation would be equally incomplete. Therefore, to do justice to the study and development of granular backfills and embankments, a combined report covering both gradation and consolidation seemed appropriate.

This investigation is part of an extensive research program concerning the design and construction of pavement foundations which is being conducted jointly by the Michigan State Highway Department, of which Charles M. Ziegler is State Highway Commissioner, and the Michigan State College Engineering Experiment Station, of which Dean H. B. Dirks is Director. The gradation study, as well as consolidation studies of certain materials, was carried out in the State Highway Testing Laboratory in Ann Arbor under the direction of G. O. Kerkhoff, Soils Engineer. The major work of consolidation was performed at the Michigan State Highway Research Laboratory at East Lansing, and on construction projects in the field under the immediate supervision of Gail Blomquist, formerly Physical Research Engineer in the Highway Department, and the author. The Michigan State Highway Testing and Research Laboratories are functional units of the Department's Testing and Research Division, which is under the direction of W. W. McLaughlin, Testing and Research Engineer.

The author wishes to express his gratitude to all members of the Highway Department who assisted him in the study and in particular to E. A. Finney, Director of the Research Laboratory, for his helpful suggestions and guidance; to the Mason County Road Commission, for the loan of hauling and loading equipment; to the Electric Tamper and Equipment Company of Ludington, Michigan, who cooperated by furnishing vibratory equipment; and to Warren W. McNicol, then a senior at Michigan State College, who conducted the laboratory consolidation studies.

The Use and Treatment of Granular Backfill

By ROY L. GREENMAN

Settlement of trench backfills, bridge approaches and embankments has long been an obstacle to the attainment of the smooth-riding highway sought by designers, engineers and contractors. Specifications for the requirements of gradation and compaction have been changed from time to time in an attempt to eliminate highway irregularities caused by settlement of the pavement foundation. In spite of all of these precautions, settlement over trenches, at bridge approaches, and in large fills continues to occur. Trench backfills and bridge approaches have settled to the point of being traffic hazards, and many dollars are spent each year in patching up these areas.

The Michigan State Highway Department believed, a few years ago, that the answer to the problem had been found when granular backfill was required in all places where there was any possibility of detrimental settlement. However, settlement continued in these areas throughout the state, not as serious as previously, but still sufficient to be a menace to the fast and heavy traffic of today. Having in mind the hazards presented to traffic by these conditions, the Michigan State Highway Department began a comprehensive investigation to determine the causes of settlement, and to find the material, and the method or methods for placing it, that would best eliminate settlement.

The investigation embraced a series of individual studies including: 1) the drainage characteristics of backfill materials as influenced by gradation, with the view of establishing proper specifications for such materials, 2) laboratory consolidation studies on various types of granular backfill materials involving the use of different types of compaction methods to determine the most efficient method, and 3) field studies of vibratory methods for consolidation of granular backfill material under actual construction conditions.

It was found that settlement or shrinkage of granular backfill may be due to loss of material by filtration, by consolidation of the material, or by a combination of both factors. It has been definitely established that any type of granular material, if placed loose, will undergo a rearrangement of particles when subjected to traffic vibrations. The rearrangement of the particles will result in settlement, varying in amount depending on the material and the amount of granular backfill in place. The gradation studies covered by the first part of this paper show that serious settlement can and does result from the loss of base material and adjacent soil by infiltration and "piping" when backfill materials are not properly graded. In addition, the consolidation study brings out the fact that volume changes up to 20 percent can and do occur in unconsolidated granular fills. The investigation further disclosed that certain types of vibratory equipment are readily adaptable to the consolidation of granular materials and that the cost of placing granular backfill with vibratory equipment will be less than the cost of placing it by other common methods. Specification requirements for backfill material were also developed from the investigation.

This bulletin presents in detail the work covered by the two major investigations – the gradation study and the consolidation study, the latter including work both in the laboratory and on actual construction projects.

GRADATION STUDY

The primary purpose of a granular backfill is to provide drainage for the structure. The granular backfill must then, first of all, function as a filter. To do this it must be so graded that: 1) adjacent soil material will not infiltrate into the granular backfill, 2) the fines in the granular backfill will not be displaced by water movement, and 3) the granular backfill will be permeable.

A comprehensive study of these requirements was made by the United States Corps of Engineers in the Waterways Experiment Station located at Vicksburg, Mississippi. The following formulas were developed as a result of this study:

1. To insure the prevention of detrimental movement of adjacent protected soil (subgrade soil or filter layer) –

$$\frac{\text{15\% Size of filter material}}{\text{85\% Size of adjacent protected soil}}, \text{ Not greater than 5}$$

2. To prevent movement –

$$\frac{\text{85\% Size of filter material}}{\text{Size of perforation or slot opening}}, \text{ Greater than 2}$$

3. To insure a satisfactory permeability—

$$\frac{\text{15\% Size of filter material}}{\text{15\% Size of adjacent protected soil}}, \text{ Greater than 5}$$

These formulas are to be applied, by laboratory test, to each different backfill material, each change in subgrade soil, and each type of outlet having openings of different size.

The investigation covered by this paper includes a study made for the purpose of establishing a gradation range for granular backfill, a range to satisfy the formulas above and include most of the natural gravel and sand deposits in Michigan. The study was made in the Michigan State Highway Laboratory at Ann Arbor under the supervision of G. O. Kerkhoff.

GRADATION TESTS

The studies to determine proper gradation for filter materials were, as mentioned above, carried on in the Michigan State Highway Laboratory located in Ann Arbor. These studies included mechanical analyses and filter tests on several different kinds of materials. The curves for the mechanical analyses and the application of the formulas for this phase of the study will be found in Appendix A of this bulletin.

The filter tests were made in a glass-faced tank 30 inches long, 13 inches high and 4 inches thick. Two metal cylindrical screens, 1¼ inches in diameter having 1/32-inch openings spaced 1/32 inch apart were placed 6 inches apart and 3¾ inches above the bottom of the tank.

Filter Test 1: The first test was made with a coarse gravel material having the gradations shown in Table 1.

TABLE 1—*Gradation of granular materials before and after water filter test 1*

Size of sieve	Gradation of gravel before test	Gradation of sand before test	Gradation of material after 5-hour test
	Percent passing	Percent passing	Percent passing
1″	70	—	74
½″	45	—	50
No. 4	20	—	26
No. 8	14	—	16
No. 10	10	—	15
No. 20	7	—	13
No. 40	5	100	12
No. 50	4	—	10
No. 60	—	93	—
No. 100	2	—	2
No. 140	—	18	—
No. 200	0	9	0
No. 270	—	5	—

After thorough mixing, this material was placed in the tank. Water was introduced from above and uniformly distributed by running the water down a slanting trough with openings in the bottom.

Water and sediment went through the screens into overflow cans at the rear of the tank. After the water had been running through the tank for about 20 hours, the sediment that had passed through the screens and into the overflow cans was dried and weighed. Its dry weight was 8 grams. Figure 1 shows the material in the tank after 20 hours of test.

Thirty pounds of sand was then spread over the top of the coarse aggregate. The sand had the gradation shown in Table 1. The screen outlets were closed, and water was introduced from above until the water level was from 1 to 2 inches above the surface of the sand. The valves at the outlets of the screens were gradually opened until they were completely open at the end of 3 hours. Sufficient water was introduced to keep the water level approximately 1½ inches above the sand. The amount of water added decreased somewhat after several hours. At the end of 5 hours, no movement of material inside the tank was visible. Fifty-two grams (after drying) of sediment

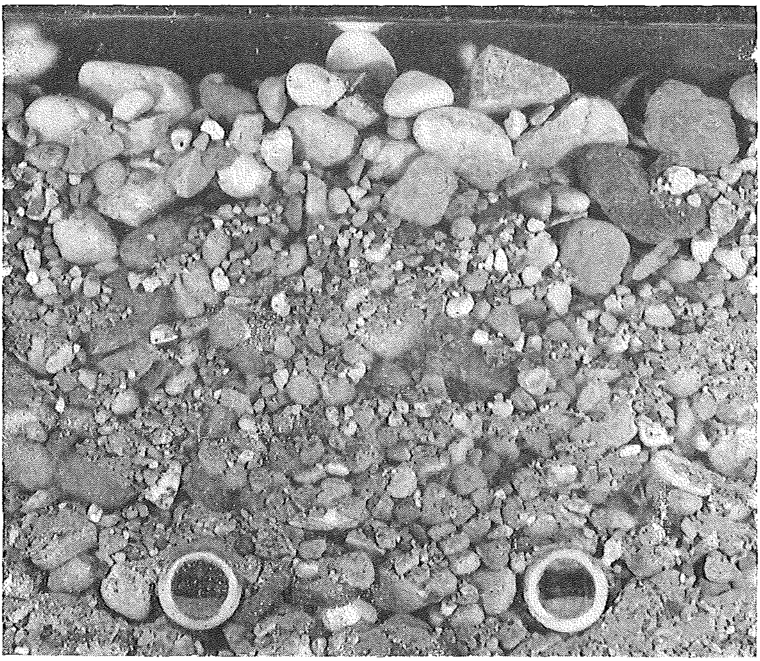


Fig. 1. Coarse aggregate filter test showing arrangement of particles after water had passed through sample for 20 hours.

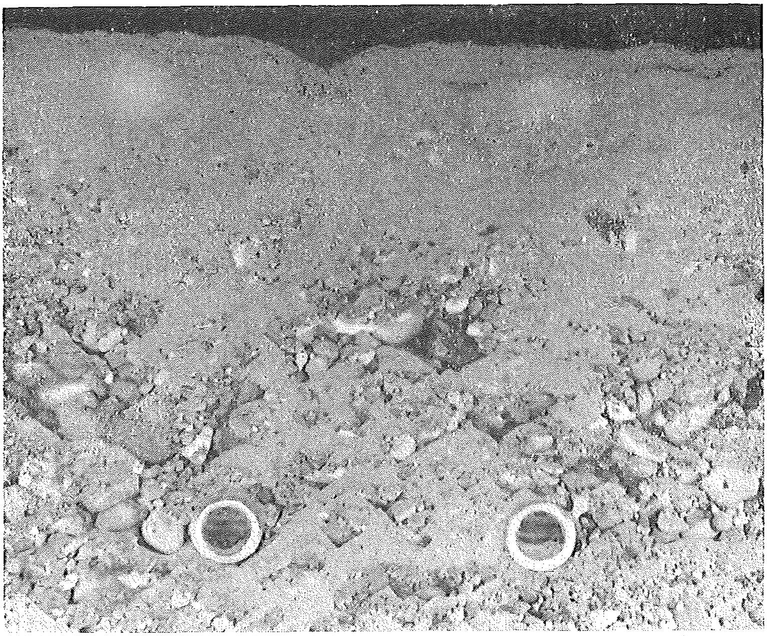


Fig. 2. Coarse aggregate filter test after sand had been placed on the surface and water passed through sample 5 hours.

was removed from the overflow cans. The gradation of the material is given in Table 1. Figure 2 shows the material in the tank at the end of the test.

Filter Test 2: The second filter test was made with finer material having the gradation given in Table 2. After mixing, the material was placed in the tank and covered with water to a depth of about 4 inches.

TABLE 2—*Gradation of granular materials before and after water filter test 2*

Size of sieve	Gradation of gravel before test	Gradation of silty loam before test	Gradation of material after 36-hour test
	Percent passing	Percent passing	Percent passing
No. 4	70	—	69.1
No. 8	57	—	53.4
No. 10	50	—	50.7
No. 20	40	—	40.1
No. 40	30	100	24.4
No. 50	25	—	19.0
No. 60	—	99	—
No. 100	20	—	4.6
No. 140	—	94	—
No. 200	5	79	1.1
No. 270	—	65	—
.005 mm	—	2	—

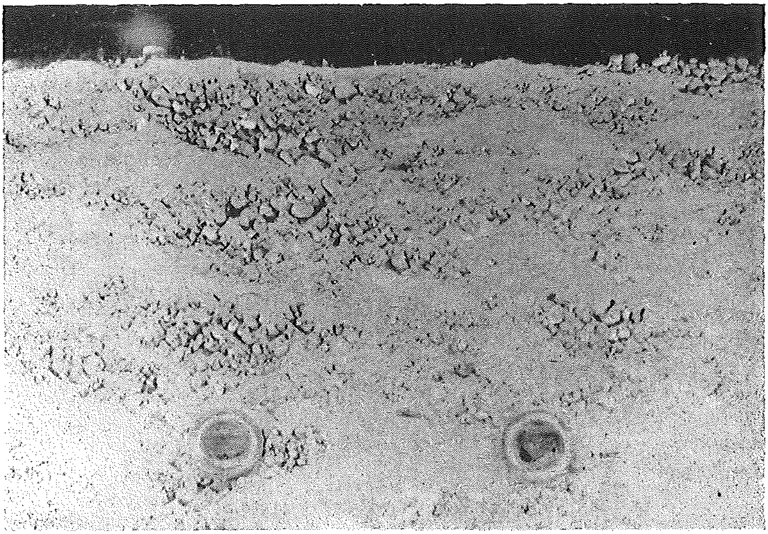


Fig. 3. Fine gravel and sand material at start of test.

This depth was maintained as nearly as possible throughout the test. Figure 3 shows the material in the tank at the beginning of the test.

The water moved through the material rather slowly and at a gradually diminishing rate with the cylindrical screens running about

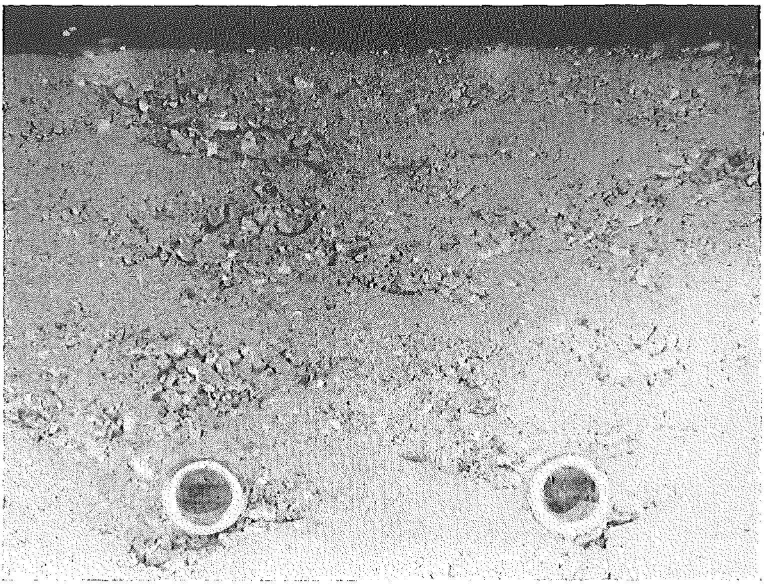


Fig. 4. Fine gravel and sand material after 18-hour test.

half full. The ponding operation was continued for a period of 18 hours; during this time the water flowed through the outlets at the rate of 2.08 gallons an hour. At the end of the 18-hour period the tank was drained and the sediment in the overflow cans dried and weighed. Only 1.8 grams had passed through the drains. Figure 4 shows the material at the end of 18 hours of ponding.

The ponding was continued for 2 hours more. Then 30 pounds of silty loam was placed in the water above the fine gravel and sand. The gradation of the silty loam is given in Table 2. The movement of water through the silty loam and the fine gravel and sand was now reduced to 1.52 gallons an hour. After 36 hours of ponding, the water was moving through at the rate of 0.88 gallon an hour. During the last few hours of the test, there was no net decrease in the flow, but some variation. Sediment cans contained 0.3 gram of material after drying. Figure 5 shows the material in the tank at the end of the 36-hour period.

Six inches of the fine gravel and sand material, taken just above the screens and weighing 46 pounds, had the gradation given in Table 2. A study of this gradation and the gradation at the start of the test will show that little, if any, of the silt loam material was carried into the filter material by the action of the water.

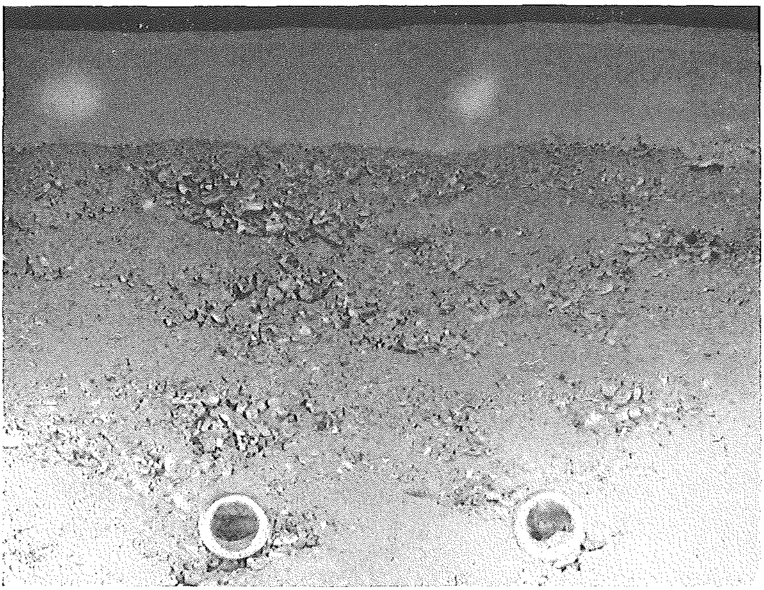


Fig. 5. Fine gravel and sand material after 36 hours of ponding.

TABLE 3—*Gradation of granular materials used in water filter test 3*

Size of sieve	Gradation of gravel before test	Gradation of fine sand before test
	Percent passing	Percent passing
1½″	100	—
1″	79	—
½″	33	—
No. 4	0.5	—
No. 40	—	100
No. 60	—	93
No. 140	—	18
No. 270	—	5

Filter Test 3: The third filter test was made with 6-A gravel having the gradation shown in Table 3.

The material was well mixed and placed in the tank and water introduced. The amount and rate of water passing through the material was limited only by the size of the outlet pipes. Figure 6 shows the 6-A material in the tank before the test was started. After the water had been run through the tank for a short time, 12½ pounds of the 6-A material was removed and 27½ pounds of fine sand was placed on top of the remaining 6-A material. The gradation of the sand material is given in Table. 3.

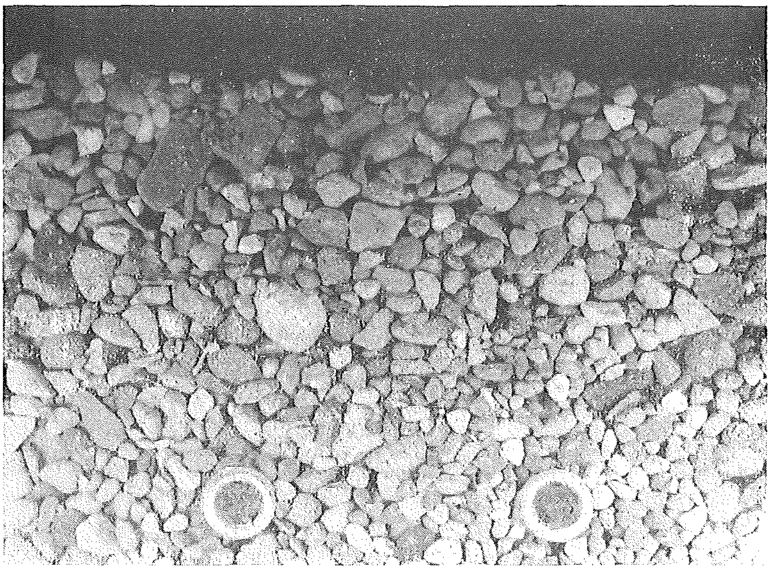


Fig. 6. 6-A gravel before test.

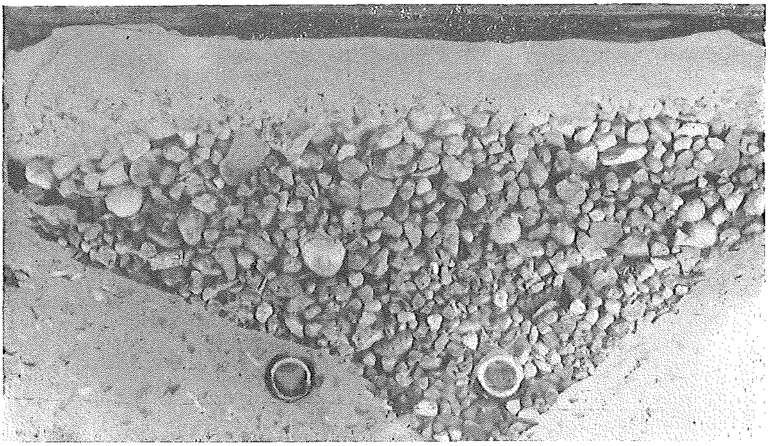


Fig. 7. 6-A aggregate test showing the movement of sand through coarse backfill materials.

The addition of the fine sand had no apparent effect on the rate of flow through the tank. Only one valve was opened during the first part of this test, and the effect can be noted in Fig. 7. Piping was very evident in the upper left corner of the tank. Water and sand moved through the 6-A aggregate very rapidly. After the second valve was opened, the piping effect was accelerated and distortion of the sand surface was very pronounced, as is shown in Fig. 8. The water and sand were passing through so rapidly in this test that the sediment carried through was not recovered. The water was passing through the outlets at the rate of about 4 gallons a minute.

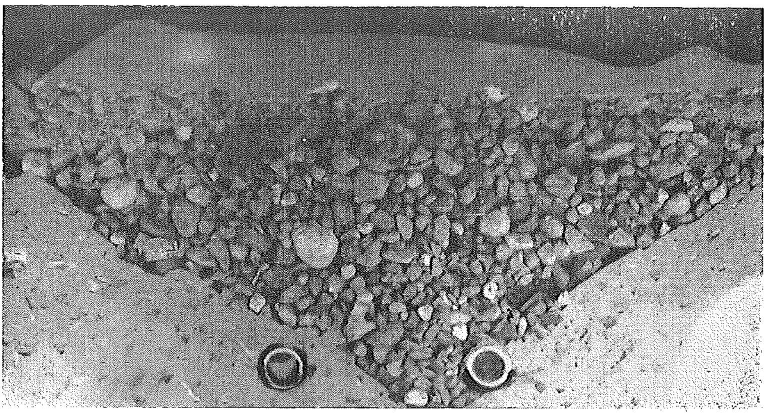


Fig. 8. 6-A aggregate test showing distortion of sand blanket surface.

Fig. 9. 6-A aggregate and sand mixture before test.

Filter Test 4: Test 4 was made with a mixture consisting of 70 percent 6-A aggregate and 30 percent 2NS sand. The gradation of the 6-A aggregate and the 2NS sand are given in Table 4.

The two materials were mixed together and placed in the tank. Some segregation occurred during the placing of this material in spite of the fact that the utmost care was exercised in this step. The addition of water caused some additional segregation or consolidation. The segregation or consolidation can be noted in Fig. 9.

After the water was introduced and the head established at several inches above the surface of the filter material, 26 pounds of fine

TABLE 4—*Gradation of granular materials before and after water filter test 4*

Size of sieve	Gradation of 6-A aggregate before test	Gradation of 2NS sand before test	Gradation of fine sand material before test	Gradation of material after test
	Percent passing	Percent passing	Percent passing	Percent passing
1½"	100	—	—	100.0
1"	75	—	—	83.8
½"	40	—	—	56.6
No. 4	0	99.7	—	30.9
No. 8	—	86.7	—	—
No. 10	—	—	—	25.4
No. 16	—	61.9	—	—
No. 20	—	—	—	17.0
No. 30	—	38.1	—	—
No. 40	—	—	100	8.8
No. 50	—	9.6	—	—
No. 60	—	—	93	3.2
No. 100	—	1.2	—	—
No. 140	—	—	18	0.3
No. 200	—	—	—	0.2
No. 270	—	—	5	—
Loss by washing	—	0.6	—	—

sand was placed in the water and spread over the surface of the filter material. The water level was maintained at approximately 1 inch above the sand. The gradation of the sand that was added and spread over the surface of the filter is given in Table 4.

A few minutes after the fine sand was added, the water was flowing through the outlets at the rate of 28 gallons an hour. Seven hours later the flow had decreased to 15.3 gallons an hour. Several depressions were noted in the surface of the fine sand, and some of the fine sand that was placed in the water above the filter material had penetrated down into the sand-gravel mixture several inches. The test was interrupted at this point, and the material permitted to drain. Several hours later the test was again started, and the rate of flow was 14 gallons an hour. Five hours later the flow had dropped to 8 gallons an hour. Figure 10 shows the condition of the material at the end of 48 hours.

The gradation of a sample of the material 6 inches thick taken from a layer immediately above the outlet screens is given in Table 4. The sediment collected in the overflow cans was dried and weighed and amounted to only 0.5 gram after 48 hours. The small amount of sediment collected indicates that the gradation of the material near the drains was such that it functioned very well as a filter.

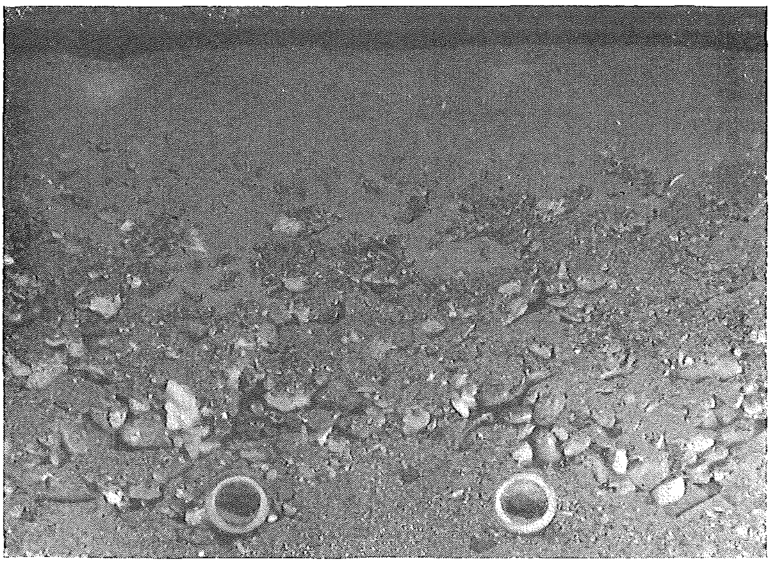


Fig. 10. Note the penetration of the fine sand into the filter material after 48 hours.

TABLE 5—*Gradation of slag before test 5*

Size of sieve	Gradation of slag before test	Gradation of sand before test
	Percent passing	Percent passing
2″	100.0	—
1½″	98.0	—
1″	73.0	—
½″	31.0	—
No. 4	1.8	—
No. 10	—	100.0
No. 20	—	99.7
No. 40	—	99.2
No. 60	—	79.9
No. 140	—	5.6
Loss by washing	0.8	3.0

Filter Test 5: The fifth and last filter test was made with 6-A slag having the gradation given in Table 5. Figure 11 shows the slag aggregate in the tank before beginning the test. The slag was carefully placed in the tank to prevent segregation, and water was then introduced until the surface of the slag was covered to a depth of about 3 inches. Sand having the gradation given in Table 5 was then spread over the slag.

At first only one valve was opened, and the water moved through the filter very rapidly, carrying much of the sand down into the slag and a considerable portion through the outlets. The first valve was

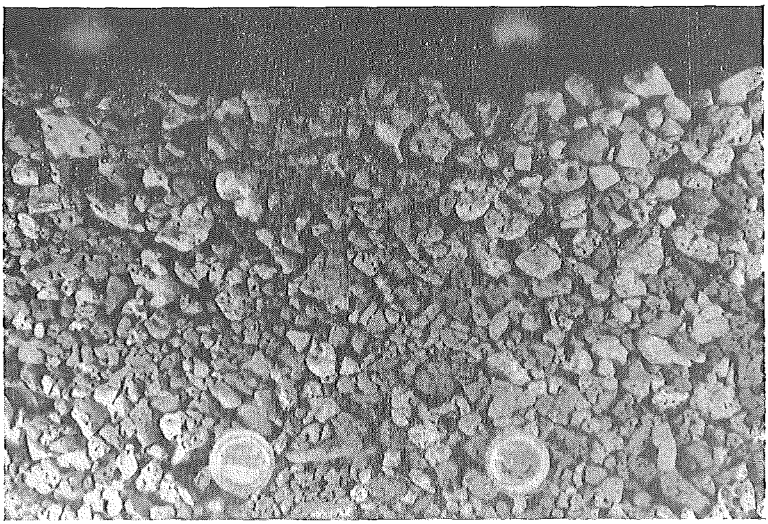


Fig. 11. 6-A slag in tank before test was started.

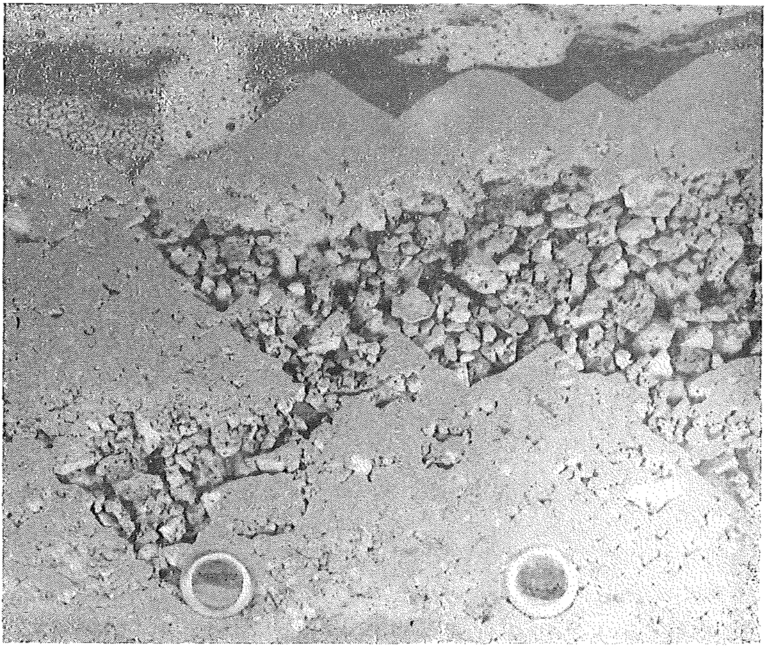


Fig. 12. 6-A Slag Filter showing infiltration of sand base.

then closed and the second one opened. The water then moved through the filter very slowly, indicating that the sand had settled around the drain so tightly as to be nearly impervious. When the first valve was again opened, the discharge rate was as great as before. Figure 12 shows the condition of the slag filter at the end of the test, as well as the distortion of the surface of the sand layer.

SUMMARY OF GRADATION STUDIES

From a careful study of the foregoing tests and of the properties and characteristics of the natural deposits of sand and gravel soils in Michigan, specifications for granular backfill were written. It is believed that these specifications, which are quoted in the following paragraphs, will satisfy all requirements for a granular backfill. Necessary materials are readily obtainable in most of the sand and gravel deposits of Michigan.

The specifications covering the requirements for nearly all granular backfill materials are at present available as supplementary specifications for Porous Backfill, Grade B, and Porous Backfill, Grade C. They are as follows:

> "Aggregate for use as a porous backfill, Grade B, shall consist of a mixture of pebbles, or broken stone, or slag and sand conforming to the following grading requirements:

Size of sieve	*Percent passing*
2″	100
1″	70-100
½″	45-100
No. 4	20- 70
No. 10	10- 50
No. 40	5- 30
Loss by Washing	0- 5

> "Aggregate for use as a porous backfill, Grade C, shall consist of a mixture of pebbles, or broken stone, or slag and sand conforming to the following grading requirements:

Size of sieve	*Percent passing*
1″	100
½″	75-100
No. 4	50- 70
No. 10	35- 50
No. 40	20- 30
Loss by Washing	0- 5

> "When either Grade B or Grade C porous-backfill material is to be used as backfill for underdrains, the upper limits of the mix will be governed by the size of the openings in the underdrain. Fifteen percent of the backfill material shall be at least twice the size of the openings."

Grade B material can be found in most of the sand and gravel deposits in Michigan and will meet all requirements for backfill material on approximately 90 percent of the projects.

Grade C material may be found in natural deposits or it may be necessary to process the material to obtain proper gradation. Grade C material is specified for use in areas where the adjacent soil is largely silt or very fine sand (0.005 to 0.105 mm.). The gradation of the Grade C material is such that there will be no infiltration of the adjacent fine-grained soils.

The tests described in the foregoing definitely established the importance of proper gradation in granular backfill material. Of equal importance is the manner in which the backfill material is placed. The remainder of this paper is devoted to the study of this phase of granular backfill treatment.

CONSOLIDATION STUDY

The consolidation studies were divided into four different steps, as follows:

1. Small-scale laboratory tests of different methods of consolidation with different kinds of materials. The methods of consolidation used were ponding, vibrating, and tamping. This phase of the study was made at the Michigan State Highway Department Testing Laboratory at Ann Arbor, Michigan.

2. Tests on a larger scale, using the same methods of consolidation on a wider variety of materials, were next made at the Michigan State Highway Department Research Laboratory at East Lansing, Michigan. Consolidation was obtained by vibrating, tamping, and ponding. The vibrating was done with a bullet-nosed concrete vibrator. The materials were dune sand, natural sand, bank-run gravel, pea gravel, coarse aggregate, and slag.

3. Consolidation by vibration on large masses of bank-run gravel. This step was a study of different types of vibratory equipment.

4. Consolidation on actual field projects with different types of equipment on the granular material available in that particular area.

The studies made at the Ann Arbor Laboratory were primarily on dune sand and pea gravel chosen for uniformity of gradation, and the methods of consolidation were by ponding, by vibration, and by tamping. The results of the various tests made with the dune sand will be found in detail in Appendix A of this report and are briefly covered in the following pages.

The tests on the loose material were designed primarily to determine the effect of bulking, the percentage of voids in the loose material, and the weight per cubic foot of the loose material at different percentages of moisture content. Approximately 95 tests were made on dune sand and, of this number, 30 tests were on the material as loose as it could be placed in the container. Consolidation was obtained by flooding in 21 tests, by vibration in 9 tests, and by tamping in 16 tests.

The density of dune sand loose and dry, with 1 percent or less moisture content, was found to be 93.5 pounds per cubic foot. The average density of 10 samples, having moisture contents varying from 1.13 percent to 17.6 percent, was found to be 75.63 pounds per cubic foot. The average moisture content was 8.21 percent. The high density for this series was 78 pounds per cubic foot, with a moisture content of 1.13 percent, and the low density was 73.34 pounds per cubic foot, with a moisture content of 15.6 percent. The average percentage of voids for the 10 samples having varying moisture content was 54.4.

With this data, it is reasonable to assume that any loose deposit of dune sand or sand of similar gradation and specific gravity, will have a density of approximately 76 pounds per cubic foot and a total percentage of voids of about 54.

CONSOLIDATION OF DUNE SAND

The next step was to determine the most efficient method of consolidation for dune sand in the loose state, with varying moisture content, so as to obtain the greatest weight per cubic foot, thereby decreasing the total percentage of voids to a minimum.

Inundation: Inundation with water was first used. In this test, the top of the sample was covered with water as the test progressed. Five tests of this type were made on sand having an original moisture content of less than 1 percent. The average density before consolidation by inundation was 94 pounds per cubic foot, and the average total voids was 43.4 percent. The average density after inundation was 97 pounds per cubic foot and the average total voids 41.6 percent. The average moisture content of the five samples after ponding and before any appreciable time for draining had elapsed was 21.5 percent.

The same test was made on six samples of dune sand having an original moisture content of from 1 to 2 percent. At the start of the test, the average density was 76 pounds per cubic foot, and the total percentage of voids of the material was 54.2. The average moisture content of the six samples was 1.37 percent.

After ponding was completed, the average density was 88 pounds per cubic foot, and the total percentage of voids was 47. The average moisture content before draining was 25.2 percent.

The original moisture content in a dune-sand material apparently has an appreciable effect on the final density when consolidated by ponding. It will be noted that in the tests described above the average density of the material having an original moisture content in excess of 1 percent amounted to 88 pounds per cubic foot, or 6 pounds per cubic foot less than the average density of loose, dry material.

Similar tests were then made on the dune-sand material, but the water was introduced from the bottom of the sample rather than ponded on the surface. Four tests were made, starting with material having a moisture content of less than 1 percent. The average density of the loose material at the start of the test was 93.9 pounds per cubic foot. The average total percentage of voids was 43.4, and the average moisture content was 0.156 percent. The average density of the dune sand used in these four tests, after indirect inundation, was 95.8 pounds per cubic foot and the average percentage of voids was 42.2. The average moisture content before draining was 27.5 percent.

Indirect inundation of the material having an original moisture content in excess of 1 percent resulted in an average density of 89.7 pounds per cubic foot and an average total voids of 46 percent. The average density at the start of this test was 74.7 pounds per cubic foot and the average total percentage of voids was 54.9. The low density of the loose material was 70 pounds per cubic foot, with a moisture content of 10.6 percent, and the high density of the loose material was 89.1 pounds per cubic foot, with a moisture content of 1.29 percent.

The moisture content at the beginning of this test again had considerable influence, and the final density of the material having an original moisture content in excess of 1 percent was less than the density of the loose, dry material.

Vibration: In the next series of dune-sand tests, consolidation was obtained by vibration. The apparatus used for this test consisted of a vibrating platform mounted on springs. The springs deadened much of the impact imparted by the vibrating motor. The term "mild vibration" used in the following data indicates the vibration transmitted to the sample with the springs in operation. The term "extreme vibration" indicates the vibration imparted to the sample with the springs blocked out by wooden inserts between the platform and the vibrating frame.

Two tests were made with dry dune sand and five tests with dune sand having a varying moisture content. The density of the first dry sample after mild vibration for a period of 4 minutes was 100.3 pounds per cubic foot with total voids of 39.6 percent. The density of the second test sample after extreme vibration for a period of 4 minutes was 106.44 pounds per cubic foot with total voids of 35.9 percent. In the remaining five vibration tests, the average density before vibration was 71.3 pounds per cubic foot, with total voids of 57 percent. Three of these tests were run with mild vibration and two with extreme vibration. The average density of the first three samples after 4 minutes of mild vibration was 96.3 pounds per cubic foot, and the average total voids was 42 percent. The average moisture content of the three samples tested was 5.36 percent. The average density of the two samples after extreme vibration was 104.35 pounds per cubic foot. The average total voids was 37.1 percent, and the average moisture content was 4.1 percent.

Cone: The cone density test[1] is a procedure for consolidating samples of granular material. The results of this method compare very favorably with the densities obtained by extreme vibration. The test provides a simple and rapid means of checking density requirements either in the laboratory or the field.

From the data sheets in Appendix A, it will be noted that 14 tests were made with dry dune sand and 13 tests with dune sand having varying moisture contents. The average density of the 14 dry samples was 105.25 pounds per cubic foot. The average total voids was 36.6 percent.

The results of the 13 tests made on dune sand having a moisture content varying from 3.5 percent to 13.7 percent, show the average density to be 104.1 pounds per cubic foot, the total voids, 37.3 percent, and the average moisture content, 8.4 percent.

A.A.S.H.O. Test T-99: The American Association of State Highway Officials (A.A.S.H.O.) Test for the Compaction and Density of Soils, Designation: T-99-38, was used for 16 samples of dune sand with variable moisture content. The average density of the material compacted by this method was 104.3 pounds per cubic foot, the total voids was 37.16 percent, and the average moisture content was 7.41 percent.

[1]The Cone Density Test is described in Appendix D.

The low density of the series was 100.5 pounds per cubic foot, with a moisture content of 1.24 percent, and the high density was 105.9 pounds per cubic foot with a moisture content of 13.02 percent.

The last three methods of test, i.e., by vibration, by cone and by tamping, gave the most uniform results and much higher densities. Of these methods, the one believed to be most applicable to general construction practice is the method of consolidation by vibration. The standard A.A.S.H.O. Test T-99 gave results that compared very favorably with the densities obtained by the vibration and cone methods. However, this test is primarily for use on soils of very fine gradation and therefore is not generally adaptable to deposits of granular materials.

CONSOLIDATION OF PEA GRAVEL

The next series of tests was made on pea gravel having a gradation as follows:

Size of sieve	*Percent passing*
½″	100
⅜″	96
No. 4	20
No. 10	2

The average density of the loose pea gravel was 94.17 pounds per cubic foot. The average total voids was 41.9 percent, and the average moisture content was 1.6 percent. These averages were based on 55 tests.

Inundation: The average density of the pea gravel after ponding and drying was 93.3 pounds per cubic foot. The average total percentage of voids was 42.4. These averages were based on only six tests, but it will be noted that the average density after saturation was less than the loose dry density, and the percentage of total voids was higher after ponding than in the loose, dry material. From these tests it is apparent that ponding or saturation of pea-gravel backfill is ineffective in obtaining consolidation.

Cone: The pea gravel was then consolidated by the cone method. The average results were a density of 107.1 pounds per cubic foot and a total percentage of voids of 33.96, or an increase in density over the loose dry weight of 13.7 percent and a decrease in voids of 18.9 percent.

Vibration: Consolidation by mild vibration resulted in an average density of 104.28 pounds per cubic foot and a percentage of total voids averaging 35.7. These tests resulted in an increase in density of 10.7 percent and a decrease in total voids of 14.61 percent.

The value of these tests on pea-gravel material is primarily to show the consolidation that may take place under traffic vibrations and loads if it is not properly compacted when placed. However, it should be kept in mind that from the preceding tests on filtration it was determined that pea gravel, because if its gradation, is not suitable for use as a granular backfill.

The tests briefly described in the foregoing have conclusively proved that two factors must be considered in a discussion of granular backfill. First, the question of gradation has been found to be an important one. It is true that there might be some cases where gradation of backfill would not be of paramount importance. This would probably be true in a location where the adjacent soil was clay or possibly clay loam and there would be little or no danger of infiltration of the adjacent soil. Under such conditions the gradation of the backfill material could be much coarser. Even in such an area, however, a coarse backfill material would need to be used with caution, for if the grading was opened up too much, the surfacing material would tend to infiltrate into the coarse backfill and cause irregularities in the riding surface.

The second factor brought out by the above-mentioned tests was that, regardless of the gradation of the backfill material, unless it is compacted in place, consolidation will occur and in time result in settlement of the slab.

VOLUME CHANGE AND CONSOLIDATION METHODS

With these two factors in mind, the next step in this study was to determine how much settlement might be expected in a backfill of selected granular material because of consolidation of that material, and the most efficient method of consolidating different kinds of granular backfill materials.

The preliminary tests for determining how much settlement might be expected were made with dune-sand material. The first step was to determine the minimum voids possible in dune sand compacted to maximum density. This was found to be approximately 30 percent. Working on this basis and checking the values in the laboratory by vibration and tamping, it was found that a 10-foot fill of dune sand placed loose would have a possible settlement of 29 inches, or approximately 25 percent. When ponded into place, a 10-foot fill of dune sand would have a possible settlement of 14.4 inches or 12 percent, and when compacted by tamping or by vibration, its possible settlement would be from 0 to 7.2 inches, depending on the effort exerted for the consolidation by these methods.

From this point, the study was continued on a larger scale, and for this phase the operations were moved to the laboratory at East Lansing. A model was constructed, with one side simulating the back of an abutment and the opposite side representing the slope of an excavation. The volume of the model was 15.6 cubic feet, and it was so constructed that the front face could be removed to facilitate unloading and also to provide opportunity for visual inspection of the material after compaction. The model was so arranged that a set of platform scales could be rolled under it and the entire model weighed and the density of the material being tested checked in this manner.

The purpose of this phase of the study was to substantiate the results of the preceding tests and to determine:

1. The compactive effort required to prevent detrimental settlement in backfills of dune sand, natural sand, bank-run gravel, pea gravel, coarse aggregate, and slag.

2. The effect of moisture content on compaction of granular materials by vibration.

3. The relative density that should be specified for granular materials when used as structure backfill.

4. The most efficient means of obtaining such density.

With these problems in mind, a standard procedure was set up for making the tests. The plan of procedure was as follows:

1. The material to be tested was placed in the model loose and the weight per cubic foot was determined. Figure 13 shows the model filled before consolidation.

2. The material was then consolidated by a bullet-nosed vibrator, shown in Fig. 14. In this operation, the vibrator was permitted to penetrate the material and was also operated on the surface. At some time during each of the tests, the vibrator was permitted to come in

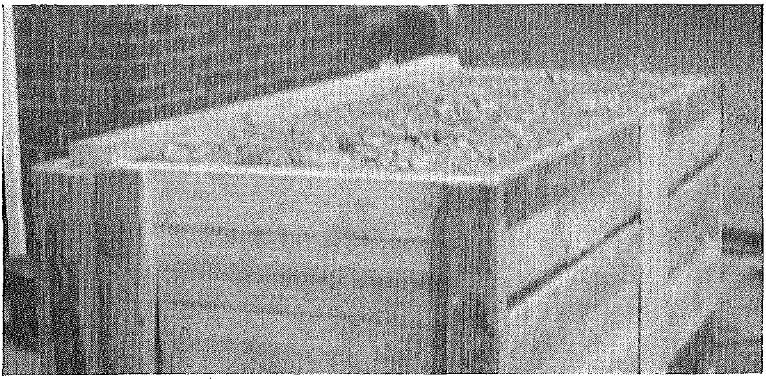


Fig. 13. Model filled with coarse aggregate.

contact with the sides and bottom of the model. When this happened, the entire model was set in vibration, and undoubtedly this had a definite effect on the test. This point is mentioned here because of difficulties encountered when using this type of vibrator in tests on a larger scale. After vibration, the volume change was measured as shown in Fig. 15 and the weight per cubic foot determined. Figure 16 shows the stability of the material after compaction.

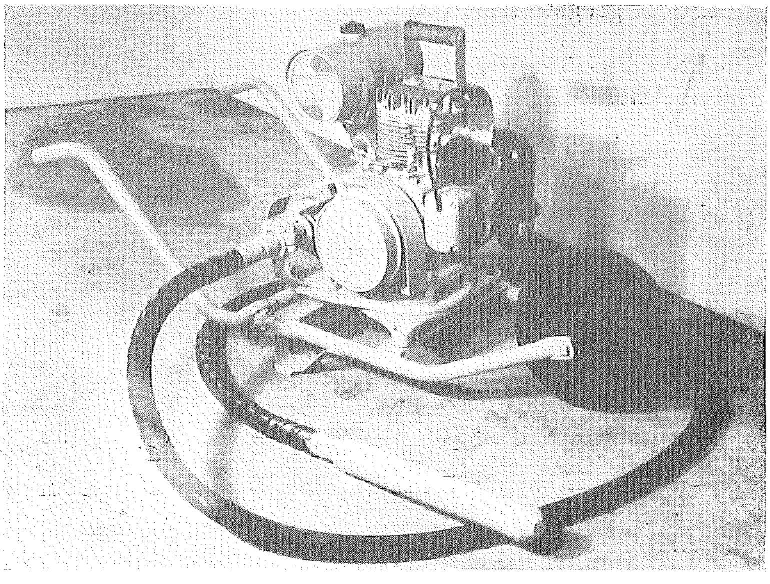


Fig. 14. Bullet-nosed vibrator.

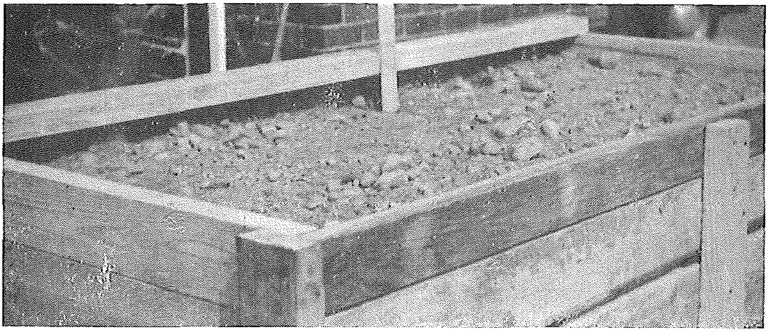


Fig. 15. Checking settlement after consolidation.

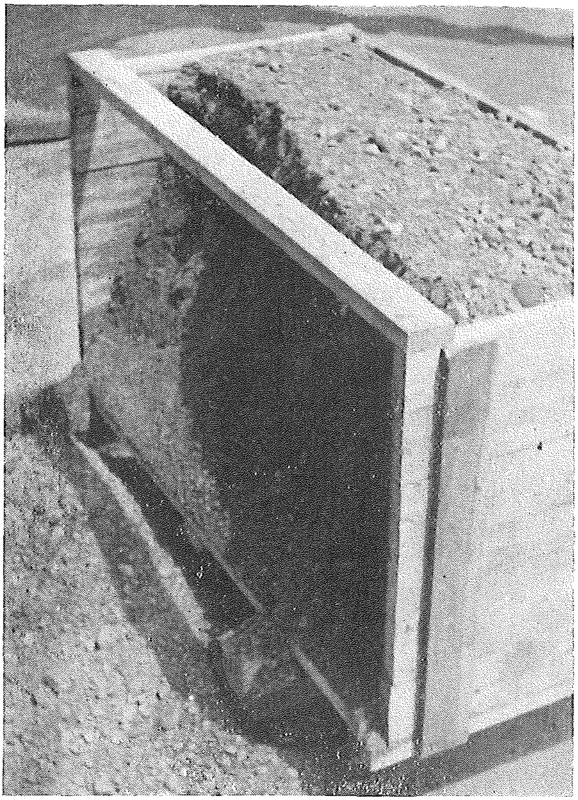


Fig. 16. Front face removed showing stability after compaction.

3. The material was then removed and again placed in the model while the vibrator was operated continuously.

4. The next step was to place the material in the model in 4- to 6-inch layers and tamp each layer with an 8-inch by 8-inch tamper.

Moisture contents and relative densities were recorded on all tests, and after compaction by vibration or tamping, the material was flooded for 20 minutes. After a reasonable length of time the moisture content was again checked to determine the relative drainability of the material.

Detailed records of these tests will be found in Appendix B of this report. The conclusions arrived at as a result of these tests are briefly summarized as follows:

1. No material exists that can be placed with no compactive effort without resultant detrimental settlement.

2. Sand or well-graded bank-run gravel will give the greatest density with the least compactive effort. Coarser particles will require greater compactive effort.

3. Moisture content has no appreciable effect on densities of materials compacted by vibration, but it has a great effect on densities of those compacted by tamping or inundation.

4. Tamping may be used on any granular material, and maximum densities can be obtained if the material is tamped in layers of 4 to 6 inches or less.

5. Vibration will give results equal to tamping in consolidating well-graded mixtures or those in which the particles are round. It would be advisable to use vibration instead of tamping on these soils since the same effect can be obtained with less effort.

6. When granular material is to be tamped in place, the moisture content should be at or near optimum as determined by A.A.S.H.O. Test T-99.

The foregoing tests all tended to confirm these conclusions, but very little information was available to make any recommendations as to the most efficient method of consolidating the backfill material in the field. It was definitely determined that consolidation is necessary to prevent settlement. However, the method of hand tamping or

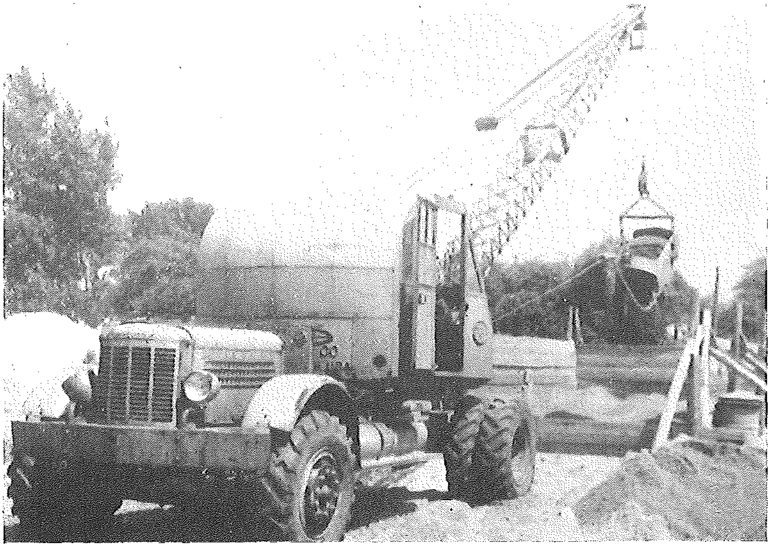


Fig. 17. Charging bin with road equipment.

even mechanical tamping was slow and tedious; the use of the bullet-nosed concrete vibrator was also slow, too slow, it was believed, to be of practical value on field projects.

FIELD CONSOLIDATION STUDIES AT SCOTTVILLE

The next series of tests was carried on at Scottville, Michigan. For these tests, a large bin was built, 16 feet long, 10 feet wide, and 8 feet deep. The bin was constructed of 3-inch lumber secured in place by posts set solidly in the ground. County maintenance equipment was used to charge the bin and to handle the larger types of vibrating equipment. Bank-run gravel was the material used in this study. Figure 17 shows the method of charging the bin with gravel.

The first vibrator units used, bullet-nosed concrete vibrators, were found to be ineffective in a large mass of soil material. This type of equipment consolidated a very small area in the vicinity of the vibrating head and had little or no effect 8 or 10 inches away. Figures 14 and 18 show two different models of the bullet-nosed vibrator.

After determining that the bullet-nosed vibrator is relatively ineffective in consolidating granular materials, it was still believed that some type of penetrating vibrator was desirable. Consequently, the

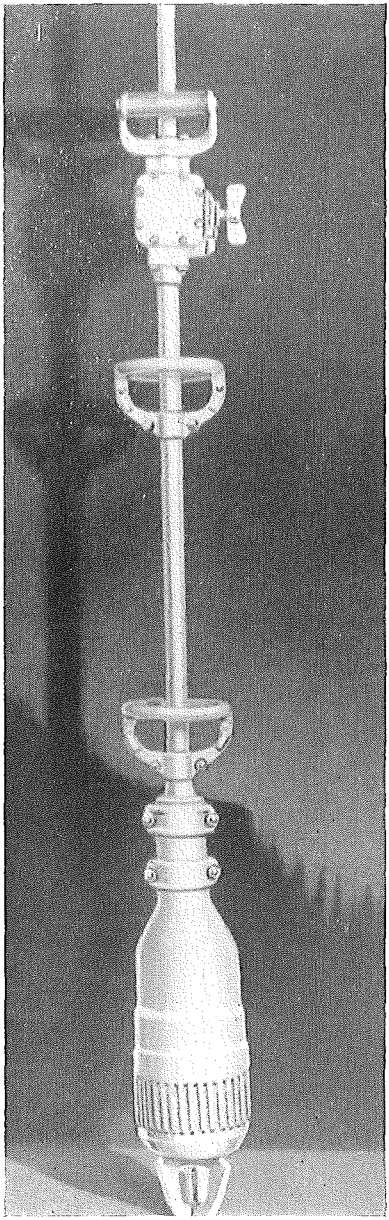


Fig. 18. Large bullet-nosed vibrator.

vibrators shown in Figs. 19 and 20 were constructed. This type of vibrator, if effective, would permit the placement of material in thick lifts. However, the operation of these units in the gravel mass produced much the same effect as that of the bullet-nosed vibrators. Good consolidation was obtained in the area immediately surrounding each tine of the unit, but it did not extend to the adjacent tine.

The idea of developing a penetrating type of vibrator was abandoned, and efforts were turned to the development of a vibrator that could be operated on the surface, thereby imparting a force to the layer of fill similar to that of a heavy, well-worn, caterpillar tread tractor.

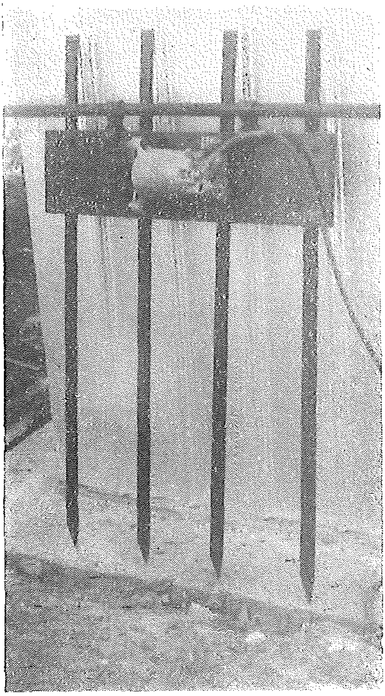


Fig. 19. Penetrating vibrator.

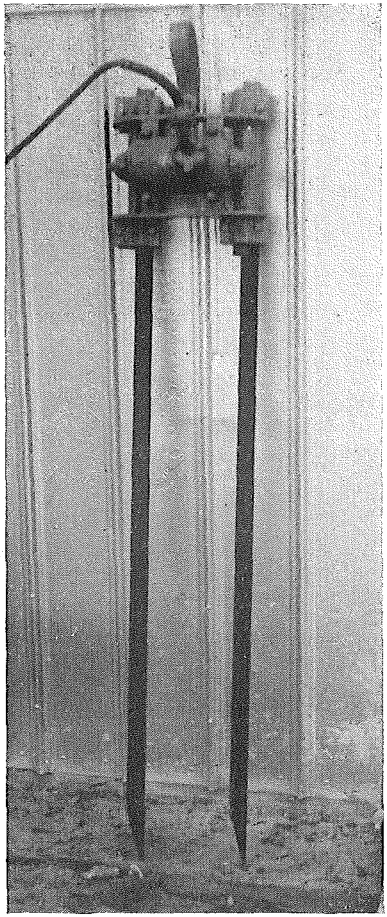


Fig. 20. Penetrating vibrator.

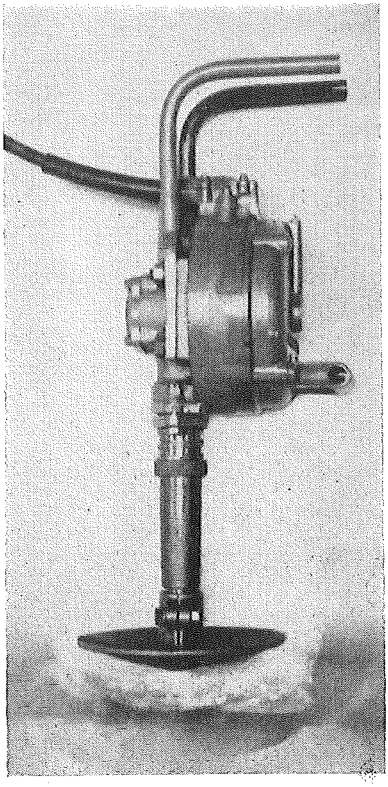


Fig. 21. Tamper.

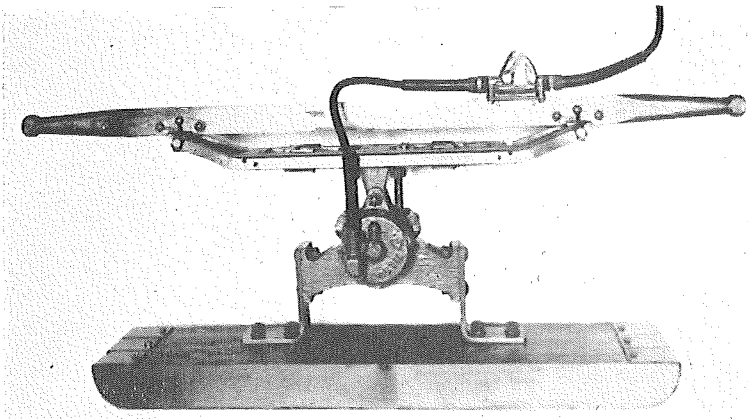


Fig. 22. Platform vibrator.

Figure 21 shows the first equipment of this type that was studied; a backfill tamper operated by a 110-volt, 3-phase, 60-cycle AC motor with a striking member forged integrally with the rotor shaft. Good densities were obtained with this tamper, but the method was slow owing to the small size of the tamping plate. It was necessary, of course, to handle the apparatus entirely by manual labor, and the layers of material tamped could not exceed 6 inches.

The platform vibrator shown in Fig. 22 was found to be very effective and, while it also was manually operated, satisfactory densities over larger areas were obtained more rapidly than with the backfill tamper first used. The results obtained with this unit encouraged the construction of a larger platform vibrator, Fig. 23, which would consolidate relatively large masses of granular material at a rate that would cause little, if any, delay to the contractors' operations. This unit must be handled by power equipment and is not adaptable to confined areas around structures.

The results with the larger unit were very satisfactory, and as a modification it was decided to make similar tests using a standard paving tube vibrator. This unit consists of a vibrator motor, mounted crosswise at the center of two 8-foot lengths of 2⅜-inch tubing. Figure 24 shows this equipment at the top of the fill in the test bin. The standard tube was found to float on top of the material, and much of the vibrator effect was lost. A piece of 2½-inch by 2½-inch by ⅛-inch angle iron was then welded to the front tube as shown in Fig. 25.

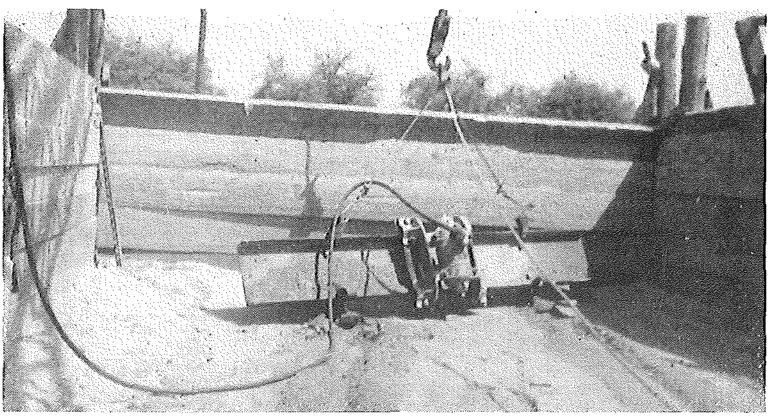


Fig. 23. Modified platform vibrator.

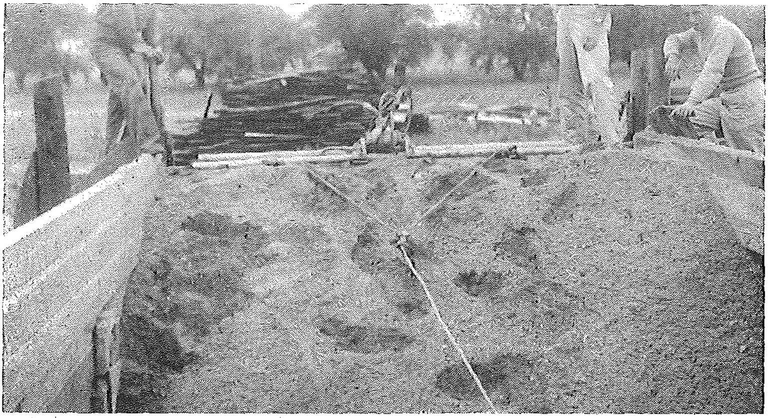


Fig. 24. Paving tube vibrator.

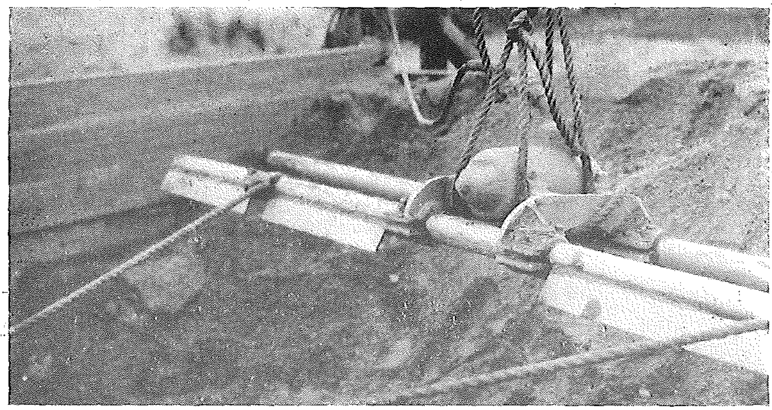


Fig. 25. Improved paving tube vibrator.

When operated at approximately 4,000 vibrations per minute and pulled down a sloping face of the material at the rate of about 13 feet a minute, this machine was very effective on relatively large quantities of material. The addition of the angle iron caused the unit to dig into the material, as shown in Fig. 26, thereby tending to transmit all compactive efforts to the granular material.

The results of these studies clearly demonstrated the effectiveness of the paving tube type of vibrator. The small platform vibrator seemed to offer greater possibilities for consolidating granular materials in confined areas, and plans were formulated to continue the study

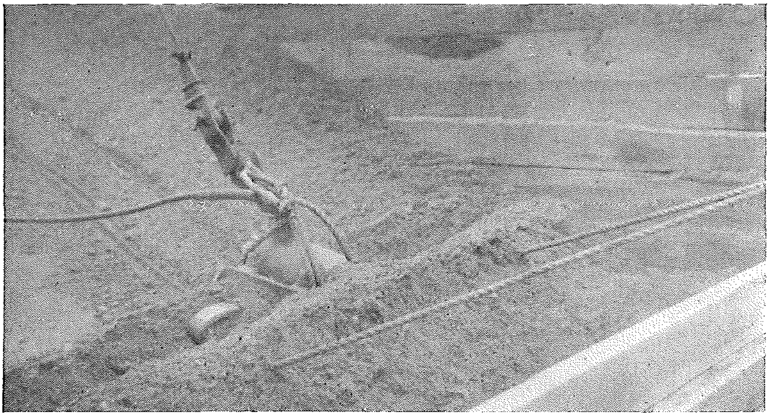


Fig. 26. Paving tube vibrator in operation.

and develop the equipment along the lines indicated. Detailed data covering these studies will be found in Appendix C.

No further study of the paving tube vibrator was considered necessary, as it had proved very satisfactory for special cases.

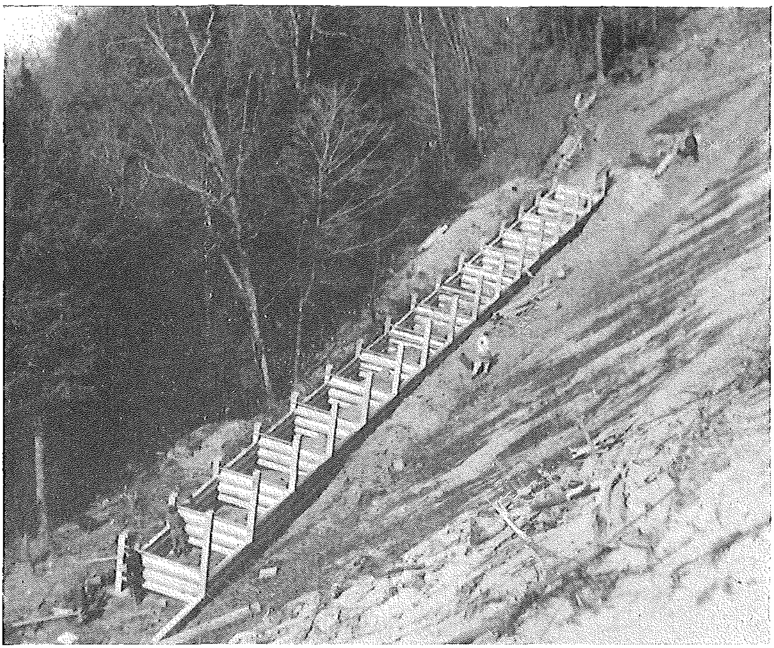


Fig. 27. Cut River metal crib.

CONSOLIDATION OF CRIB FILL (CUT RIVER BRIDGE PROJECT)

The platform vibrator was modified and next tested on a project near the Cut River Bridge on US-2, west of St. Ignace. The work consisted of the construction of a metal-crib retaining wall. Each crib or cell was 10 feet by 10 feet by 20 feet deep. The cribs were built up in sections, and the backfill inside the cribs and in back of the wall was placed simultaneously with the crib assembly. (See Fig. 27.)

The 10-foot by 10-foot cribs provided an excellent opportunity for comparing the compaction obtained by the vibrator with that obtained by the tamping method which the contractor used, and also with the densities of the loose material as it was shoveled into the crib.

The platform vibrator as modified for this work is shown in Fig. 28. Four different types of base plates were used in this study. Two of

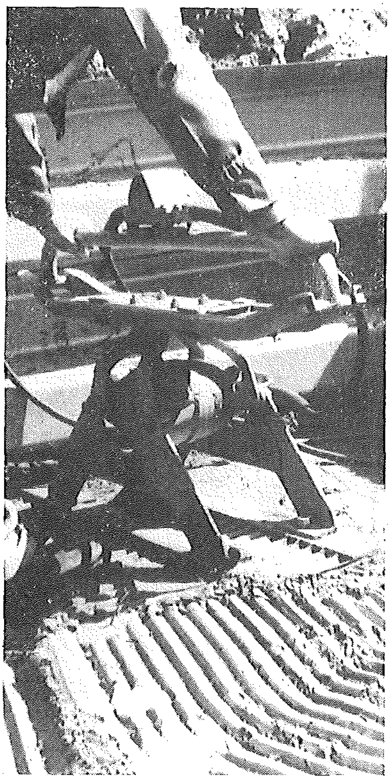


Fig. 28. Modified platform vibrator.

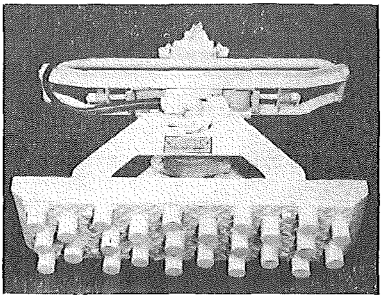


Fig. 29. Platform vibrator with sheepsfoot base plate.

the base plates are shown in Figs. 29 and 30. The one shown in Fig. 30 was found to be the most effective.

At this time it was noted that the unit as modified had a tendency to travel under its own power at the rate of about 10 feet per minute.

The material used to fill the crib was a dune sand and was placed in the cribs in approximately 9-inch lifts. To cover this 10-foot by 10-foot area with the tamper used by the contractor required 15 to 20 minutes to obtain satisfactory density. With the vibrator, equal

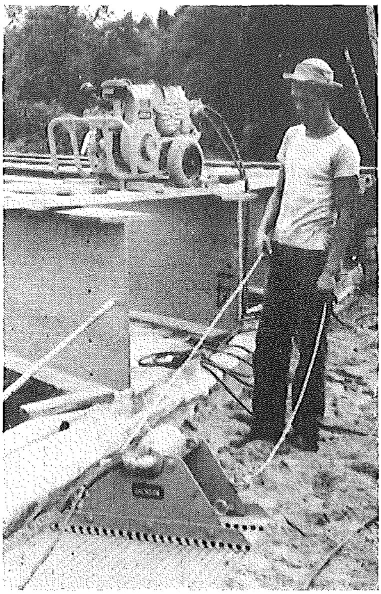


Fig. 30. Improved platform vibrator.

or greater densities could be obtained over the same area in from 3½ to 4 minutes. Detailed results of this study will be found in Appendix C.

STUDIES WITH THE PLATFORM VIBRATOR

The platform vibrator was also used on two different bridge projects, one in Grand Traverse County between Kingsley and Traverse City over the Boardman River and the other on a project south of Sparta on M-37. For these projects, the vibrator had been remodeled slightly for convenience in handling. Figure 31 shows two of these vibrating units.

The area of a sand surface affected by the vibration of the platform vibrator had a diameter of approximately 50 feet. This effect was demonstrated by small cones of sand built up approximately 25 feet from the unit. When the vibrator was started, the cones of sand immediately started to flatten out. The action was also noted on a reed tachometer placed at different locations in the area surrounding the vibrating unit.

The most effective frequency for compacting the granular materials used in this study was found to be in a range between 2,700 and 4,000 vibrations per minute. There is the possibility of encountering a ma-

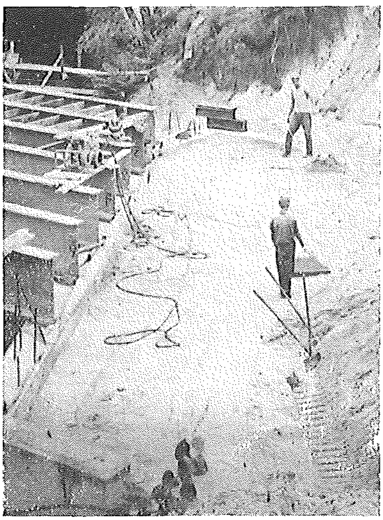


Fig. 31. Improved platform vibrators.

terial for which the efficient frequency will be outside the range mentioned above. A study is being started at this time for the purpose of developing a method to determine the most efficient frequency for different materials.

Layers of granular material 3 feet in thickness have been consolidated satisfactorily with this type of equipment, but consolidation of layers of this thickness does require a considerable length of time. In this study, consolidation was found to be most efficient in layers approximately 12 inches in thickness.

Usually one pass of the vibrator was sufficient to obtain densities of 95 percent or higher. Time-density curves indicate that satisfactory densities, 95 percent, are developed in approximately 13 seconds. Additional density is obtained rather slowly, but it is believed that densities equal to 95 percent of the density as determined by the "cone" method or by the vibrating table are satisfactory and that no detrimental settlement will occur in granular backfills or embankments compacted to this degree.

Density determinations in the field were made by several different methods including calibrated sand, heavy oil, mold of known volume and the rubber-membrane density apparatus. The rubber-membrane apparatus for determining volume was selected as the most satisfactory for average field conditions.

Additional studies are to be made on the type of equipment developed during the course of this study and on other types of compacting equipment for the purpose of improving efficiency, if possible. However, it should be clearly understood that to obtain satisfactory densities and thereby eliminate the present-day settlements in our backfills and embankments there must necessarily be some sacrifice in the speed of placing granular materials. While additional studies may lead to the development of more efficient vibrating equipment, definite conclusions have been arrived at as a result of the foregoing studies. These conclusions are briefly outlined in the following paragraphs.

CONCLUSIONS

The gradation of backfill material was found to be an important factor in any location where there is a possibility of infiltration of adjacent soil, sub-base material, or infiltration of the backfill material into and through the drainage structure. Where there is no possibility of such infiltration, obviously the question of gradation is of less importance. Backfill around the sub-structures of bridges or culverts in areas where the adjacent soil is heavy clay, which would have no water moving through it, would not require gradation control. However, if a granular sub-base were to be placed over the top of the backfill material, some precaution should be taken to prevent the infiltration of the sub-base fines down into the backfill material.

The question of the necessity of consolidation of granular materials was definitely answered by this study, as it was found that volume changes as great as 19 percent could and do occur in granular fills placed without consolidation. This volume change varies with the type of material and the method of placement, but it was found that all granular materials, unless properly consolidated when placed, will consolidate under traffic and weathering, thereby resulting in settlement of the surface.

The conclusions as to the most efficient means of consolidating granular materials may change as data is obtained from additional studies. However, as a result of this study these conclusions are made:

1. Satisfactory densities can be obtained by hand tamping and mechanical tamping if the layers are not over 6 inches in thickness. The method is slow and expensive.

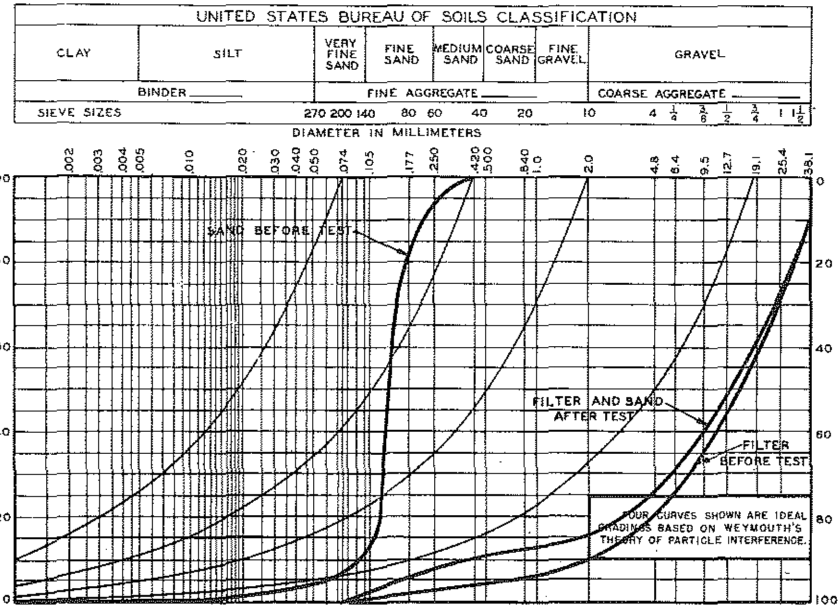
2. Penetrating vibrators, such as bullet-nosed concrete vibrators and vibrating spears or tines, are not efficient because the vibratory effect is confined to a very small area adjacent to the vibrating member.

3. The surface-type vibrators such as the modified paving tube vibrators and the self-propelled platform vibrators, proved to be the most efficient. The platform-type vibrator was found to give satisfactory densities on layers of material up to 12 inches in thickness. The paving tube type could be adapted to large unconfined areas, such as bridge approaches, embankments and granular bases. The platform type is especially suitable for those areas inaccessible to large equipment. The platform vibrator when used in a "gang" of four or more units is also adaptable to larger areas.

APPENDIX A

GRADATION STUDY

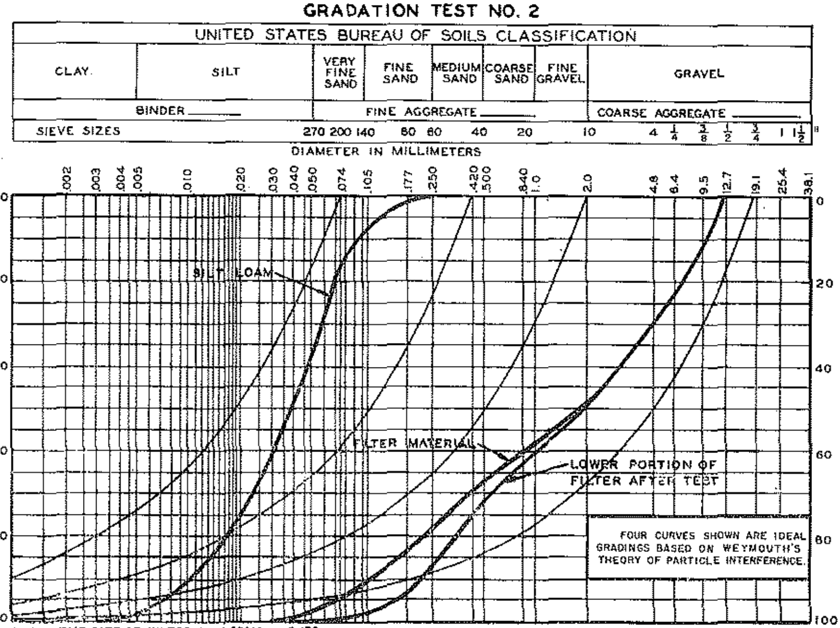
GRADATION TEST NO. 1



$$\frac{\text{15 PER CENT SIZE OF FILTER MATERIAL}}{\text{85 PER CENT SIZE OF BASE MATERIAL}} = \frac{3.60}{0.185} = 19.46 \text{ UNSTABLE}$$

Fig. 32. Gradation Test No. 1.

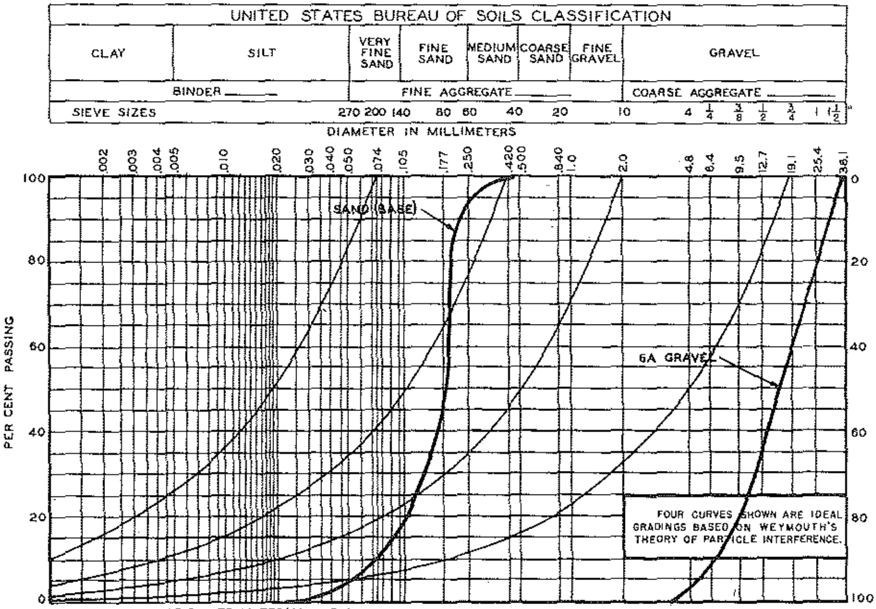
GRADATION TEST NO. 2



$$\frac{\text{15 PER CENT SIZE OF FILTER MATERIAL}}{\text{85 PER CENT SIZE OF BASE (SILT LOAM)}} = \frac{0.175}{0.08} = 2.2 \text{ STABLE}$$

Fig. 33. Gradation Test No. 2.

GRADATION TEST NO. 3

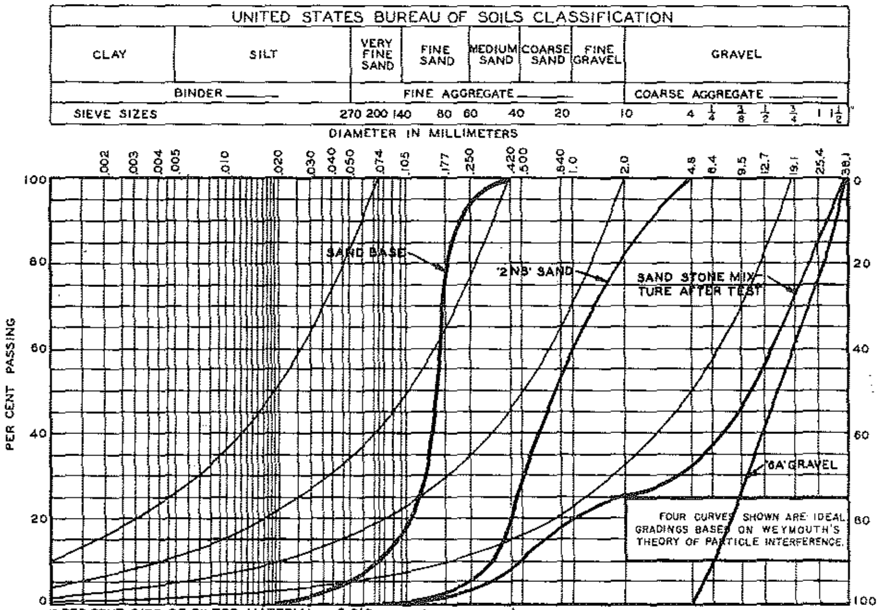


$\frac{\text{15 PER CENT SIZE OF FILTER MATERIAL}}{\text{15 PER CENT SIZE OF BASE}} = \frac{8.4}{0.90} = 9.3$ (SATISFACTORY PERMEABILITY)

$\frac{\text{15 PER CENT SIZE OF FILTER MATERIAL}}{\text{85 PER CENT SIZE OF BASE}} = \frac{8.4}{0.20} = 42.0$ (UNSTABLE)

Fig. 34. Gradation Test No. 3.

GRADATION TEST NO. 4

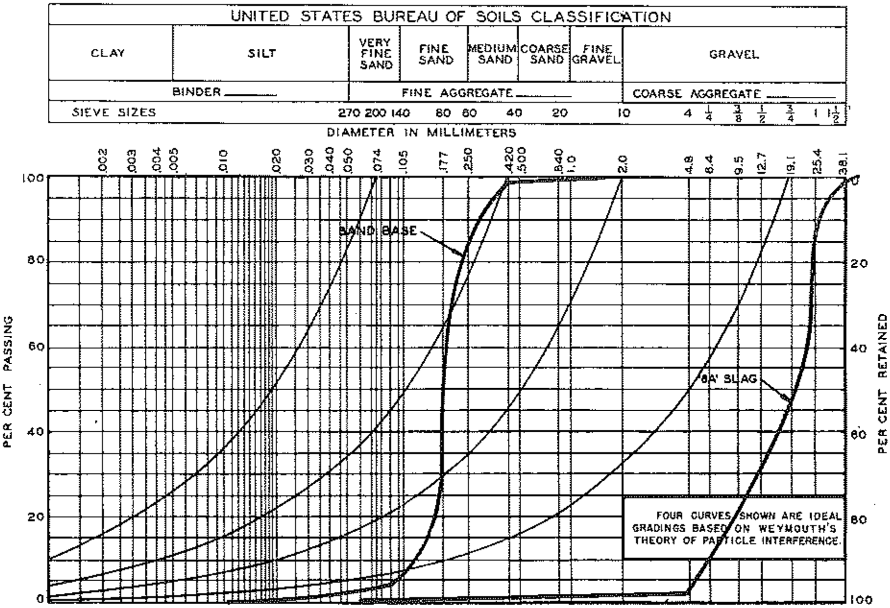


$\frac{\text{15 PER CENT SIZE OF FILTER MATERIAL}}{\text{15 PER CENT SIZE OF BASE MATERIAL}} = \frac{0.812}{0.094} = 8.6$ (PERMEABLE)

$\frac{\text{15 PER CENT SIZE OF FILTER MATERIAL}}{\text{85 PER CENT SIZE OF BASE MATERIAL}} = \frac{0.812}{0.210} = 3.87$ (STABLE)

Fig. 35. Gradation Test No. 4.

GRADATION TEST NO. 5



$\frac{\text{15 PER CENT SIZE OF FILTER MATERIAL}}{\text{15 PER CENT SIZE OF BASE MATERIAL}} = \frac{8.00}{0.159} = 50.3$ (PERMEABLE)

$\frac{\text{15 PER CENT SIZE OF FILTER MATERIAL}}{\text{85 PER CENT SIZE OF BASE MATERIAL}} = \frac{8.00}{0.290} = 27.6$ (UNSTABLE)

Fig. 36. Gradation Test No. 5.

POROUS BACKFILL GRADE 'B & C'

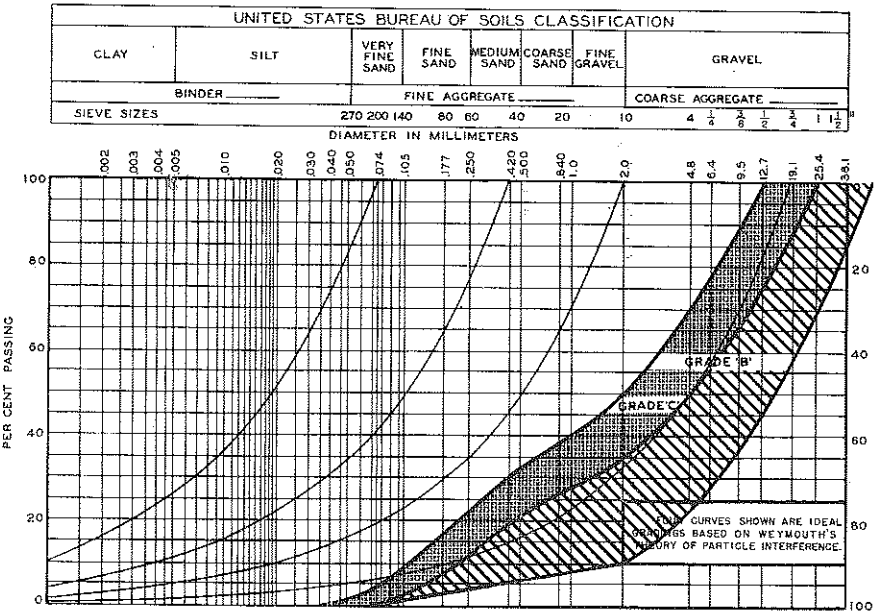


Fig. 37. Porous backfill grade B and C.

DUNE-SAND DENSITY STUDY

Bridgman Sand

TABLE I—*Loose densities with varying moisture content*

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids
1...........	0.09	93.50	43.7
2...........	1.13	78.00	53.0
3...........	2.37	76.97	53.9
4...........	3.50	75.36	54.6
5...........	4.50	75.25	54.7
6...........	5.90	74.71	55.4
7..........	7.40	74.71	55.0
8..........	11.90	75.63	54.5
9..........	12.2	75.63	54.3
10..........	15.6	73.34	55.8
11..........	17.6	76.70	53.8

TABLE II—*Loose densities and densities after inundation. Water introduced from top only*

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids
1..........	0.00	93.54	43.6
	23.75	96.06	43.5
	25.60	96.06	43.5
	26.50	96.34	43.5
2..........	0.00	94.04	43.4
	22.12	96.34	42.0
3..........	0.00	93.59	43.6
	23.04	96.34	42.0
4..........	0.00	93.59	43.6
	22.90	96.34	42.0
5..........	1.73	75.31	54.6
	26.03	88.93	46.4
6..........	1.09	76.35	54.0
	24.47	88.76	46.5
7..........	1.99	74.15	55.3
	0.00	93.63	43.6
	22.58	98.14	40.9
8..........	1.12	77.08	53.6
	27.70	88.65	46.6
9..........	0.00	94.97	42.8
	23.51	97.94	41.0
10..........	1.23	75.77	54.4
	24.16	86.10	48.1
11..........	1.04	77.06	53.6
	23.26	87.86	47.1

TABLE III—*Loose densities and densities after indirect inundation. Water introduced from sides of model into the sand. A load of 2½ pounds per square inch was maintained on the surface of sand*

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids	Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids
1..........	0.00	93.59	43.6	5..........	10.64	70.03	57.7
	27.50	95.88	42.2		21.38	88.17	46.9
2..........	1.35	75.39	54.6	6..........	5.30	73.10	56.0
	29.05	89.57	46.0		25.99	90.60	45.4
3..........	3.00	73.31	55.8	7..........	7.05	73.10	56.0
	27.50	89.02	46.4		25.13	90.40	45.5
4..........	0.00	93.59	43.6	8..........	11.19	71.50	56.9
	10.60	95.88	42.2		26.37	88.55	46.7

TABLE IV—*Dune-sand densities as determined by the cone method**

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids	Number of blows per layer	Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids	Number of blows per layer
1......	0.00	104.1	37.2	15	15......	7.30	102.8	38.1	15
2......	0.00	104.7	36.9	15	16......	7.40	103.3	37.8	20
3......	0.00	104.7	36.9	15	17......	6.90	104.2	37.2	25
4......	0.00	103.1	37.9	15	18......	10.20	103.1	37.9	15
5......	0.00	104.9	36.8	15	19......	9.80	105.2	36.6	20
6......	0.00	105.3	36.6	15	20......	9.80	104.7	36.9	25
7......	0.00	105.2	36.6	15	21......	8.50	105.3	36.6	20
8......	0.00	105.6	36.4	20	22......	13.60	105.0	36.7	15
9......	0.00	105.3	36.6	20	23......	13.70	105.4	36.5	20
10......	0.00	105.7	36.3	25	24......	11.70	107.0	35.5	25
11......	0.00	105.9	36.2	25	25......	0.00	104.9	36.8	15
12......	3.70	101.5	38.9	15	26......	0.00	107.0	35.5	20
13......	2.60	102.4	38.3	20	27......	0.00	107.1	35.5	25
14......	3.50	103.5	37.7	25					

*Cone method described in Appendix D.

TABLE V—*Dune sand, densities loose and densities determined by different degrees of vibration on the vibrating table*

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Type of vibration	Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Type of vibration
1.......	0.00	93.70 100.00	4 Minutes—Mild	5.......	0.00	93.70 106.44	4 Minutes—Hard
2.......	3.00	71.12 96.00	4 Minutes—Mild	6.......	3.09	71.35 99.01	4 Minutes—Mild
3.......	5.42	70.49 96.59	4 Minutes—Mild			106.41	4 Minutes—Hard
4.......	7.60	71.77 96.28	4 Minutes—Mild	7.......	5.12	71.99 97.19	4 Minutes—Mild
						102.30	4 Minutes—Hard

TABLE VI—*Dune-sand densities determined by standards A.A.S.H.O. Test T-99*

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids	Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids
1.	0.38	104.15	37.3	9.	1.24	100.5	39.5
2.	3.00	103.71	37.5	10.	2.73	103.9	37.4
3.	6.16	103.46	37.7	11.	4.66	103.5	37.6
4.	8.56	105.46	36.5	12.	6.42	104.1	37.3
5.	12.22	105.38	36.5	13.	8.38	104.4	37.1
6.	11.78	105.88	36.2	14.	11.11	105.0	36.7
7.	15.08	103.83	37.5	15.	13.02	105.9	36.2
8.	0.00	104.20	37.2	16.	13.89	105.3	36.5

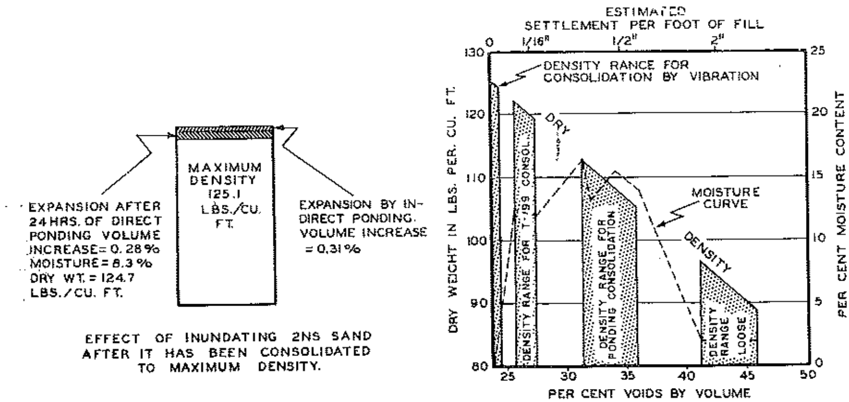


Fig. 38. Density characteristics of Michigan State Highway 2NS Sand Specification.

PEA-GRAVEL DENSITY STUDY

TABLE VII—*Loose densities of pea gravel with varying moisture content*

Remarks	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids
Average of 10 tests.	0.0	94.54	41.7
Average of 11 tests.	0.0	94.54	41.7
Average of 2 tests.	0.5	96.29	40.7
Average of 2 tests.	0.5	96.29	40.7
Average of 10 tests.	1.0	95.06	41.4
Maximum Density of 10 tests.	1.0	95.71	41.0
Average of 4 tests.	1.0	95.71	41.0
Average of 6 tests.	1.0	95.07	41.4
Minimum Density of 10 tests.	1.1	94.43	41.7
One test.	1.1	94.45	41.7
Average of 2 tests.	1.7	95.44	41.2
Average of 2 tests.	1.7	95.44	41.2
Average of 2 tests.	1.9	93.24	42.5
One test.	2.2	92.50	42.9
Average of 3 tests.	3.0	93.08	42.6
Average of 3 tests.	3.0	93.08	42.6
Average of 3 tests.	3.8	90.16	44.4
Average of 6 tests.	4.5	90.00	44.5

TABLE VIII—*Pea-gravel densities after inundation*

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids	Remarks
1........................	12.44	95.0	41.4	Indirect ponding
2........................	12.50	95.5	41.1	Direct ponding over surface
3........................	4.50	90.0	44.5	Saturated and drained
4........................	3.80	90.2	44.4	Saturated and drained
5........................	1.90	93.2	42.5	Saturated and drained

TABLE IX—*Pea-gravel densities after vibration*

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids	Remarks
1........................	12.38	101.7	37.2	Mild vibration
2........................	12.44	101.4	37.4	Mild vibration
3........................	9.70	106.6	34.3	Mild vibration for 3 minutes
4........................	10.07	104.8	35.4	Mild vibration for 2 minutes
5........................	10.14	105.2	35.2	Mild vibration for 4 minutes
6........................	9.63	104.9	35.3	Mild vibration for 3 minutes
7........................	10.88	103.3	36.3	Mild vibration for 2 minutes

TABLE X—*Pea-gravel densities determined by the cone method**

Test	Percent moisture	Density Lb./Cu. Ft. (dry)	Percent voids	Remarks
1........................	Dry	106.2	34.5	All samples consolidated in three layers with 20 blows a layer.
2........................	Dry	108.2	33.3	
3........................	Dry	107.7	33.6	
4........................	Dry	106.5	34.3	
5........................	Dry	106.8	34.1	
6........................	Dry	107.1	34.0	

*The Cone method of determining density is described in Appendix D.

APPENDIX B

TABLE I—*Consolidation tests made in model abutment*

Material	Percent moisture	Volume—Cu. Ft. Original	Volume—Cu. Ft. Final	Percent volume change	Density Lb./Cu. Ft. (dry)	Percent voids	Remarks
Dune sand	0.2				93.5	43.7	Loose
	0.2				100.3	39.6	Mild vibration
	0.2				106.4	35.9	Extended vibration
	11.0				105.9		Tamped—A.A.S.H.O., T-99
Bank-run gravel	3.16	15.6			106.0	27.0	Loose
	3.16	15.6	12.60	19.2	131.7		Mild vibration of total volume
	3.00				131.0		Continuous vibration while filling model
	4.69	11.73	10.00	14.7	144.3		Extended vibration of total volume
	3.26				141.3		Tamped in 4″ layers
Pea gravel	1.60	15.6	13.60	12.8	117.5	34.9	Total volume vibrated
	2.77	15.6	13.55	12.8	119.1		Total volume vibrated
	4.20				122.1		Tamped in 4″ layers
	0.76				128.1		Tamped in 4″ layers
	1.60				102.5	39.0	Loose
	2.77				103.5		Loose
Natural sand	4.77	15.6	13.00	16.7	108.6	31.0	Extended vibration
	4.77				90.2		Loose
Coarse aggregate	1.06	15.6	13.55	13.1	122.6		Extended vibration
	1.06				124.7		Tamped
	1.06				107.1		Loose
Slag	2.5	15.6	13.55	13.1	90.0		Extended vibration
	4.99				95.5		Tamped in 4″ layers
	2.50				78.1		Loose

TABLE II—*Test data, volume change and densities of different materials placed in the model abutment box*

Test	Material	Method of consolidation	Weight of box and gravel (pounds)	Weight of box	Weight of gravel (pounds)	Gravel surface below top of box		Volume (Cu. Ft.)		Percent volume change	Percent moisture	Density Lb./Cu. Ft. (dry)		Remarks
						Before consolidation	After consolidation	Original	Final			Before consolidation	After consolidation	
1....	Gravel........	Vibration.	1,923	425	1,498	4″	6″	11.73	10.00	14.7	3.65	123.0	144.4	After consolidation all test samples were saturated with no change in volume resulting.
2....	Sand..........	Vibration.	1,932	455	1,477	Full	2.63″	15.60	13.00	16.7	4.77	90.4	108.4	
3....	Pea gravel......	Vibration.	2,146	480	1,666	Full	2″	15.60	13.55	13.1	2.77	103.9	119.6	
4....	Coarse aggregate	Vibration.	2,147	475	1,672	Full	2″	15.60	13.55	13.1	1.06	106.1	122.1	
5....	Slag..........	Vibration.	1,695	445	1,250	Full	2″	15.60	13.55	13.1	2.50	78.2	90.0	
6....	Gravel........	Tamped..	1,980	425	1,555	....	5.38″	....	10.55	....	3.18	....	142.8	
7....	Sand..........	Tamped..	1,893	475	1,418	....	5.00″	....	10.80	....	5.40	....	124.6	
8....	Pea gravel......	Tamped..	2,031	480	1,551	....	3.75″	....	12.00	....	0.76	....	128.3	
9....	Coarse aggregate	Tamped..	2,075	475	1,600	....	3.00″	....	12.70	....	1.06	....	124.7	
10....	Slag..........	Tamped..	1,659	445	1,214	....	3.50″	....	12.10	....	4.99	....	95.6	

APPENDIX C

TABLE I—*Consolidation of bank-run gravel in a bin with a capacity of approximately 1,000 cubic feet*

Test	Description of consolidation method	Dry densities in Lb. per Cu. Ft.			Remarks
		Surface	1-Foot depth	2-Foot depth	
1.....	Loose density..............	107.5(1)	....	108.5(2)	(1) Average of 7 tests. (2) Average of 5 tests.
2.....	Vertical spears or tines on 2-foot layers.	109.2	108.1	107.1	Averages of 6 tests.
3.....	Paving tube vibrator in vertical position.	116.0	112.6	....	Averages of 5 tests.
4.....	Paving tube vibrator dragged over the surface.	111.5	107.5	107.3	10 minutes vibration. Before modification of vibrator.
5.....	Same as test 4.............	108.2	109.7	110.1	20 minutes vibration. Before modification of vibrator.
6.....	Same as tests 4 and 5. (See remarks).	115.6	113.7	113.5	Vibrator buried under 2 foot fill.
7.....	Tube vibrator dragged down sloping face of fill.	110.9	113.9	115.3	Vibrator modified as shown in Fig. 25.
8.....	Tamped..................	117.2	111.3	....	
9.....	Platform vibrator..........	109.2	110.3	106.0	Original platform vibrator.
10.....	Platform vibrator (modified)..	Average 109.4 after one pass. Average 111.8 after three passes.			
11.....	Filled one end of bin to the top allowing it to flow down at natural angle of repose. Dragged vibrating tube down through fill material, pushing some ahead of the vibrator and compacting it against the opposite end of the bin which simulates the abutment face.	111.0	113.7	115.5	Placed a 2-foot lift and vibrated 20 minutes. From these tests this method appears to be the most effective and adaptable for large volumes of backfill.

TABLE II—*Consolidation of sand backfill in metal crib retaining wall*

Test	Description of test	Moisture content percent	Dry density Lb./Cu. Ft.	Remarks
1....	Laboratory...	12.2	108.1	Tamped.
2....	Natural sand .	5.6	98.8	Undisturbed.
3....	Field check...	5.6	111.3	Prolonged tamping.
4....	Field check...	5.3	103.6	Normal tamping.
5....	Field check...	5.8	108.2	Normal tamping.
6....	Field check...	6.3	102.3	Normal tamping.
7....	Field check...	5.0	105.5	Normal tamping.
8....	Field check...	5.3	103.8	Normal tamping.
9....	Field check...	4.9	102.5	Vibrated—3-inch plank base with metal base plate having large corrugations.
10....	Field check...	5.8	99.9	Same as test 9.
11....	Field check...	6.8	101.6	Same as test 9 except metal base plate had small corrugations.
12....	Field check...	6.9	95.6	Same as test 11.
13....	Field check...	5.4	105.5	Vibrated—Plank base and sheepsfoot metal base.
14....	Field check...	5.9	103.3	Same as test 11.
15....	Field check...	5.1	102.2	Same as test 9.
16....	Field check...	5.7	104.4	Same as test 9.
17....	Field check...	5.7	109.0	Same as test 9 except two passes were made.
18....	Field check...	6.8	110.6	3-inch plank removed for remainder of tests. Base plate with small corrugations.
19....	Field check...	5.6	107.4	Same as test 18. Vibrated 4 minutes. One pass at 3600 v.p.m.
20....	Field check...	6.1	104.8	Same as test 18. Vibrated 5 minutes at 2400 v.p.m.
21....	Field check...	8.4	105.3	Same as test 18.
22....	Field check...	8.2	114.8	Prolonged vibrating with sheepsfoot base plate.
23....	Field check...	8.3	101.5	Same as test 18. Vibrated only 2½ minutes.

TABLE III—*Consolidation of Mayfield Bridge backfill*

Test	Stockpile density Lb./Cu. Ft.	Laboratory density Lb./Cu. Ft. (loose dry)	Laboratory density Lb./Cu. Ft. (cone)	Field density	Shrinkage stockpile to fill (percent)	Remarks
1.....	89.3	96.8	114.2	105.4	18.1	Tests 1, 2, 3 and 4 were made on a 33″ lift and were taken from depths of (2″-7″), (2″-7″), (12″-20″) and (26″-30″). Tests 5, 6 and 7 were made on a 15″ lift and from depths of (2″-8″), (2″-8″) and (12″-20″). Tests 8, 9, 10 and 11 were made on a 36″ lift and from depths of (2″-8″), (2″-10″), (12″-20″) and (33″-38″). Tests 12, 13, 14 and 15 were made, after vibrating a 12″ lift, from depths of (2″-8″), (2″-10″), (12″-20″) and (30″-35″).
2.....	89.3	96.8	114.2	104.0	16.4	
3.....	89.3	96.8	114.2	103.0	15.3	
4.....	89.3	96.8	114.2	94.5	5.8	
5.....	89.3	96.8	114.2	102.0	14.2	
6.....	89.3	96.8	114.2	101.7	13.8	
7.....	89.3	96.8	114.2	104.2	16.7	
8.....	98.8	103.6	113.6	107.0	8.3	
9.....	98.8	103.6	113.6	110.4	11.7	
10.....	98.8	103.6	113.6	110.6	12.0	
11.....	98.8	103.6	113.6	104.7	6.0	
12.....	98.8	103.6	113.6	110.1	11.4	
13.....	98.8	103.6	113.6	116.0	17.4	
14.....	98.8	103.6	113.6	110.2	11.5	
15.....	98.8	103.6	113.6	104.9	6.2	

On this project approximately 6 feet of backfill was placed before consolidation was started. Three feet of the fill was placed under water. This type of operation does not result in as good densities as the densities obtained when the entire embankment, from the first layer through to the top, is consolidated.

APPENDIX D

THE COMPACTION AND DENSITY OF GRANULAR SOIL

Scope:

1. This density test for granular soil determines the dry weight per cubic foot under a standard method of compaction.

Apparatus:

2. The apparatus shall consist of the following:

a) Mold — A funnel-shaped mold having a solid bottom in the large end and equipped with a stopper for the small end. The bottom shall be so shaped that there will be no sharp corners inside the mold. The base or large end of the mold shall be approximately 5¾ inches in diameter and the small end shall be not less than 2½ inches. The mold shall be approximately 8½ inches in height and shall have a volume of approximately 1,300 cubic centimeters or 0.0459 cubic feet (Fig. 39).

b) Balances — A balance or scale of 6,000-gram (12-pounds) capacity sensitive to 1.0 gram or 0.01 pound, and a balance sensitive to 0.10 gram.

c) Drying — Any approved method for drying soil samples.

Procedure:

3. a) A 3,500-gram sample shall be taken from the source of the material to be used.

b) The sample shall be thoroughly mixed, then compacted in the mold in three equal layers, each layer receiving 25 blows. The blows shall be delivered by raising the mold approximately 4 inches and striking it sharply down on a concrete or heavy timber base. After the third layer has been placed the blows shall be continued with the wood stopper reversed and held firmly over the opening. Sand shall be added at intervals to keep the mold full, and operations continued until no further consolidation occurs. The compacted soil shall be carefully leveled off to the top of the mold and weighed.

c) The weight of the compacted sample and mold, in pounds, minus the weight of the mold, shall then be divided by the volume of the mold in cubic feet and the result recorded as the wet weight in pounds per cubic foot of the compacted soil.

The wet weight per cubic foot can also be obtained by dividing the weight of the sample in grams by the volume in cubic centimeters and multiplying by 62.4.

d) The compacted mass of soil shall be removed from the mold and a sample of approximately 100 grams taken from the center of the mass shall be weighed immediately and dried to determine the moisture content.

Calculations:

4. The moisture content and the dry weight of the soil as compacted shall be calculated by the following formulas:

$$\text{Percent moisture} = \frac{\text{A-B}}{\text{B-C}} \times 100$$

where A is the weight of dish and wet soil,
B is the weight of dish and dry soil, and
C is the weight of the dish

$$\text{Dry wt./cu. ft. of compacted soil} = \frac{\text{Wet wt. in lbs./cu. ft.} \times 100}{\text{Percent moisture} + 100}$$

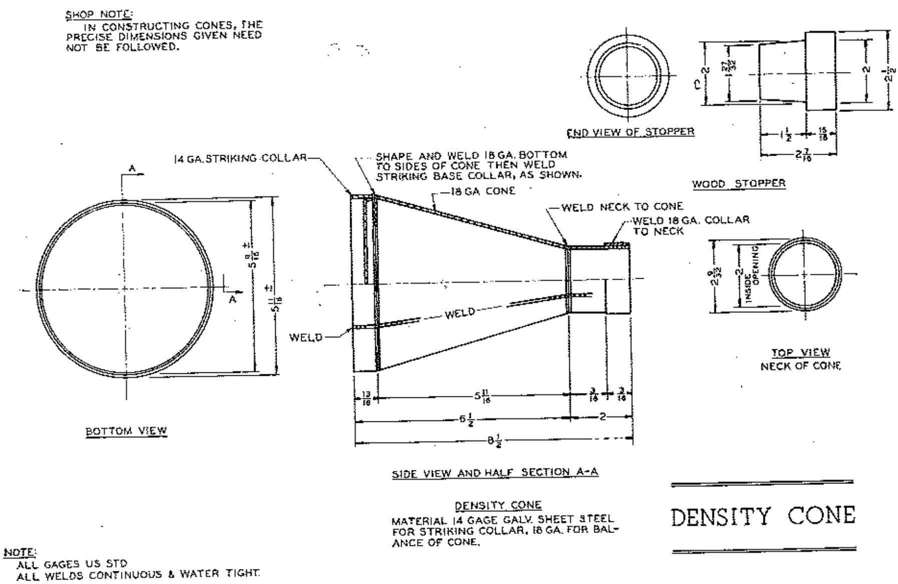


Fig. 39. Density cone.

BULLETINS OF THE MICHIGAN ENGINEERING EXPERIMENT STATION

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No. 38. "Design and Construction of the Telomat," by W. H. Bliss. 1931.

No. 41. "Some Observations on the Dilation of Several Common Steels," by H. E. Publow, C. M. Heath, and M. E. Batchelor. 1931.

No. 43. "The Electrodeposition of Tin from Solutions of Sodium Stannate," by D. T. Ewing and Alfred Clark. 1932.

No. 53. "The Structure of Carbon Steel at Elevated Temperatures and Under Reduced Pressure," by H. E. Publow and T. D. Parker. 1933.

No. 62. "The Relation Between Axial Length Changes and Rockwell Hardness of Carbon Steels," by H. E. Publow and Lyle Clark. 1935.

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