GRADATION AND CONSOLIDATION OF POROUS BACKFILL MATERIALS

Thesis for the Degree of

Civil Engineer

MICHIGAN STATE COLLEGE

Gail Charles Blomquist

1948

This is to certify that the

thesis entitled

GRADATION AND CONSOLIDATION OF POROUS

BACKFILL MATERIALS

presented by

GAIL CHARLES BLOMQUIST

has been accepted towards fulfillment
of the requirements for

Professional degree in Civil Engineering

C L Allen
Major professor

Date May 18, 1948

M-795

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By

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A THESIS

Submitted to the School of Graduate Studies of Michigan State College of Agriculture and Applied Science in partial fulfillment of the requirements for the degree of

CIVIL ENGINEER

Department of Civil Engineering

1948

THESIS

6/9/47
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FOREWORD

One of the most important problems today for any structural or highway engineer is that of settlement in foundations or of substructural materials. While this may be a relatively simple problem as compared to other problems of soil mechanics it is a very important one to the highway engineer. Generally, even the most elaborate subsoil investigations fail to give a complete picture of all conditions, hence the application of any such investigations must be tempered with judgment and experience.

The deformation of soils is much greater than that of steel or concrete. The usual construction materials are considered rigid and to follow the principles of elasticity within their elastic limits. Soils, on the other hand, must be considered as plastic solids under some circumstances and as viscous liquids under others. The deformation of many types of soils is a progressive action and will not reach its maximum for some period of time. The deformation of a soil mass results from a change in both shape and volume as compared to other structural materials which only change in shape. In the study of settlements the volume change of the subsoil layers is more important than deformation due to change in shape.

Settlements of structural approaches, fill settlements and failure of pavement surfaces have brought the point to light that the selection of materials which will eliminate excessive volume changes after placement is very important to the highway engineer and also to the field of highway construction. The use of better materials and closer control has decreased the settlements on these structures considerably, but it has not proven to be the panacea for the entire problem since settlements are still apparent in structural approaches and irregularities are found in the surfaces of our modern highways.

With an aim of finding methods for further limiting the above mentioned objectionable settlements and subsequent failures of structures, a study has been carried out on the gradation and placement of embankment and structural backfill materials. This report is a presentation of the results and tests made and also the results of studies on gradation and consolidation of granular backfill materials.

GRADATION STUDIES

One of the functions of a granular backfill is that of promoting drainage away from the structure of any water seeping into the backfill material from adjacent soils or any water that might gain entrance from the surface. The gradation of a good backfill material must be such that it will allow water to pass through it without being trapped or ponded on any part of the subgrade or on the structure proper. However, any water passing into and through a backfill material in all probability will carry with it the finer particles of the adjacent soils from which it is entering, or it also may carry away the fines of the backfill material as it drains. In either event it will lead to subsequent settlement, by the formation of voids, in either the backfill material or in the adjacent soil allowing the backfill material to change shape or be decreased in volume. If the fines from the adjacent soil are carried into the backfill material it will become choked and ponding will occur on the surface or in adjacent soil masses making them viscous, and deformation will occur causing settlements in the backfill material.

A backfill material should be of such texture that it can be placed during any weather condition and will not require special handling; for example, drying before being placed. The ideal material would meet all of these requirements and also require a minimum of effort to consolidate it to such density that no further detrimental settlement would occur.

It has been definitely established that any type of backfill material when placed loose will undergo rearrangement of particles due to the vibrations set up by traffic. The amount of settlements will vary with types of backfill material and the size and proportions of the backfill in place. From the foregoing facts it is evident that the material should be well consolidated before placing the road surface over it.

The United States Corps of Engineers made a comprehensive study of the above requirements of soils in the Waterway Experiment Station at Vicksburg, Mississippi. From their studies the following formulae were developed:

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 satisfactory permeability -

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

The above formulae to be applied following laboratory investigations, to each type backfill material, each change in subgrade soil, and each type of outlet having varying size openings.

The scope of investigations covered herein includes a study made with the view in mind of establishing a gradation range for all granular backfills. The gradation range established included the majority of the natural gravels and sand deposits of Michigan. (These studies were carried on in the Michigan State Highway Laboratory.)

The studies included standard Mechanical Analysis tests and filter tests on varying kinds of natural materials. Tabulated records of these studies can be found in Appendix A.

The filter tests were carried out in a glass faced rectangular tank 30 inches in length, 13 inches high, and 4 inches wide. Imbedded in the material and an integral part of the tank were two cylindrical metal screens 1-1/4 inches in diameter having 1/32 square inch openings spaced 1/32 inch apart. These two screens were placed six inches apart and 3-3/4 inches above the bottom of the tank, being symmetrically located on the face of the tank. These screens provided an outlet, or overflow, for the water filtering through the material in the tank after water was introduced from the top of the tank.

The first filter test was run with a coarse aggregate having the following gradation:

Size of Sieve	Per Cent Passing
1"	70
1/2"	45
# 4	20
# 8	14
# 10	10
# 20	7
# 40	5
# 50	4
#100	2
#200	0

The material was thoroughly mixed and was then placed in the tank. Water was introduced into the tank through a sloping trough that had openings provided in its bottom to allow the water to fall on the surface of the material uniformly.

The water filtered down through the material and seeped out through the two screen openings into overflow cans where all water and sediment were collected. Following a twenty-four hour period, during which the water was being introduced continually, the sediment in the overflow cans was collected and dried. This material totaled 8 grams, dry weight. Figure 1 is a view of the face of the tank following the twenty-four hour test.

Figure No. 1. Coarse aggregate filter test following twenty-four hour test.

Following the twenty-four hour test, 30 pounds of sand was spread in a uniform layer over the top surface of the coarse aggregate. The sand used had the following gradation:

Size of Sieve	Per Cent Passing
# 40	100
# 60	93
#140	18
#200	9
#270	5

Before water was introduced on the surface the screen openings were closed by means of valves provided to maintain a head on the filter tank. Water was introduced as before and allowed to fill the tank to about 2 inches covering the sand layer. When this level was reached the valves on the screen openings were gradually opened, allowing water to drain off. At the same time, additional water was introduced to maintain a 1-1/2 inch head on the sand layer. As before the sediment was collected from the overflow. The valves were opened gradually and were completely open at the end of a three-hour period. The amount of water added to maintain the head decreased gradually for several hours. After running the test for five hours no movement of material was evident within the tank. Following the test the sediment in the overflow cans was dried and weighed. The dry weight of the sediment was 52 grams.

Figure 2 shows the material in the tank at the conclusion of the test.

Figure No. 2. Coarse aggregate filter test after sand was added to the surface and water allowed to pass through.

The gradation of the material in the tank following the two foregoing tests was as follows:

Size of Sieve	Per Cent Passing
1"	74
1/2"	50
# 4	26
# 8	16
# 10	15
# 20	13
# 40	12
# 50	10
#100	2
#200	0

The second filter test was carried out with a finer material as a medium. The gradation was as follows:

Size of Sieve	Per Cent Passing
# 4	70
# 8	57
# 10	50
# 20	40
# 40	30
# 50	25
#100	20
#200	5

The material was thoroughly mixed and placed in the tank. Water was introduced and allowed to rise to a head of four inches over the material. This is shown in Figure 3.

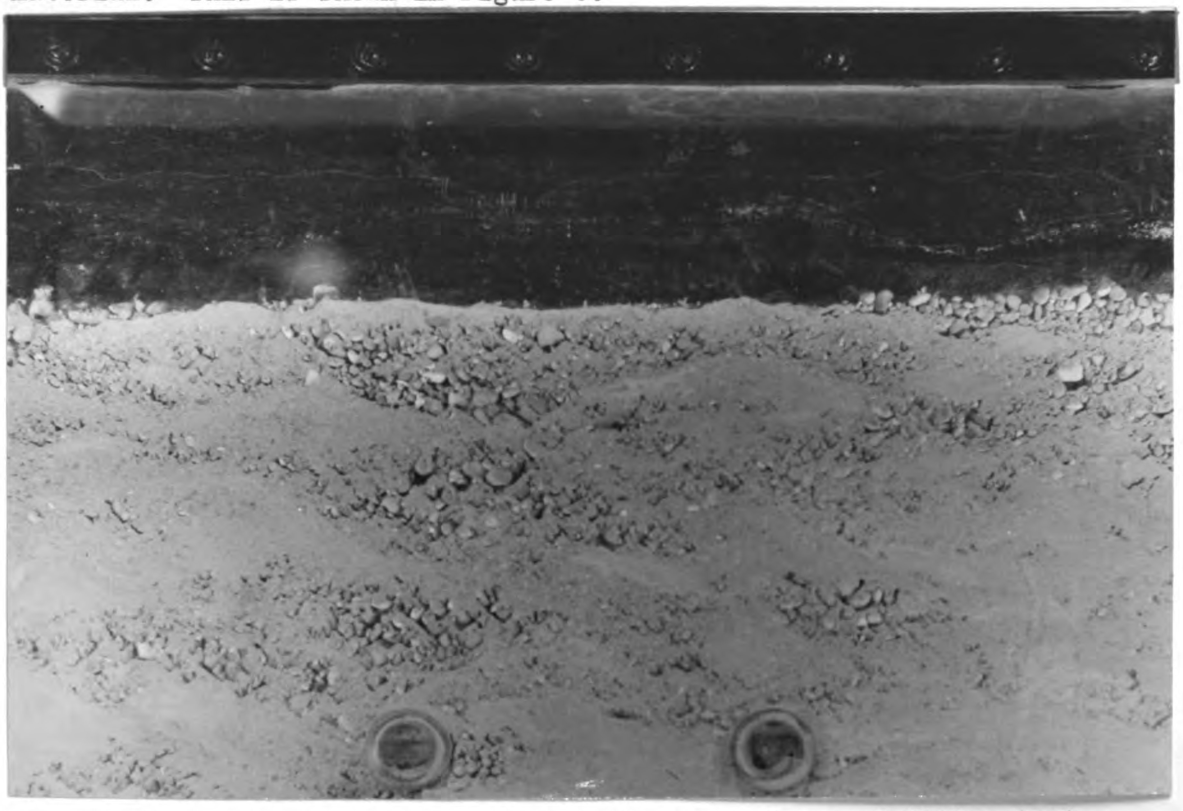


Figure No. 3. View showing material at start of second filter test.

The valves were opened as before and water introduced to maintain a constant head or as near as possible. The water drained off slowly and diminished its rate gradually. This operation was continued for eighteen hours. During the test period water was filtering through at the rate of 2.08 gallons per hour. Following the eighteen-hour test the tank was allowed to drain and the sediment in the overflow cans was dried and weighed. During the entire test only 1.8 grams of material was filtered out. Figure 4 shows the tank following the test.

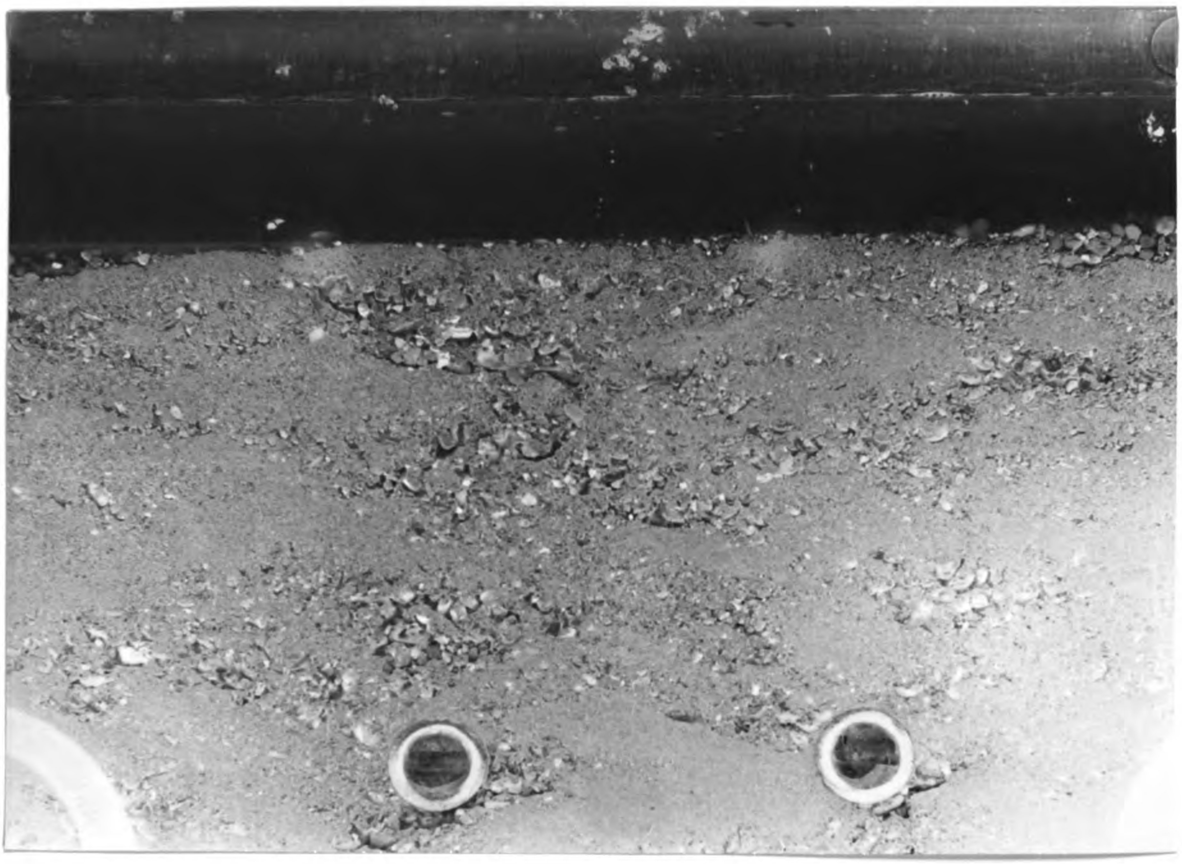


Figure No. 4 Fine gravel and sand following eighteen-hour ponding test.

The ponding was continued for an additional 2 hours at which time a uniform layer of 30 pounds of silty loam material was spread in the water on top of the fine gravel and sand filter. The gradation of the silty loam was:

Size of Sieve	Per Cent Passing
# 40	100
# 60	99
#140	94
#200	79
#270	65
* .005 mm	

* Mechanical Analysis

With the addition of the silty loam the amount of water passing was reduced to 1.52 gallon per hour. The test was continued to 36 hours at which time only 0.98 gallon per hour was passing through with the same head as at the start of the test. During the last few hours of the test there was no apparent decrease in the flow below 0.98 gallon per hour. The sediment cans contained a total of 0.3 grams of material following the addition of the silty loam filler. Figure 5 is a view of the tank at the completion of the test.

A six-inch layer of material was removed from immediately over the screens and a gradation study made on it as follows:

Size of Sieve	Per Cent Passing
# 4	69.1
# 8	53.4
# 10	50.7

(continued)

Size of Sieve	Per Cent Passing
# 20	40.1
# 40	24.4
# 50	19.0
#100	4.6
#200	1.1

The above gradation when compared to the original gradation will show very little of the silty loam soil was carried into the material, or through it, from the sediment weight following the test.

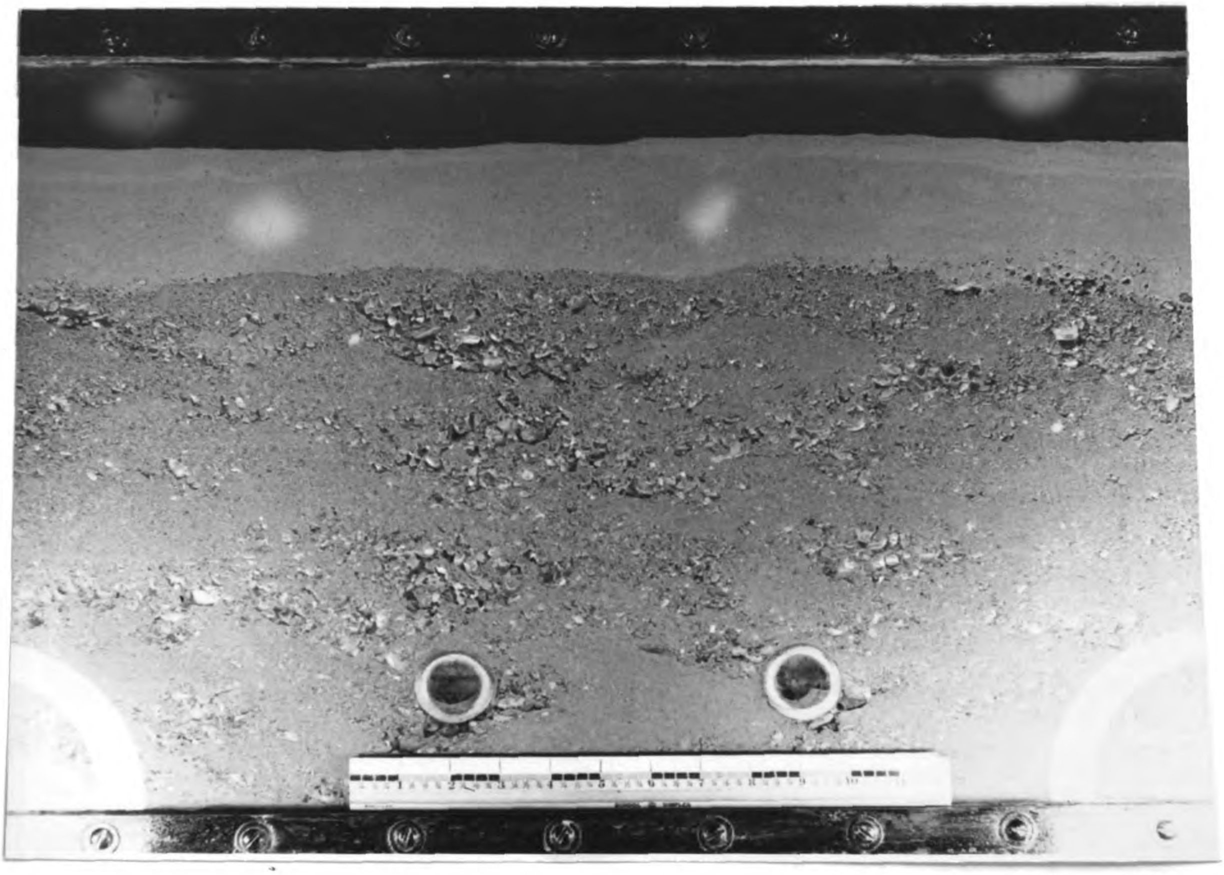


Figure No. 5. Fine gravel and sand material after 36-hour ponding.

The standard Highway 6-A Specification gravel was used in the third filter test. The gradation of the 6-A gravel is as follows:

Size of Sieve	Per Cent Passing
1-1/2"	100
1"	79
1/2"	33
# 4	0.5

Following a thorough mixing, the gravel was placed in the tank and water introduced as before. As compared to the foregoing tests, the only limit to the amount of water flowing through the material was the size of the outlet pipes. Figure 6 shows a view of the tank prior to the test.

Figure No. 6. Standard 6-A gravel in tank before test.

After testing on the 6-A gravel for a short time without any evident change in its structure or any sediment carried out by the water, a layer of the 6-A gravel was removed and replaced by 27-1/2 pounds of sand spread in a layer of uniform thickness. The gradation of the added sand was:

Size of Sieve	Per Cent Passing
# 40	100
# 60	93
#140	18
#270	5

With only one of the outlets open to flow the test was continued as before. There was no apparent reduction in the rate of flow as compared to the 6-A gravel test. A view of the tank under these conditions can be noted in Figure 7. The sand was observed to move through the 6-A gravel with little difficulty. Piping can be noted in the upper left corner of the tank.

When the second valve was opened the piping effect was accelerated leaving a very distorted surface on the sand layer, as shown in Figure 8.

With the rapid flow of water through the filter medium the sediment carried through was not recovered, consequently no data on per cent passing through the tank was available. The rate of flow through the tank was approximately four gallons per minute.

A mixture of 6-A gravel and 2NS sand was used in test No. 4. The proportions were 70 per cent and 30 per cent respectively.

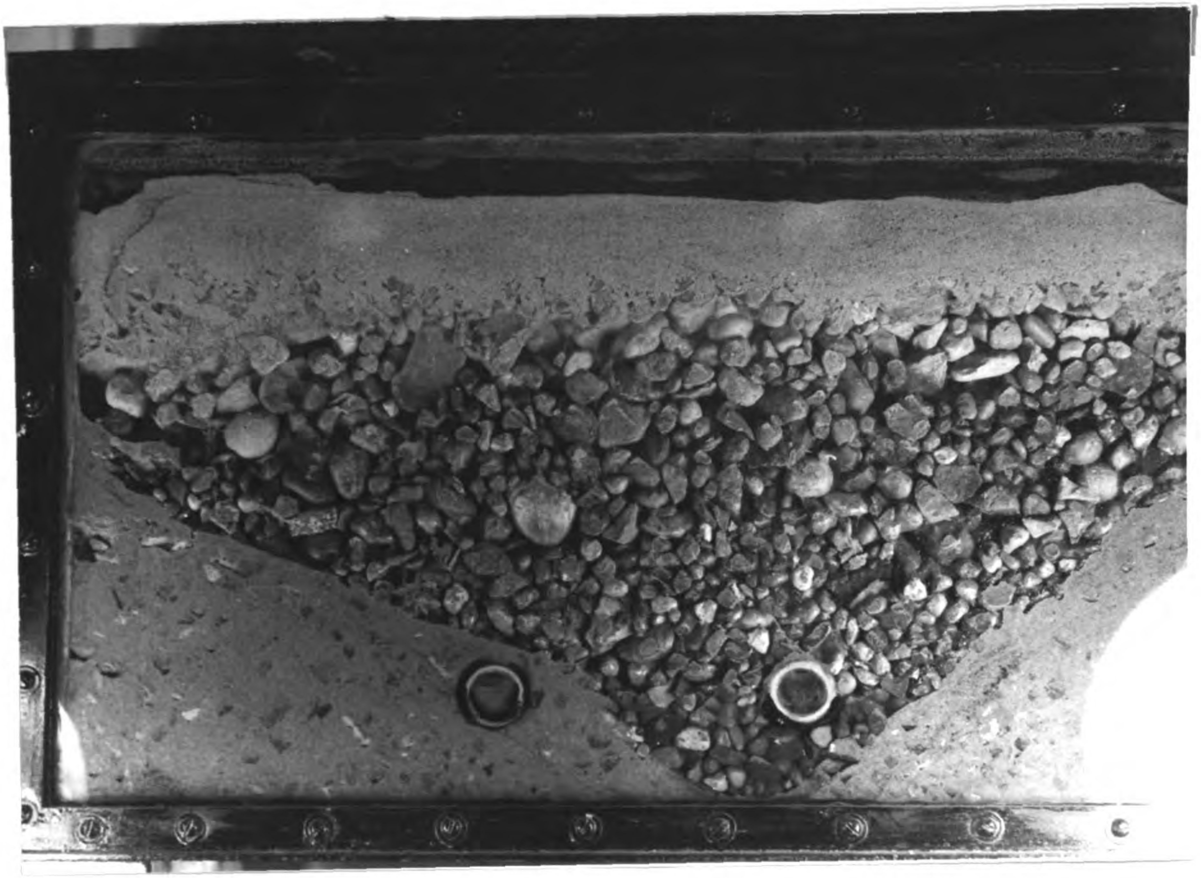


Figure No. 7. 6-A aggregate test showing the movement of sand through the backfill material.

The gradation of the 6-A gravel was as follows:

Size of Sieve	Per Cent Passing
1-1/2"	100
1"	75
1/2"	40
# 4	0

The 2NS sand had the following gradation:

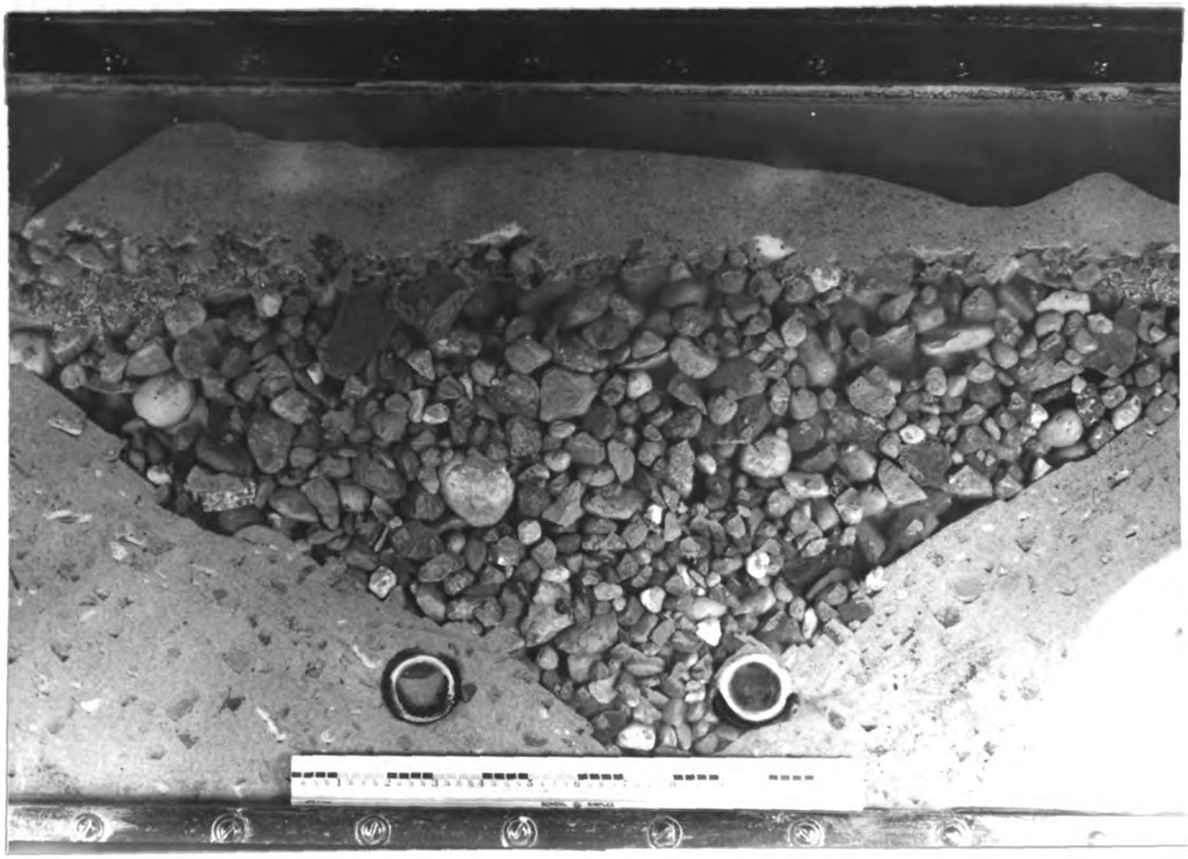


Figure No. 8. 6-A aggregate test showing distortion of sand surface.

Size of Sieve	Per Cent Passing
# 4	99.7
# 8	86.7
# 16	61.9
# 30	38.1
# 50	9.6
#100	1.2
Loss on Washing	0.6

The two materials were mixed thoroughly and placed in the tank. As noted in Figure 9, segregation occurred during the placing of the material despite all precautions and care taken. Additional segregation and consolidation of the entire filter was evident with the addition of water.

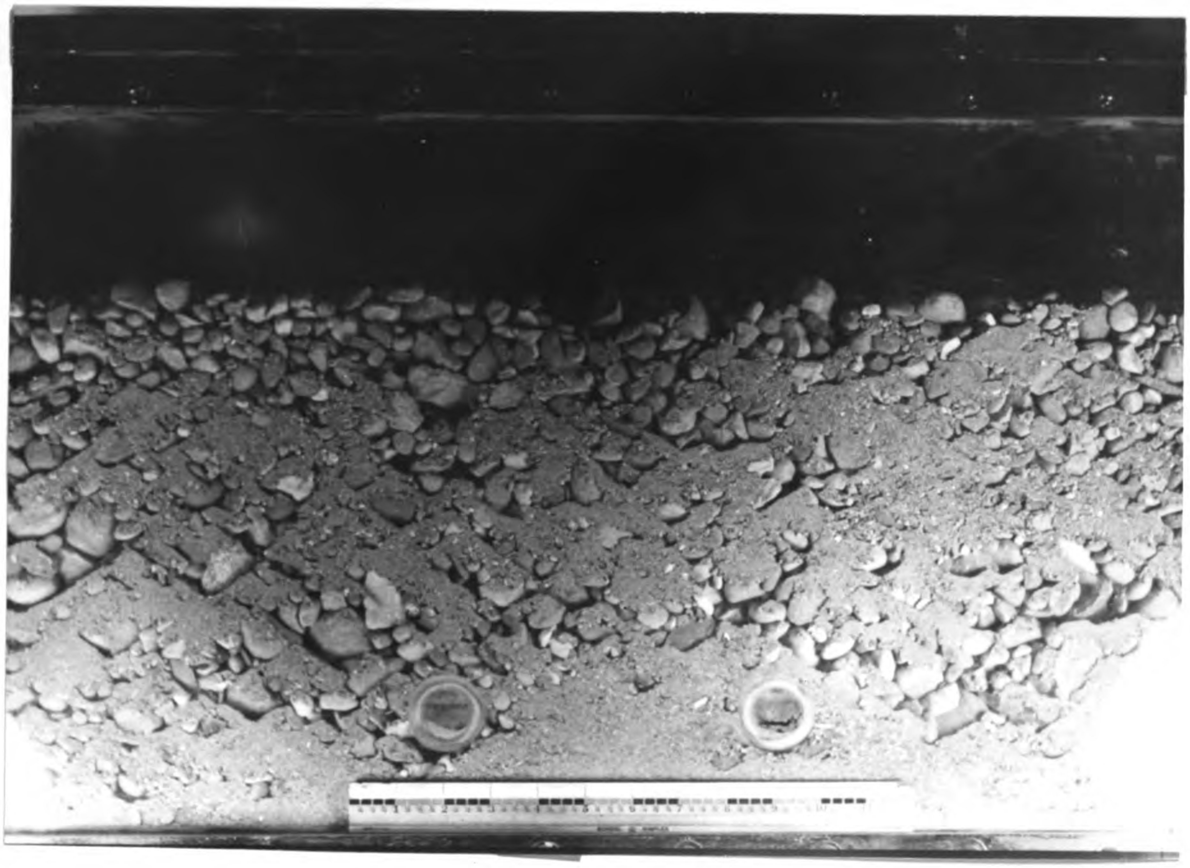


Figure No. 9. 6-A aggregate and sand filter before test.

The water was introduced into the tank until a head of several inches was established over the surface of the filter material. While holding this head level, a uniform layer of 26 pounds of sand was spread over the filter surface. The head was then maintained at one inch over the surface of the sand. The gradation of the sand layer added was as follows:

Size of Sieve	Per Cent Passing
# 40	100
# 60	93
#140	18
#270	5

Following the addition of the sand layer, water was flowing through the outlets at the rate of 28.0 gallons per hour. Seven hours of continuous running under the one-inch head of water, the rate of flow had decreased to 15.3 gallons per hour.

The surface was distorted by piping action and the sand had penetrated to a depth of several inches in many portions of the filter material.

The test was shut down and allowed to drain for several hours. Again the water was started with a resulting discharge of 14 gallons per hour. Five hours following the second portion of the test, the flow through the discharge pipes had dropped to eight gallons per hour. Figure 10 shows a view of the test following forty-eight hours of testing.

A layer of the filter material immediately above the screens was removed and tests carried out on it. The resulting gradation of the material was as follows:

Size of Sieve	Per Cent Passing
1-1/2"	100
1"	83.8
1/2"	56.6
# 4	30.9

(continued)

Size of Sieve	Per Cent Passing
# 10	25.4
# 20	17.0
# 40	8.8
# 60	3.2
#100	0.3
#200	0.2

Figure No. 10. Note penetration of fines into the filter material after 48-hour test.

Following the 48-hour test the sediment collected was dried and weighed. The amount was found to be only 0.5 grams. The small amount of sediment collected following the extensive test indicated the material at the screens was functioning as a very efficient filter.

The last test was carried out on a commercial filter material composed of standard 6-A slag with the following gradation:

Size of Sieve	Per Cent Passing
2"	100
1-1/2"	98
1"	73
1/2"	31
# 4	1.8
Loss on Washing	0.8

The slag was placed in the tank with all precautions to prevent any segregation before the introduction of the water. Water was introduced and allowed to accumulate to a head of three inches over the slag filter. While maintaining this head a layer of sand of the following gradation was spread over the slag:

Size of Sieve	Per Cent Passing
# 10	100
# 20	99.7
# 40	99.2
# 60	79.9
#140	5.6
Loss on Washing	3.0

As in the test of the filter of standard 6-A gravel, only one of the valves was opened at the start of the test with the result that the water moved through the filter very rapidly and carried with it much of the sand, depositing it in the slag. The water also carried a considerable amount of the sand out through the outlets into the sedimentation cans.

The valve was then closed and the second valve opened. The water drained out very slowly following the opening of the second valve indicating a choking up of the drain by the sand carried down during the first part of the test. The first valve was again opened with a resulting discharge as great as before which would further indicate the impervious layer over the second drain. A view of the slag filter at the close of the described test is shown in Figure 11. The distortion of the surface of the sand layer can also be seen in Figure 11.

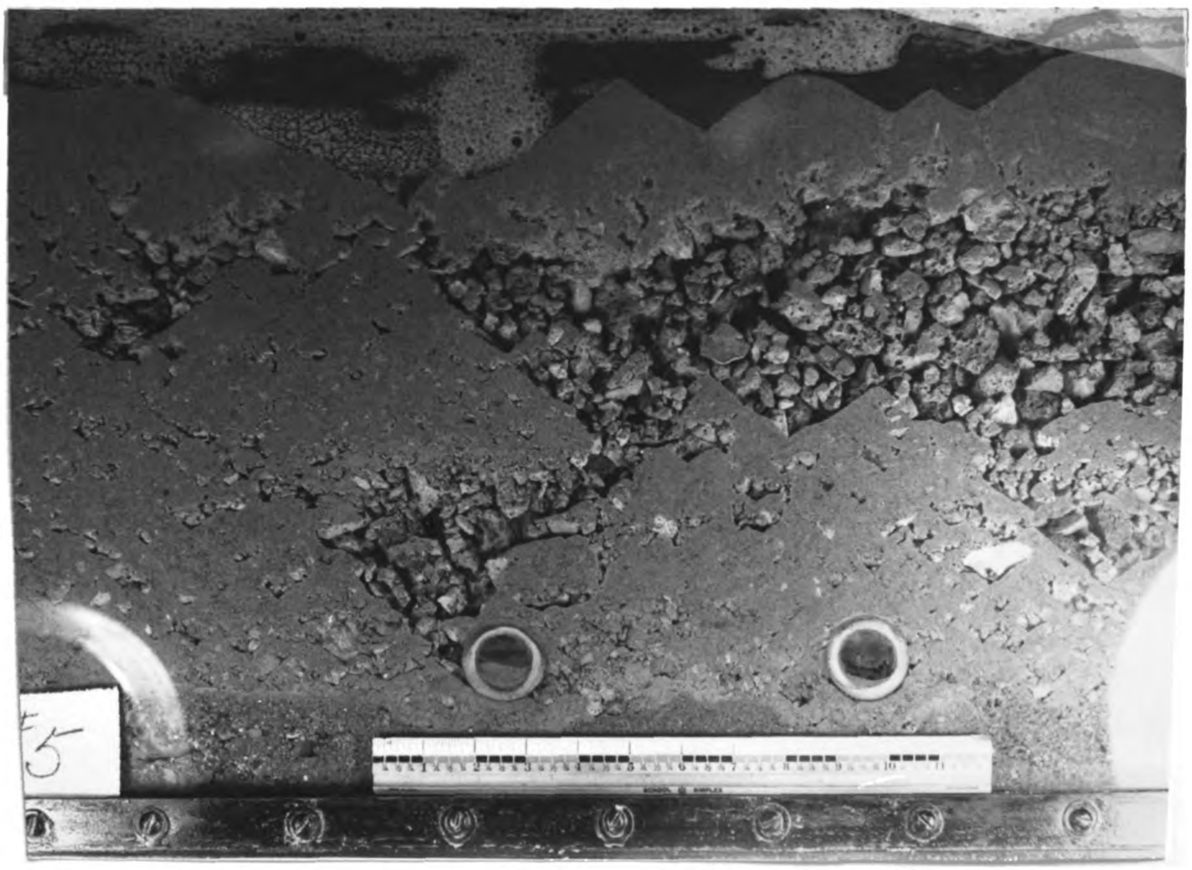


Figure No. 11. A 6-A slag filter showing infiltration of sand filter.

After a careful analysis and study of the foregoing tests and their results, a specification for granular backfills was decided upon. The specification, which is quoted in the following paragraph, is believed to cover most requirements of a granular backfill and the materials are readily available and obtainable in practically all of the sand and gravel deposits of Michigan.

The specification of the over-all requirements of a granular backfill is as follows:

Size of Sieve	Per Cent Passing
2"	100
1"	70 - 100
1/2"	45 - 100
# 4	20 - 70
#10	10 - 50
#40	5 - 30
Loss on Washing	0 - 5

The problem of a backfill material to meet the requirement where the adjacent soil is largely silt or very fine sand (.005 to 0.105 mm) could not be covered in an over-all specification that would function favorably on most soil types encountered. Therefore, the following specification is recommended for use in backfills of this class:

Size of Sieve	Per Cent Passing
1"	100
1/2"	75 - 100
# 4	50 - 70
#10	35 - 50

(continued)	Size of Sieve	Per Cent Passing
	#40	20 - 30
	Loss on Washing	0 - 5

Backfill materials recommended in the first specification can be found in natural deposits in Michigan and are available to the majority of construction projects.

The material recommended in the second specification may be found in natural deposits in some localities and in others it would require processing to proper gradation. This material is such that there will be no infiltration of the adjacent material.

The tests described bring out concisively the need for proper gradation for granular backfill materials. Of equal importance with the gradation is the method of placing these materials.

CONSOLIDATION STUDY

Following the tests to determine the specification for the gradation of granular backfill materials the study was centered on methods of placing materials in backfills and their manipulation to achieve the desired densities. This portion of the study was carried out on scale models in the laboratory and later moved into the field on a larger outdoor model. On the outdoor model heavy construction equipment was employed while in the laboratory only simulated and comparative tests were conducted. The next and final step was to move the study into the field on actual construction projects in various portions of Michigan. A controlled density study of backfills under construction was the purpose of this phase of the study. As part of this final study, a density investigation was carried out on backfills in place that varied in age from one to several years.

The first test on methods of placing and consolidation was carried out on dune sand and pea gravel. These two materials were chosen for their uniformity of gradation. Methods of consolidation were by ponding, tamping and vibration. The tabulated results of these tests can be found in Appendix A. A brief discussion of the tests will be covered in the following paragraphs.

Approximately 95 tests were conducted on the dune sand. Of this number 30 were on the material placed in its loosest possible state; 21 tests were carried out to study consolidation by flooding or ponding; 9 tests were carried out employing vibration as a means of consolidation, and 16 tests with tamping.

The tests carried out on the loose material were to study the effects of bulking, voids, and to determine a loose volume density. All of these were determined for varying moisture contents which were used for comparison in later tests.

The density of dune sand with less than one per cent moisture was found to be 93.5 pounds per cubic foot. The average density of ten samples with moisture contents varying from 1.13 per cent to 17.6 per cent was found to be 75.63 pounds per cubic foot. The average moisture content of all the tests in the loose state was 8.21 per cent with a moisture content of 1.13 per cent yielding the high density of 78.0 pounds per cubic foot and a low density of 73.34 pounds per cubic foot with a moisture content of 15.6 per cent.

The average per cent voids for the ten samples, with varying moisture contents, was 54.4

From the above tests it would be a good assumption that any loose deposit of dune sand, or a sand of comparable gradation and specific gravity, would approximate a density of 76 pounds per cubic foot with a total per cent voids of about 54.

Following the determination of the above standards for dune sand, tests were carried out to experiment with varied methods of consolidation with the view of reducing the voids ratio to a minimum with the minimum effort and high efficiency. The moisture content was varied in each test to also study its relationship to consolidation and efficiency.

Inundation Tests: The first method of consolidation tried was inundation with water. Before the tests were carried out on the material in this series it was found to have moisture contents below one per cent and a loose density of less than 93 pounds per cubic foot. The voids totaled 43.4 per cent for all of the material prior to flooding. The average density of all tests following the inundation period was 97 pounds per cubic foot. The moisture content immediately following the draining off of excess water from the material was found to be 21.5 per cent. A total of five tests were carried out to arrive at the above average data.

Similar tests were carried out on six samples of dune sand with moisture contents ranging from one to two per cent. Average density and per cent voids prior to flooding were 76 pounds per cubic foot and 54.2 per cent respectively. The average moisture content was 1.37 per cent for all six trials. Following the flooding the density averaged 88 pounds per cubic foot and an average per cent voids of 47. The moisture content taken, as before, was found to be 25.2 per cent.

The above tests tend to indicate that the per cent of moisture in the dune sand prior to test has a marked effect on its consolidation range by the inundation method. The second series of tests indicate a final density which was lower than the loose density when the moisture content was less than one per cent.

The next series of tests, four in number, were also run on dune sand but varied in the sense that the water was introduced from the bottom of the sample instead of ponding it on the surface. This series of tests were also carried out on the material in its loosest state with moisture content varied as before. The first series of four tests had an average density, prior to inundation, of 93.9 pounds per cubic foot, voids totaling 43.4 per cent, and an average moisture content of 0.156 per cent. Following the test with indirect inundation the density had increased to an average of 95.8 pounds per cubic foot, and the voids decreased to 42.2 per cent with the moisture content averaging 27.5 per cent before complete draining.

The indirect inundation tests on material having a moisture content in excess of one per cent gave results of an average density of 89.6 pounds per cubic foot and total voids of 46 per cent. The average density before the test was found to be 74.7 pounds per cubic foot resulting from a low density of 70 pounds per cubic foot and a high of 89.1 pounds per cubic foot with moisture contents of 10.6 per cent and 1.29 per cent respectively.

The conclusion from the first group of tests is further borne out by the results of this group of tests. Low moisture contents before inundation resulted in greater densities with lower voids.

<u>Vibration Tests</u>: In the next series of tests the dune sand was consolidated by means of vibration. The vibration was accomplished on a vibrometer table which is mounted on springs that can be either employed or not as desired. The springs tend to dampen out the sudden shock of vibrating motion when they are employed in the system. The vibration is obtained by a conventional type vibrating motor attached to the under side of the table. In these tests the term "mild vibration" will be used to indicate the springs are in force and "extreme vibration" when they are not.

Two tests were carried out with dry dune sand. The first of these tests was under mild vibration for four minutes and it gave a resulting density of 100.3 pounds per cubic foot, with 39.6 per cent voids. The second test was under extreme vibration conditions for a period of four minutes and it resulted in a density of 106.44 pounds per cubic foot and a per cent voids of 35.9.

Five other tests were run on the same material but with varying moisture contents. The first three of these tests were with mild vibration for four minute periods. The average density that resulted was 96.3 pounds per cubic foot and 42 per cent total voids. The average moisture content for the three samples was 5.36 per cent. The average density and per cent voids for the two tests with vibration were 104.35 pounds and 37.1 per cent respectively. The average moisture content was 4.1 per cent.

The above results on vibration tend to indicate a slight lowering of the attained density by increasing the moisture content, but there are not sufficient test results to draw any definite conclusions. The difference between mild vibration and extreme vibration are evident in the results, but again not conclusive.

Cone Tests: The cone test is a method of consolidating granular materials to compare attained densities of varying samples to a standard. The method compares favorably with the results obtained from the density tests by extreme vibration. The relative merits of this test over the vibrometer table test is the simple and rapid means of checking density requirements in either the field or the laboratory.

Appendix B contains a complete description of the test and equipment along with data on 14 tests carried out on dry dune sand and 13 tests on dune sand with varying amounts of moisture. From the data sheets it was found that the average density for the 14 dry samples was 105.25 pounds per cubic foot. With the above density the average total voids was 37.3 per cent.

The average density from the 13 tests on the dune sand with varying moisture contents was 104.1 pounds per cubic foot, with a total per cent voids of 37.3. The moisture contents varied from 3.5 to 13.7 per cent.

A. A. S. H. O. Test T-99: As a basis of comparison for the test conducted to this point the standard test for the compaction and density of soils as designated by the American Association of State Highway Officials was carried out on 16 of the samples of dune sand with varying moisture contents. The average density attained was 104.3 pounds per cubic foot when compacted under this specification. The total per cent voids averaged 37.16 with the moisture content averaging 7.41 per cent. The minimum density was 100.5 pounds per cubic foot with a moisture content for 1.24 per cent and a maximum density of 105.9 pounds per cubic foot at 13.02 per cent moisture.

On the test results it can be seen that the most uniform results were obtained with vibration, cone and tamping methods. They also gave appreciably greater densities. From the results of the test methods it is apparent that the method of vibration for consolidation is the one most applicable and suitable to general construction practice to achieve the maximum compactive effect with the least compactive effort. The A. A. S. H. O. test method gave comparable results with the vibratory and cone density tests. Since this test was designed primarily for use on soils of very fine gradation, it is not generally adaptable to the deposits of native granular materials, therefore not generally suited to field construction.

Consolidation of Pea Gravel

A series of tests were carried out on pea gravel having the following gradation:

Size of Sieve	Per Cent Passing
1/2"	100
3/8"	96
# 4	20
#10	2

The average loose density of the pea gravel was 94.17 pounds per cubic foot, with a total voids of 41.9 per cent and an average moisture content of 1.4 per cent. A total of 55 tests were carried out to arrive at the above data.

Inundation: The tests by ponding were carried out in the same manner as those described previously in this report.

The average density following ponding of the pea gravel was 93.3 pounds per cubic foot, the total voids averaged 42.4 per cent. These averages are not based on extensive data and are not a standards, but it can be noted the evident decrease in density with saturation and a resulting increase in total voids. Based on these six tests further tests were abandoned as it was apparent that any method of consolidation involving saturation or near saturation were ineffective and impractical.

Cone Test: Pea gravel with the same gradation as that in the preceding tests was consolidated by the cone method. The average results were as follows: Density, 107.1 pounds per cubic foot; total voids, 33.96 per cent. This is an increase of 13.7 per cent over the loose density and a decrease in total voids of 18.9 per cent.

Vibration Tests: Vibration tests were carried out on the pea gravel which resulted in an average density of 104.28 pounds per cubic foot and a total voids average of 35.7 per cent. The results of the vibration tests indicate an increase of 10.7 per cent in density and a decrease of 14.61 per cent in total voids over the loose density state.

The tests were run on the pea gravel to bring out the point of increase in the consolidation of granular materials over their attained densities when not properly compacted during placing. The resulting consolidation is caused by the traffic vibrational effect and superimposed loads of structures. It should be remembered that in foregoing tests of this study it was determined that pea gravel was not suitable for use as a granular backfill material, since the gradation of the material was of such nature that material placed over or adjacent to it infiltrated into the pea gravel medium. This was not desirable since it gradually shut off drainage through the backfill material.

The foregoing tests have proved conclusively that two factors to be considered in the discussion of granular backfill materials are gradation and consolidation. First, gradation was found to have great importance in a majority of backfill projects. There are some cases, it is true, where gradation would be of no paramount importance. An example of this would be where granular backfill materials are adjacent to native soils of clay or clay loam, where there would be little or no danger of infiltration of water from the adjacent soils. With this condition prevailing the backfill could be much coarser, and a possible material would be pea gravel. Care must be exercised with these conditions though to insure that there is a cut-off incorporated to prevent the infiltration of the surface material into the backfill material.

The second, and possibly the most important, factor is the resulting consolidation of backfill materials when they are not properly compacted during placement. This consolidation has proved to be of prime importance in bridge approaches with resulting settlements of the pavement slabs to the point of causing hazardous irregularities in the surfaces. This condition prevails in a majority of the existing bridge approaches where no controlled density was employed during the placement of the backfill material.

VOLUME CHANGE AND CONSOLIDATION METHODS

With the two factors mentioned above in mind, the next step in the study was to experiment with various means of consolidation to study settlement in backfills of various selected granular backfill materials and also the relative efficiency of each consolidation procedure.

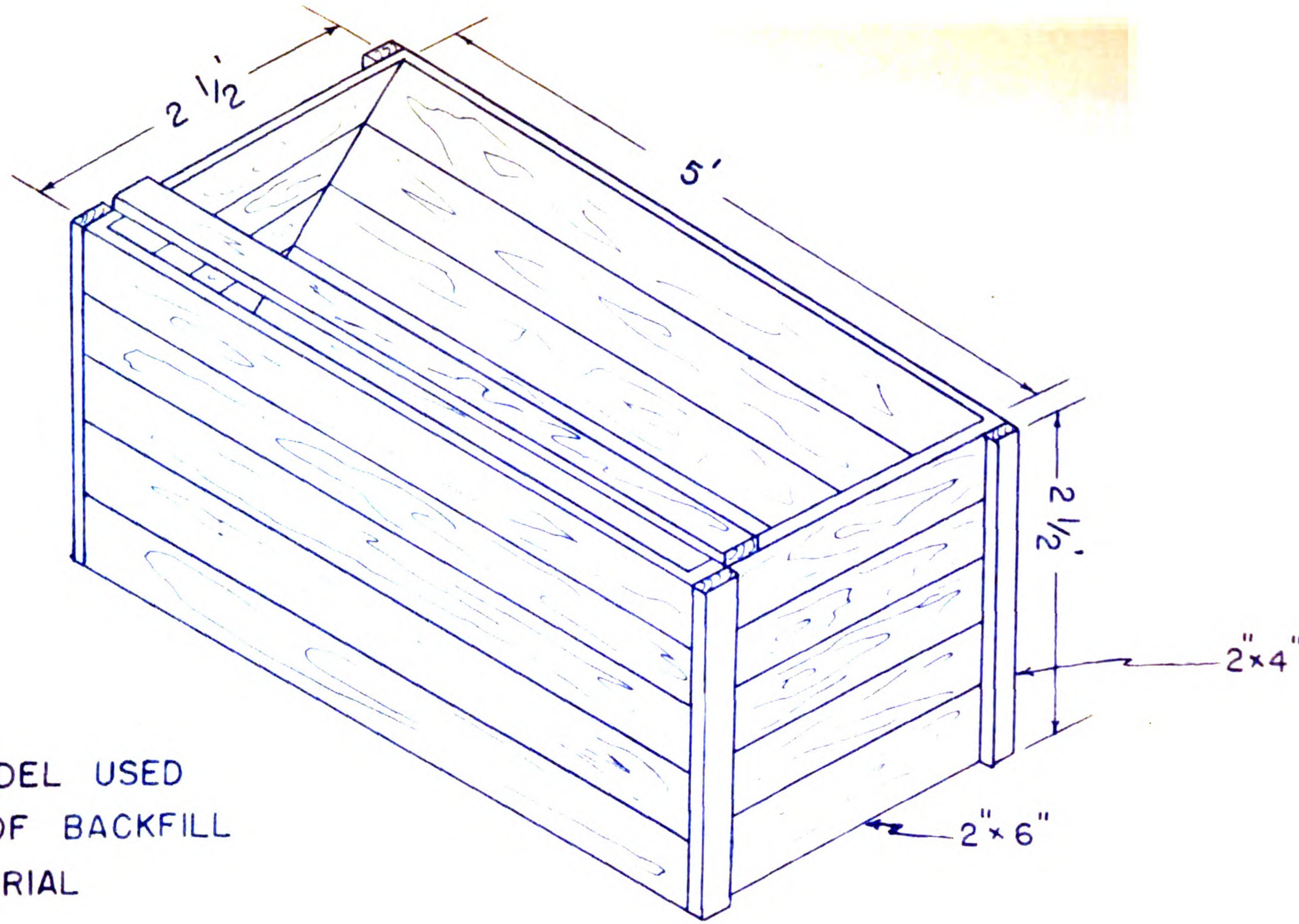
These tests were carried out on small laboratory testing equipment and were later substantiated by tests on a scale model of a bridge abutment backfill.

The preliminary laboratory tests were carried out with dune sand. First, the minimum voids were determined for dune sand when it was compacted to its maximum density. This value was found to be approximately thirty per cent. Assuming this value and comparing values obtained by vibration and tamping on a ten-foot fill, which was assumed to be an average height fill, when placed loose would have a possible settlement of twenty-nine inches. This would be a settlement equal to approximately twenty-five per cent of the original height of the fill. Tests by ponding on the ten-foot fill of dune sand would result in a possible settlement of 14.4 inches or approximately twelve per cent. With tamping and vibration employed as a means of consolidation, a possible settlement of from 0 - 7.2 inches could be expected depending on the effort exerted on these consolidation methods.

Following this series of tests to determine possible settlements following various consolidation means the operations were transferred to the bridge backfill model. This model had one side representing the back of the abutment and the opposite side was sloped to represent the back slope of the excavation. See Figure 12. The volume of the model was 15.6 cubic feet and was so constructed to permit removal of the front face, representing the abutment face, to facilitate inspection of the material following compaction. The removable front also aided in unloading and recharging of the model. The test bin was designed so that a platform scales could be used in conjunction with the test to have an absolute check on the density. Arrangements were such that the model was jacked up from the scales and lowered to the platform of the scales following the compacting stage. Care was taken to insure that no tests were carried out with the scales supporting the model. This arrangement provided a uniform and accurate method of checking density on granular materials, which has proven to be a serious problem in all such materials.

The purpose of the test with the scale model of a representative backfill was to substantiate results of the foregoing tests and to determine the following:

1. The compactive effort required to consolidate backfills of dune sand, natural sand, back run gravels, pea gravel, coarse aggregate and slag to the point where there would be no detrimental settlements.
2. The effect of moisture content on compaction of granular materials by vibration.
3. The relative density requirements for granular materials when used for structural backfill.
4. The most efficient means of attaining specified densities.



SCALE MODEL USED
IN TESTS OF BACKFILL
MATERIAL

Figure No. 12

To find the answer to these problems a number of tests were carried out. A standard procedure was followed, an outline of which follows:

1. The material to be investigated was placed in the mold loose and a weight per cubic foot was determined. Every precaution was taken to try and simulate field conditions in the way in which the soil would be placed in structural excavations. The material was allowed to slide down the incline and fill up the bin finally being struck off level with care not to compact any of the top materials. Figure 13 shows the model filled before consolidation.

2. The material was then compacted with a bullet-nosed vibrator. The vibrator was allowed to penetrate into the material and also was operated on the surface in a horizontal position. The vibrator was allowed to come in contact with the sides and back-slope of the box at times during the test, but was not allowed to operate in this manner for extended periods of time. When the vibrator did come in contact with the sides of the bin the entire model was set into vibration and would undoubtedly have a marked effect upon the results. The importance of this point is brought out later on the tests employing the larger outdoor test model. Following the vibrational period, the volume change was measured as shown in Figure 15. A number of measurements were taken on each sample from a level across the top with an average settlement figure finally determined. A weight was also taken with the

Figure No. 13. Model filled with coarse aggregate in loose state.

Figure No. 14. The bullet-nosed vibrator employed on the model tests.

scales to determine the density of the material following the consolidation. This density could be calculated by the known volume and the determined weight.

Figure No. 15. Checking settlement after consolidation.

3. The material was then removed from the model by removing the face from the model. The material was again placed in the bin with the vibrator operating continually during the filling operation. Care was exercised not to charge the model too rapidly not allowing the vibrator time to effectively consolidate the material adjacent to it. Density measurements were again taken

but no settlement data could be taken with this procedure, or phase of the test.

4. The last step in the model study tests was to place the material in the model in 4 to 6 inch layers and hand tamp each layer with the standard construction tamper which weighed 25 pounds and had a surface approximately eight inches square.

A record was kept on moisture contents and relative densities on all tests. Following each vibration or tamping test the material was flooded in the model for a period of 20 minutes to determine relative drainability of each material and test. After a 24-hour drainage period, moisture samples were taken and also check density measurements. The materials were then emptied and allowed to air dry before being used in another test.

Detailed reports on all scale model data can be found in Appendix C of the report.

The conclusions arrived upon from the series of scale model tests are as follows:

1. No material can be placed without some means of compactive effort that will not result in detrimental settlements.
2. Sand and bank run gravels will yield the greater densities with the minimum of compactive effort. The coarser the individual particles, the greater is the required compactive effort.
3. Moisture content has no apparent effect on vibratory consolidation, while with tamping a marked effect was

noted. This same observation was noted earlier in the study for the inundation methods for consolidation.

4. Tamping can be used effectively on any granular material with resulting maximum density, but close control must be maintained to keep the layers being consolidated to six inches or less.

5. Vibration will yield results comparable, and in some instances better, than tamping for consolidation of well graded granular materials or those which have uniform size and are well rounded particles.

Figure No. 16. Front face removed showing stability after compaction.

Vibration is advisable on all such soils since the same final results as tamping will be gained with far less effort and control.

6. When tamping is to be employed as the consolidation means, the moisture must be maintained at or near the optimum as determined by the standard A.A.S.H.O. test T-99.

All of the foregoing tests tend to confirm the above conclusions, but there is too little information available to arrive at any definite conclusions or recommendations as to the most efficient method of consolidating backfill materials on field projects under construction conditions. It has, however, been determined and demonstrated that some means of consolidation is needed to eliminate settlements in backfill materials regardless of the type of material being used. The method of hand tamping or even mechanical tamping would and has proven tedious in field practice. The use of bullet-nosed vibrators would also prove to be a slow and tedious means, but would be an advantage over the tamping. With the results of the previous tests and with the above considerations in mind, a means was sought to apply the vibrational consolidation to field construction projects and practices.

The next series of tests carried out were at Scottville, Michigan, where both field vibrational equipment and facilities were available for tests of the scale planned. The objectives of these tests were:

1. To study various vibratory apparatus with view toward a means of consolidating structural backfill materials to avoid detrimental settlements.
2. To determine effective vibratory range and establish a maximum lift thickness for placing materials.
3. Study effects of frequency variations on consolidation for various types of vibrators.

4. To study lateral pressures as related to consolidation by vibratory means.

5. To study various types of vibrators with correlation of effective vibrating time to degree of consolidation.

The granular material on which all of the tests were carried out was a bank run gravel from near Scottville, Michigan. The physical properties of this material are included in Appendix D.

The material was confined in an outdoor form 16 feet long, 10 feet wide and approximately 6 feet deep. The material was employed in the bin in layers as indicated in the following data. The box was constructed of 3-inch planking, well braced to avoid any bulging of the side walls so that approximations could be determined as to volume loss on various

Figure No. 17. Charging the field bin with mechanical equipment.

tests. The box had a natural ground floor to approximate actual conditions as nearly as possible that would exist on any backfill project.

Three Goldbeck Pressure Cells were installed in the bin to measure variance in lateral pressures caused by vibratory consolidation methods. All of the cells were located on one face; one in the lower corner, one on the lower center face, and the third at the intersection of the diagonals of the wall.

The following is a detailed description of the tests run and equipment employed on the project. Tables included in Appendix D indicate densities and moisture contents as recorded on all tests.

The power for the various types of electrical vibrators and mechanical tampers which were used was furnished by a standard mobile power unit

Figure No. 18. Portable power unit for electrical vibrators.

shown in Figure 18. The unit is portable, being mounted on a welded steel wheelbarrow base equipped with a pneumatic wheel. The generator is rated at 2 K.V.A., 110 volts, 3 phase, 60 cycle AC, 1920 R.P.M. at 60 cycles. The generator has a safe operating frequency range from 50 to 80 cycles, with a synchronous motor speed of 3,000 to 4,800 rpm. Maximum 3 phase load of 10.5 amps. It is powered by a self-excited and V-belt driven 5 to 6 HP air-cooled engine.

After the experimental gravel bin had been filled level full with the drag line, the first experiments tried were with a standard PT-21A heavy duty paving tube motor to which 6' - 2-1/2" - 1/8" angle irons were clamped at one end to the motor brackets, the other end being pointed or spear shaped. Figures 19 and 20.

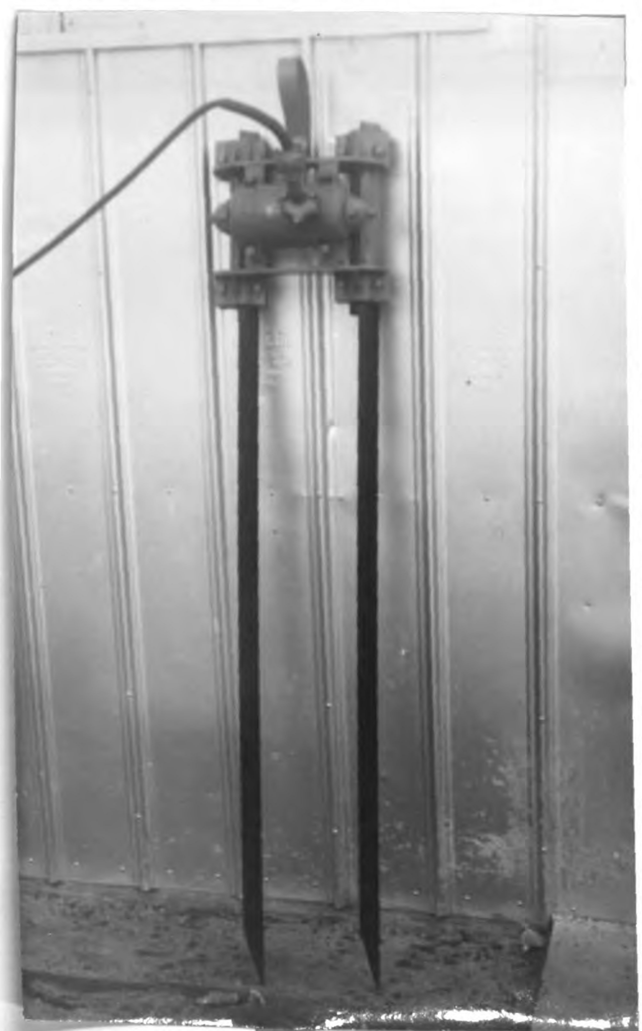


Figure 19. Penetration type vibrators.

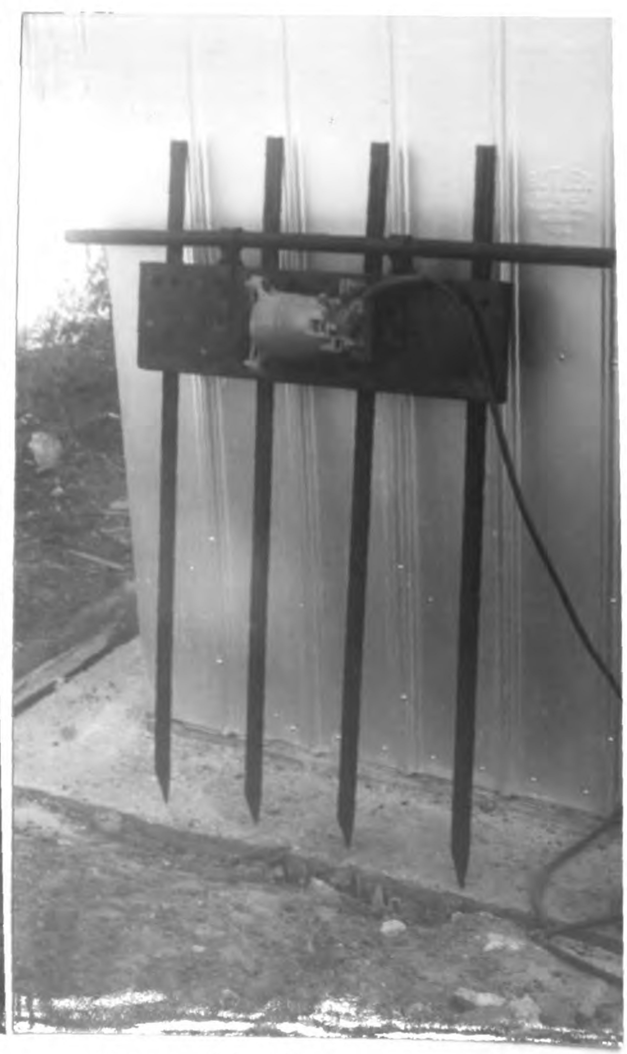


Figure 20. Spear type vibrator.

The paving type vibrator motor is a 110 volt, 3 phase motor which operates at approximately 3,600 vpm at 60 cycles.

The drag line crane lifted the vibrators of a vertical position over the bin by means of a loop strapped to the motor bracket. As the machine was started, the drag line slacked off allowing the spears of angle iron to penetrate into the material. No trouble was experienced in getting the spears to penetrate to their full depth as they were lowered. Vibrations could be felt to a slight degree at least three feet from the vibrator, but there was no visible settlement of the material surrounding the vibrator. The vibratory equipment was allowed to run in this lowered position for two or three minutes and then slowly withdrawn with no visible settlement of the adjacent materials. Densities which were taken in the area where the machine operated showed very slight increase over the loose density for the material.

The next test was carried out with a flexible shaft concrete vibrator, similar to the type used on the indoor bin tests. Figure 14. This machine was powered with a 4 HP air-cooled engine. The vibrating head used was 2-3/8 inches in diameter and 18-5/8 inches long. The head is capable of operating up to 7,500 vpm. The vibrator was inserted in the material at various locations. No trouble was experienced in effecting the tube to work down in the material. While considerable vibrations were felt throughout the area adjacent to the machine, very little settlement of the material was evident. The material displaced by the head, as it was lowered, was compacted in the area immediately surrounding the head and left a hole when it was withdrawn. Subsequent insertions and withdrawals of the vibrator, while varying the frequency of vibration of the equipment to its maximum and minimum limits, did not alter this condition. Because of the voids developed with the equipment upon withdrawal, further tests with it were abandoned.

A large bullet-nosed vibrator was tried in the next test. A view of it can be seen in Figure 21. This vibrator is a completely enclosed motor with a double unbalanced vibratory element mounted on the rotor shaft. The motor is a 110 volt, and 3 phase as before. The full load speed at 60 cycles ranges from 3.450 to 5.100 vpm. Horsepower impact rating for this motor and equipment is 1.5 HP. The vibrator head is 18.5 inches by 6 inches in diameter. This vibrator was not allowed to work itself to far into the material since too much of the material would have been displaced which would have stressed the rubber coupling connecting the vibrating head to the motor to failure upon trying to withdraw it. The machine was initially designed for work in vibrating concrete with a high range of workability.

In the next test the paving tube actuating motor was clamped by means of the paving tube motor brackets to the ends of two sections of 2-3/8 inch tubing which were 8 feet long. Figure 22. This tubing had a 3/8 inch wall section and is used for the vibrating members of the standard paving tube. This equipment was lifted to a vertical position employing the crane. The tubes were allowed to penetrate to the full depth of 6 feet. There was no visible settlement of the level of the material in the bin although vibration could be felt for some distance from the vibrator. Density tests taken indicated a high degree of compaction in the area immediately around the tube positions, but very little increase in adjacent areas. As in the case of the flexible shaft vibrator, voids were left upon withdrawal of the vibrator. Frequency variations and time increment increases did not change the effects.

Following the tests with the penetration type vibrators, which proved to be ineffective on the large volume, further such tests were abandoned with efforts turned to the experimentation and development of a surface type vibrator which would impart a force to the layers of fill material and effectively consolidate it.

Figure 21. Large bullet-nosed vibrator.

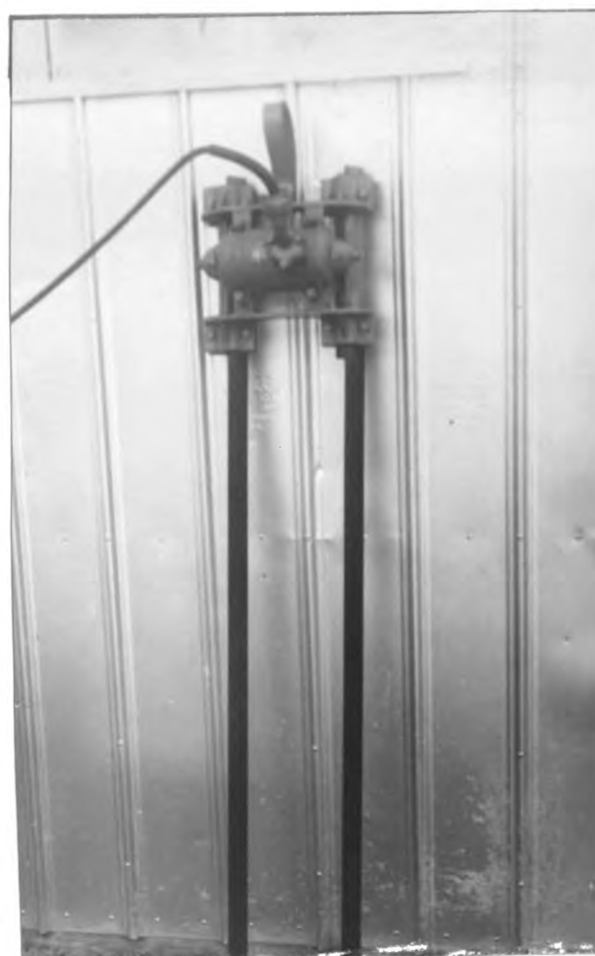


Figure 22. Penetrating vibrator.

The first surface vibrator experimented with was a tamping type. A view of this is in Figure 23. It is operated by a 110 volt, 3 phase, 60 cycle A.C. motor with a striking member forged integrally with rotor shaft. This member delivers blows to the tappet, which in turn works on a tamping bar to which a 10-inch conical tamping disc had been attached. This machine was manually operated over a small area in one corner of the bin. There was a marked settlement of the material evident in the area immediately under where the machine was employed. A density test indicated good compaction in the immediate area. Following this a platform vibrator, Figure 24, was tried and found to be very effective. Although manually operated, a large area could be effectively covered.

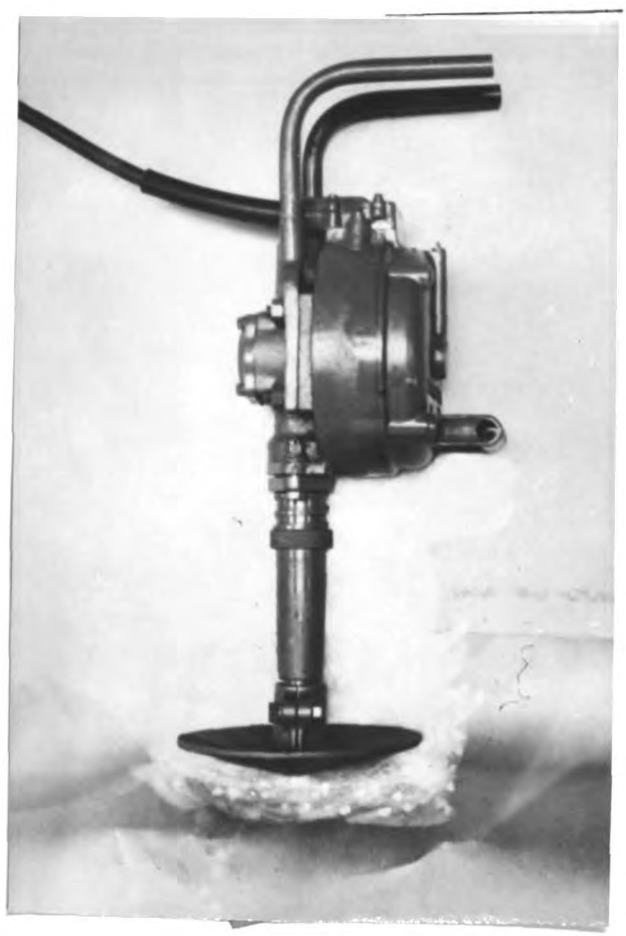


Figure No. 23. Surface tamper.

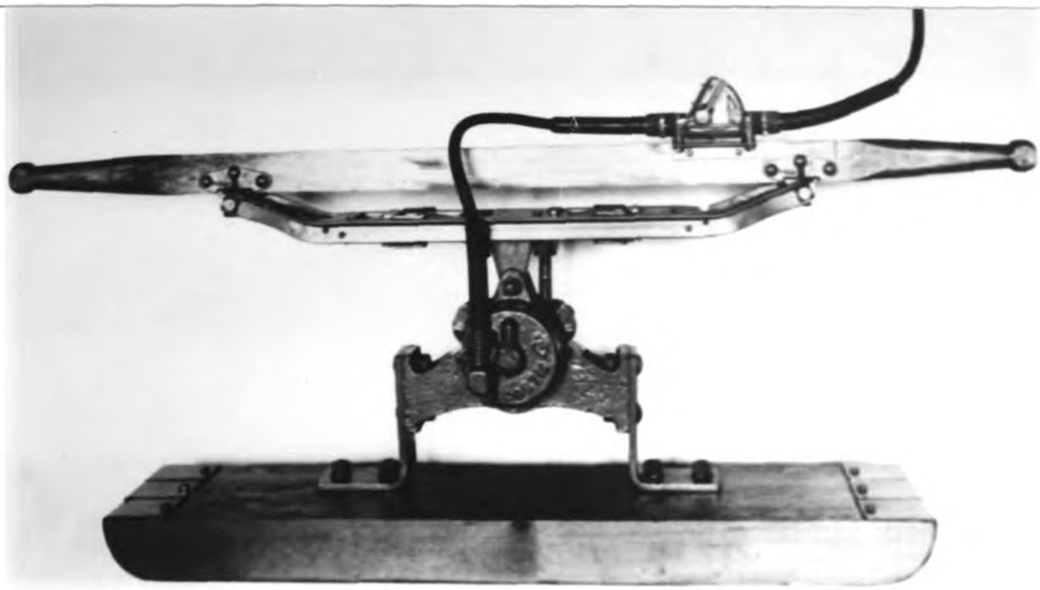


Figure No. 24. Platform vibrator.

The results with the tamper and platform vibrator encouraged the construction of a larger platform vibrator shown in Figure 25.

Figure No. 25. Modified platform type vibrator.

The standard paving type vibrator motor was mounted on a seven foot length of twelve-inch channel iron, modified type platform vibrator which was mechanically handled. The motor was bolted to the channel in a crosswise position at the center of the channel length. After the bin had been emptied and refilled to a level of two feet, the machine was put in one end of the bin and pulled over the surface of the material by the crane. A cable bail had been attached to the channel for the purpose of handling it. Skids of 3/16 inch steel, four inches wide, were attached to the under side of the channel to aid in keeping it from tipping as it was being pulled. The skids projected about six inches from the face of the channel and were slightly curved up at the ends to avoid digging in.

This equipment gave very satisfactory results when operated at frequencies approximating 4,500 vpm.

The next type vibrator to be tried was made up of two 8-foot lengths of paving tubes with the vibrator motor centered on the two limbs. This vibrator was placed in the bin in a horizontal position with a layer of gravel two feet deep placed over it. A view of this vibrator is shown in Figure 26. There was an immediate settlement of about two inches in the material in the area of the bin above the vibrator. Vibration was continued for about three minutes but no further settlement was evident. The vibrator was then slowly lifted by the crane and the sling that had been attached to the equipment. After the vibrator was lifted through the material, vibration continued with the tubes on the surface gave evidence of little if any additional settlements. The equipment was operated at approximately 4,000 vpm for the entire test. Density tests indicated good degrees of consolidation.

Figure No. 26. Paving tube vibrator.

The bin was now emptied and refilled to the top at one end with the backfill material sloping to the ground level at the open end of the bin. The vibrator shown in Figure 26 was then placed on top of the backfill material. As the machine was started the drag line pulled it down the slope gradually in an attempt to move the material to fill in against the planks on the open end of the bin. The tubes were found to have a tendency to float on the surface of the material, thereby losing the vibrational effect.

An attempt was made to force the vibrator to travel deeper by attaching four pieces of 2-1/2 - 2-1/2 - 1/8 inch angle iron to the forward edge of the tubes as shown in Figure 27. The fins were equally

Figure No. 27. Improved paving tube vibrator.

spaced and so attached that one side of the angle iron extended down from the bottom of the tube. The machine was then employed as explained previously.

Four passes of the machine were made, Figure 28, at a speed of approximately 4,000 vpm with the speed of drag approximately 13 feet per minute.

The modified model and the method employed seemed to be very effective on large quantities of material. The addition of the angle iron fins caused it to dig in and effectively apply the vibrator.

A summary of all outdoor bin tests follows:

1. Bullet-nosed vibrators were found to be very inefficient since they only proved to be effective over a very small area and tended to loosen the material when they were withdrawn. It was too heavy for manual handling but did not warrant mechanical handling.

2. Prong type or penetration type vibrators, both the plank with spears and the vertical tube vibrator, were not satisfactory since they had a very limited consolidation range and left holes in the material when they were withdrawn. With the tubes or spears sixteen inches apart, there was an unconsolidated zone between them. Both types require mechanical handling due to their weight. No settlements were observed

Figure No. 28. Paving tube vibrator in operation.

on the bulk of material with either type.

3. The puddler, both the standard type and the modified type, gave good consolidation on the surface and to a limited depth. Either could be handled manually and should be employed on very thin layers,

six to eight inches, and on small scale backfill operations.

4. The vibratory tamper had a very limited range in its compactive effectiveness. It gave good results on the surface but could not effectively set the large mass in motion to settle the material other than that consolidated by the striking force and that in a local area under the tamping shoe.

5. The horizontal tube vibrator, as employed in paving operations, proved the most effective and efficient type. With the tubes being pulled over the surface the material was consolidated to a slight extent and very little settlement resulted. Much of the vibration effect was lost and the material was compacted on the surface only.

With the tubes buried in the material and allowed to vibrate in this position, very good results were obtained and the maximum degree of settlement of all tests resulted. The vibrator was slowly lifted from the material allowing time following each lifting for the vibrator to consolidate the material filling the void left by the tubes and motor. No voids were left in the fill material when the vibrator was lifted to the surface. The consolidation was uniform for the entire depth. The area affected was the entire width of the bin and approximately five feet wide. This type of vibrator could be of varied lengths and made up in sections twelve feet long, while the vibrator under test was a special designed section eight feet in length.

When employing the horizontal tube vibrator as a drag to haul the material down the inclined slope and compacting it against one end of the bin, it proved efficient and adaptable to backfilling operations. Employed in this manner, it served a dual purpose in spreading the material and consolidating it in one operation. As the density results indicate, each additional pass tends to further consolidate one to two feet down. This action would be very favorable and reduce any tendency toward surface fluffing of the material.

6. The pressure cells were not too efficient since the material was compressible enough that when the pressure was applied to the cell it would force the material away from the face of the cell and obtain a pressure reading. On releasing the pressure from the gauge and the cell, the cell piston would return to its original position leaving a void on the face of the cell when the material did not rebound. The failure of the material to rebound would prevent any further readings with the cells with additional manipulation of the mass of material. Under this condition reliable readings could not be obtained in all instances; however, sufficient readings were obtained to qualify that no additional or excessive lateral pressures were a resultant of the vibratory equipment employed.

7. The frequency of vibration was varied on all tests and it was found that best results were obtained in all cases with a frequency varying between 4,000 vpm and 4,200 vpm. When higher frequencies were used the same results were obtained with no apparent advantage. Lower frequencies tended to slow the consolidation since the effectiveness of the vibrator seemed to be in getting the entire mass of material in movement at one time, and low frequencies would not satisfy this condition. Each type of vibrator has a natural frequency and when this is maintained the best results were obtained. Higher than natural frequency also tends to add load on the motor and damage it with continued use.

The above observations clearly demonstrate the effectiveness of the horizontal tube vibrator or paving tube vibrator as it is commonly known. For work in excavations and around footings of abutments the manually operated platform vibrator seemed to offer the greater possibilities.

No further study was carried out on the paving tube vibrator for field use, but the platform vibrator was modified and was employed on a field construction project west of St. Ignace, on US-2. The work consisted of constructing a metal crib retaining wall shown in Figure 29. Each crib or cell was rectangular in shape with dimensions of 10 by 10 by 20 feet. The crib was built up in sections and the backfill was placed in each crib simultaneously with the crib assembly.

Figure No. 29. Cut River metal crib.

Controlled density was specified for the backfilling operation in the cribs. Since the contractor was using tamping to consolidate the material it afforded an excellent opportunity to compare vibratory means of consolidation with hand tamped results.

The platform vibrator as it was modified for the work is shown in Figure 30. Additional base plates that were also employed in this study at Cut River are shown in Figures 31, 32 and 33. The modified platform vibrator with the base plate shown in Figure 32 proved to be the most effective.

Figure No. 30. Modified platform vibrator.

The modified platform vibrator was found to travel under its own power, due to the vibrational motor, at a rate of approximately ten feet per minute. This eases the problem of handling and manipulation to a great extent.

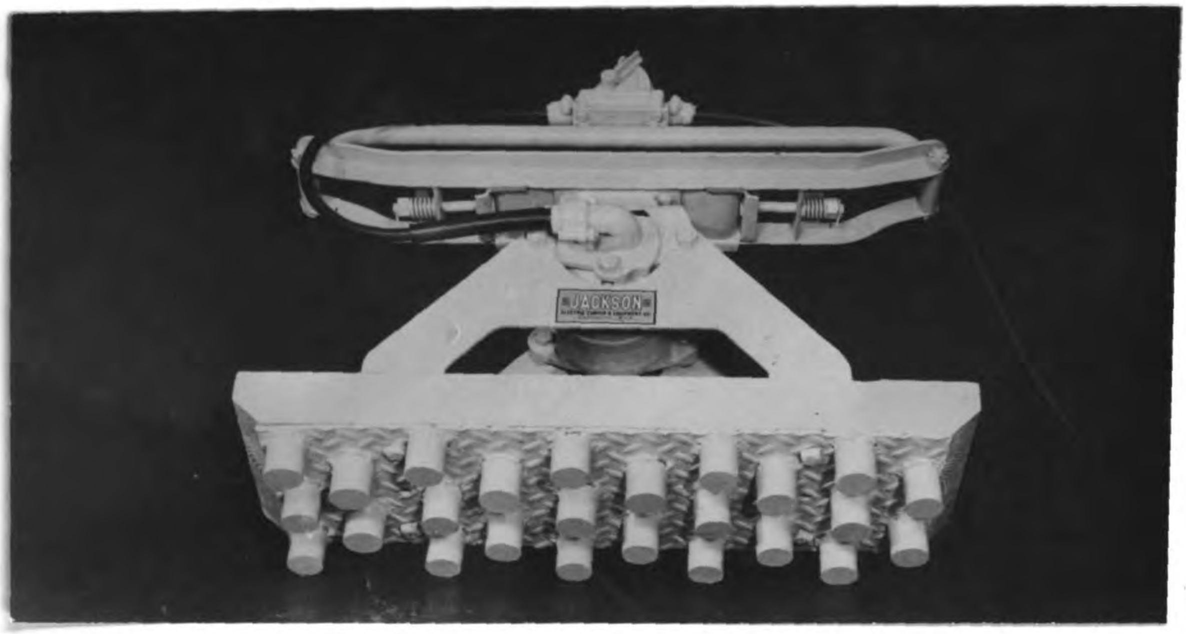


Figure No. 31. Platform vibrator with sheepsfoot base plate.

The material used to fill the cribs was dune sand and was placed in the cribs in layers of approximately nine inches. To effectively cover the 100 square feet of area with the tamper it took the contractor's equipment about fifteen to twenty minutes. With the vibratory equipment the same results were attained in approximately three to four minutes. Detailed results of the study can be found in Appendix D.

The platform vibrator was also used on two other bridge abutment projects in Michigan. The vibrator model was modified slightly for ease of handling, but the platform principle and plates were the same as used before. The modified units are shown in Figures 32 and 33.

Figure 32. Improved platform vibrator.

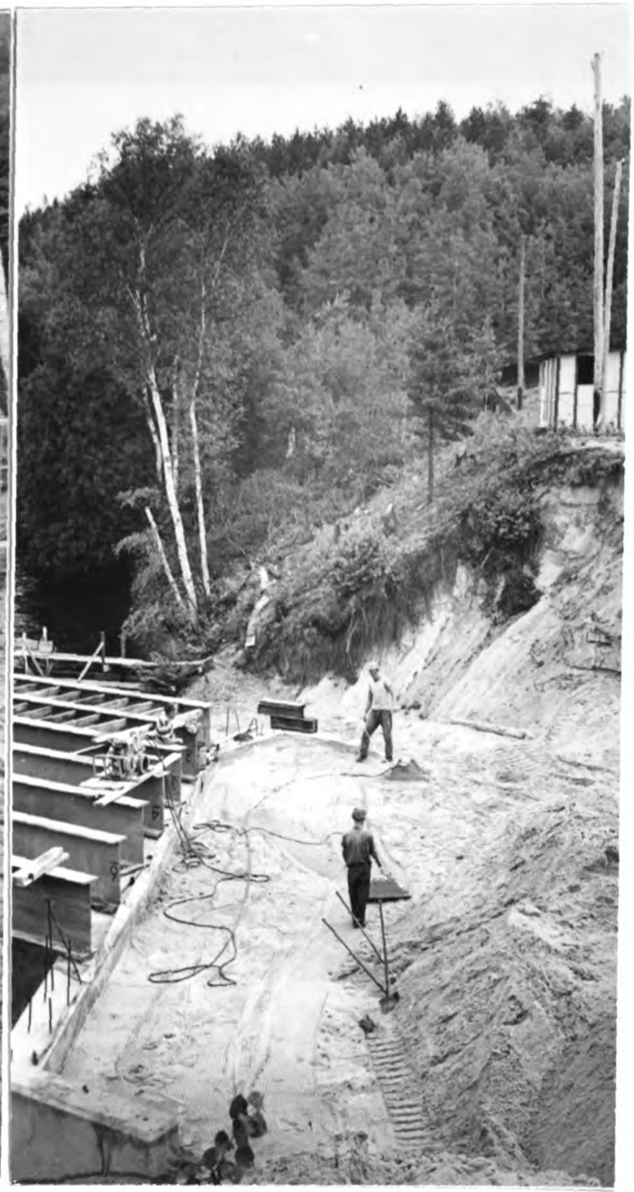


Figure 33. Improved platform vibrator.

With the modified platform vibrator, described above, there is an effective radius of approximately twenty-five feet when it is operated on a sand material. This conclusion was reached after observations of the motion of the sand surface while the vibrator was in operation. It can also be verified by readings with a reed tachometer placed at various locations in the area surrounding the vibrational unit.

The most effective frequency of vibration for the granular materials experimented with on the field projects was found to range from 2700 to 4000 vpm. With the possibility of encountering materials that would fall out of this effective range, further study is now in progress to determine vibrational frequencies for varying materials.

With the equipment used in the field tests to date it was found that layers of granular materials up to three feet in thickness could be effectively compacted. However, with layers of this depth, the duration of the vibration time was considerable to fully consolidate the material. This study has proven that layers of approximately 12 inches can be compacted most efficiently by vibrational mediums.

On the majority of the tests one pass of the vibrator was sufficient to obtain densities of 95 per cent of the maximum density or even higher in some instances. Time density studies have shown that 95 per cent of maximum density can be attained in approximately 13 seconds. Additional vibration will obtain higher density at a very slow rate, but it is believed that densities equal to 95 per cent of the "Cone" or Vibrometer densities are satisfactory. No detrimental settlements will occur in granular backfills or embankments when compacted to this degree.

Density determinations in the field tests were made by several varying means including calibrated sand, heavy oil, mold of known volume, and rubber membrane density apparatus. The rubber membrane apparatus method was adopted as the most satisfactory under average field conditions.

Although there is a definite need for additional study on vibrational equipment and additional techniques in their operation, it has been definitely proven from this study that to obtain satisfactory densities and thereby eliminate the present-day settlements in backfills or embankments there must necessarily be some sacrifice in the speed of placing granular materials. Some definite conclusions have been arrived at from the foregoing studies and are outlined briefly in the following paragraphs.

CONCLUSIONS

The gradation of backfill materials was found to be a very important factor in any location or materials where there is any possibility of infiltration of adjacent material into or through the backfill. If there is no possibility of any infiltration, gradation is not of major importance, but will also facilitate in consolidation. With backfill materials in adjacent materials of clay, which would be impervious to water moving through it, gradation control would not be required. However, if a granular subbase is intended over the granular backfill material, some precaution should be taken to prevent any infiltration of the subbase fines down into the backfill material.

The necessity of consolidation for granular materials was definitely answered by this study from the findings that volume changes as great as 19 per cent by volume could and did occur in granular backfills which were placed without consolidation means. The amount of volume change varies with types of material and consolidation mediums, but all granular materials unless properly consolidated under the vibrational effects of traffic and weathering thereby result in detrimental settlements which will disrupt the surface treatment.

Conclusions as to the most efficient means of consolidating granular materials derived from the study are:

1. Satisfactory consolidation can be obtained by hand tamping or mechanical tamping if layers are restricted to six inches or less. This method proved very slow and expensive.

2. Penetration type vibrators; i.e. bullet-nosed concrete vibrators, vibrating spears and tines, did not prove efficient mainly because the vibrations are localized in a very small zone.

3. Surface type vibrators; i.e. modified paving type vibrators and self propelled platform vibrators, proved to be the most effective and efficient type. The platform type vibrator will give satisfactory densities on layers of granular materials up to 12 inches in thickness. The paving tube vibrator would be adaptable for large unconfined areas such as deep bridge approaches, embankments, and pavement subbases. Platform vibrators are adaptable to areas inaccessible to larger equipment. Modifications of the platform vibrator for use on larger areas would be to employ it as a "gang", made up of three or more units.

A controlling conclusion is the cost of placing backfill materials when employing vibrational equipment is less than that when placed in accordance with the requirements of current standard specifications.

APPENDIX A

DUNE SAND DENSITY STUDY
Bridgman Sand

Loose densities with varying moisture content.

Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids	Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids
1	0.09	93.50	43.7	9	12.2	75.63	54.3
2	1.13	78.00	53.0	10	15.6	73.34	55.8
3	2.37	76.97	53.9	11	17.6	76.70	53.8
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				

Loose densities and densities after inundation.
Water introduced from top only.

Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids	Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids
1	0.00	93.54	43.6		0.00	93.63	43.6
	23.75	96.06	43.5		22.58	98.14	40.9
	25.60	96.06	43.5	8	1.12	77.08	53.6
	26.50	96.34	43.5		27.70	88.65	46.6
2	0.00	94.04	43.4	9	0.00	94.97	42.8
	22.12	96.34	42.0		23.51	97.94	41.0
3	0.00	93.59	43.6	10	1.23	75.77	54.4
	23.04	96.34	42.0		24.16	86.10	48.1
4	0.00	93.59	43.6	11	1.04	77.06	53.6
	22.90	96.34	42.0		23.26	87.86	47.1
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				

Loose densities and densities after indirect inundation.
Water introduced from sides of model into the sand. A load of 2-1/8 pounds per square inch was maintained on the surface of sand.

Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids	Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent 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

Dune sand densities as determined by the cone method.*

Test No.	Per Cent Moisture	Density Lbs/Cu.Ft. (Dry)	Per Cent Voids	No.of Blows Per Layer	Test No.	Per Cent Moisture	Density Lbs/Cu.Ft. (Dry)	Per Cent Voids	No.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	3.60	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 B.

Dune sand, densities loose and densities as determined by different degrees of vibration on the vibrating table.

Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Type of Vibration	Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Type of Vibration
1	0.00	93.70	4 Minutes Mild	5	0.00	93.70	4 Minutes Hard
		100.00				106.44	
2	3.00	71.12	4 Minutes Mild	6	3.09	71.35	4 Minutes Mild
		96.00				99.01	4 Minutes Hard
						106.41	
3	5.42	70.49	4 Minutes Mild	7	5.12	71.99	4 Minutes Mild
		96.59				97.19	4 Minutes Hard
						102.30	
4	7.66	71.77	4 Minutes Mild				
		96.28					

Dune sand densities as determined by the Standard A.A.S.H.O. Test T-99

Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids	Test No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent 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

PEA GRAVEL DENSITY STUDY

Loose densities with varying moisture content.

No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids	Remarks
1	0.0	94.54	41.7	Average of 10 tests
2	0.0	94.54	41.7	" " 11 "
3	0.5	96.29	40.7	" " 2 "
4	0.5	96.29	40.7	" " 2 "
5	1.0	95.06	41.4	" " 10 "
6	1.0	95.71	41.0	Max.Den." 10 "
7	1.0	95.71	41.0	Average " 4 "
8	1.0	95.07	41.4	" " 6 "
9	1.1	94.43	41.7	Min.Den." 10 "
10	1.1	94.45	41.7	One Test
11	1.7	95.44	41.2	Average " 2 "
12	1.7	95.44	41.2	" " 2 "
13	1.9	93.24	42.5	" " 2 "
14	2.2	92.50	42.9	One Test
15	3.0	93.08	42.6	Average of 3 "
16	3.0	93.08	42.6	" " 3 "
17	3.8	90.16	44.4	" " 3 "
18	4.5	90.00	44.5	" " 6 "

Pea gravel densities by inundation.

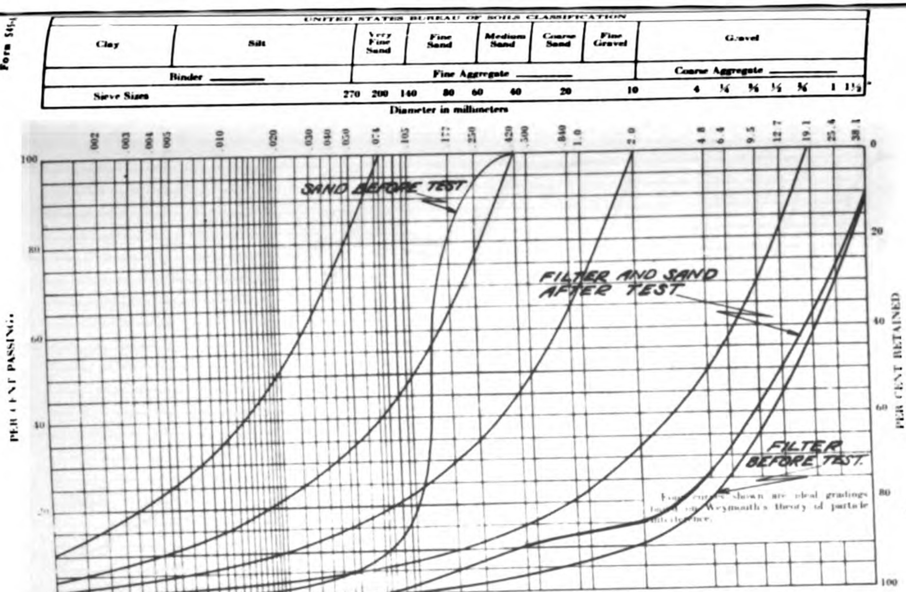
No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids	Remarks
1	12.44	95.0	41.4	Indirect ponding
2	12.50	95.5	41.1	Direct ponding over surface
3	4.50	90.0	44.5	Saturated and drained
4	3.80	90.2	44.4	" " "
5	1.90	93.2	42.5	" " "

Pea gravel densities by vibration.

No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids	Remarks
1	12.38	101.7	37.2	Mild vibration
2	12.44	101.4	37.4	" "
3	9.70	106.6	34.3	Mild vibration for 3 minutes
4	10.07	104.8	35.4	" " " 2 "
5	10.14	105.2	35.2	" " " 4 "
6	9.63	104.9	35.3	" " " 3 "
7	10.88	103.3	36.3	" " " 2 "

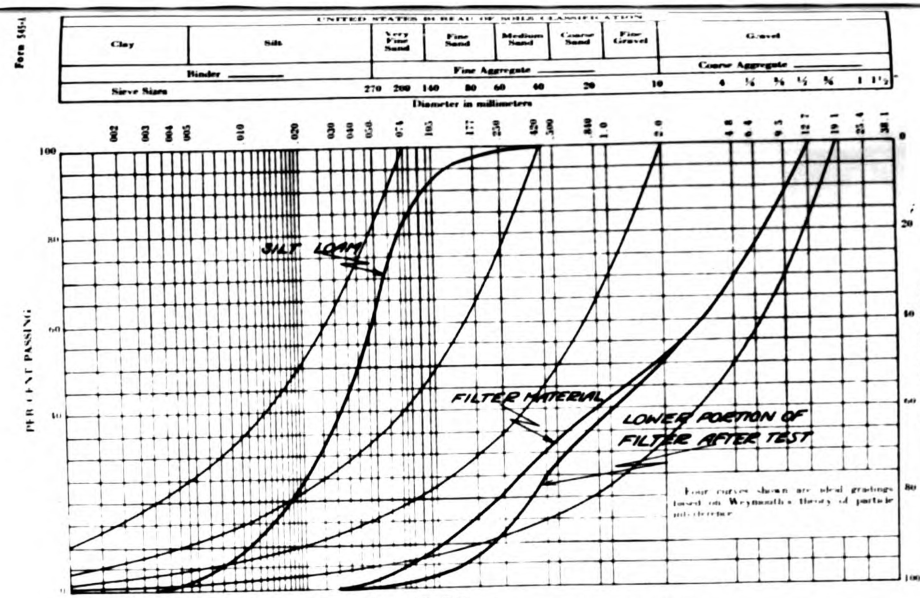
Pea gravel densities by the Cone method*

No.	Per Cent Moisture	Density Lbs./Cu.Ft. (Dry)	Per Cent Voids	Remarks
1	Dry	106.2	34.5	All samples consolidated in three layers with 20 blows per layer.
2	"	108.2	33.3	
3	"	107.7	33.6	
4	"	106.5	34.3	
5	"	106.8	34.1	
6	"	107.1	34.0	



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

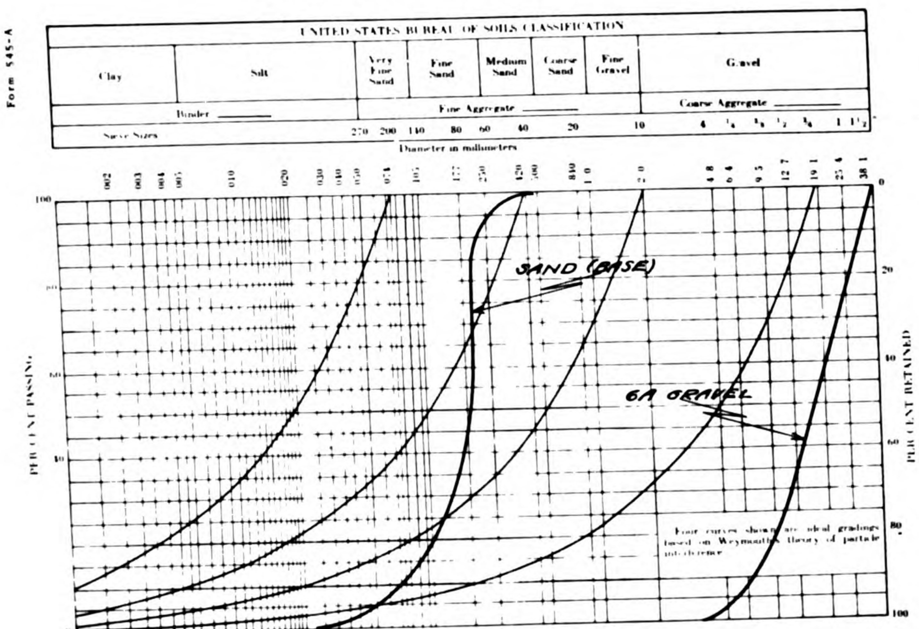
GRADATION TEST No 1.



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

GRADATION TEST No 2

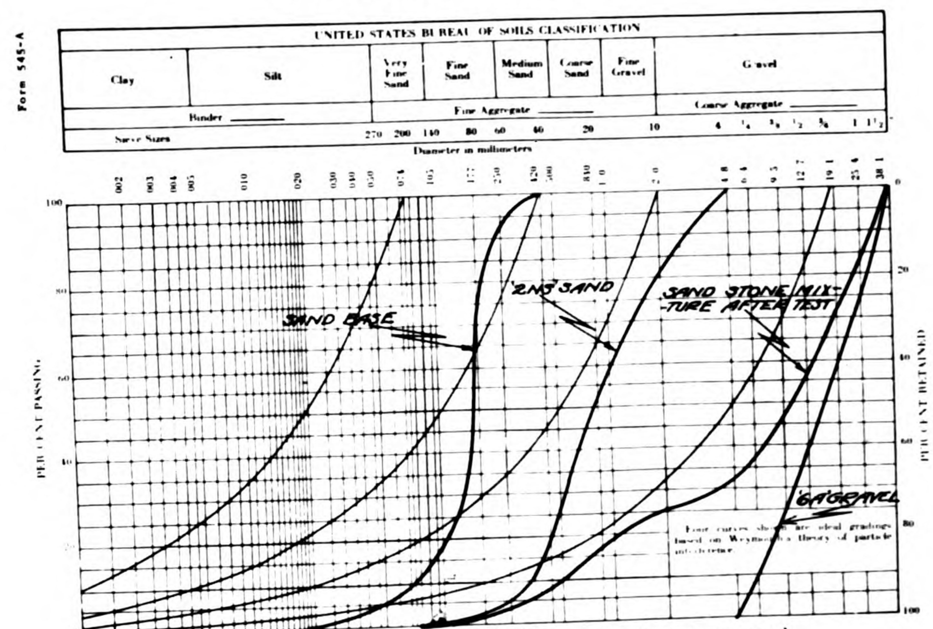
GRADATION TEST NO. 3



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

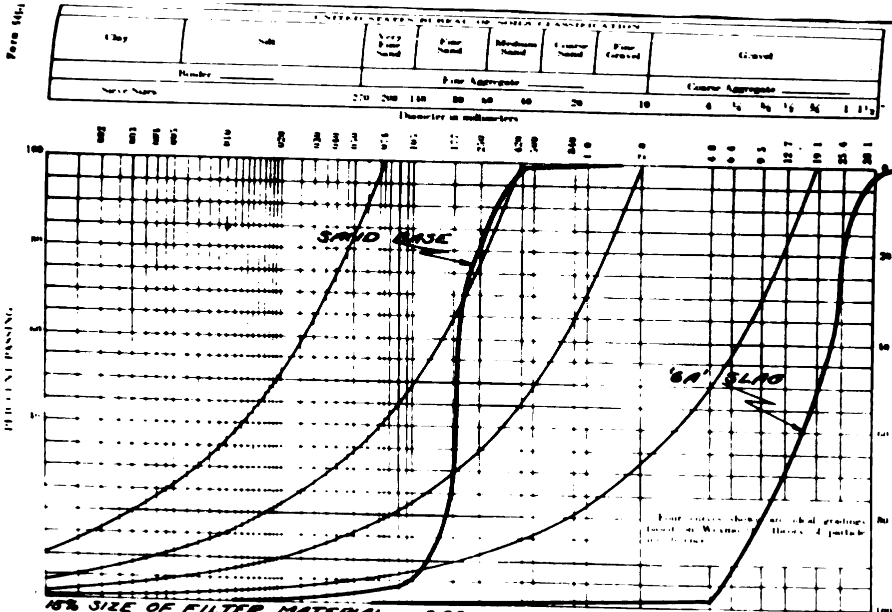
$$\frac{\text{15\% SIZE OF FILTER MATERIAL}}{\text{85\% SIZE OF BASE}} = \frac{8.4}{0.20} = 42.0 \text{ (UNSTABLE)}$$

GRADATION TEST NO. 4



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

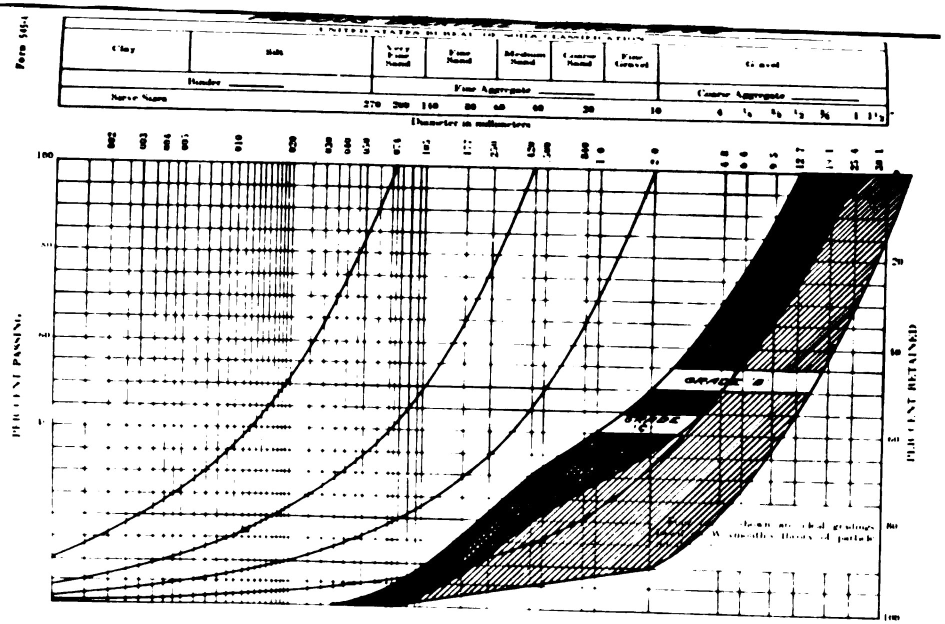
$$\frac{\text{15\% SIZE OF FILTER MATERIAL}}{\text{85\% SIZE OF BASE MATERIAL}} = \frac{0.812}{0.210} = 3.87 \text{ (STABLE)}$$



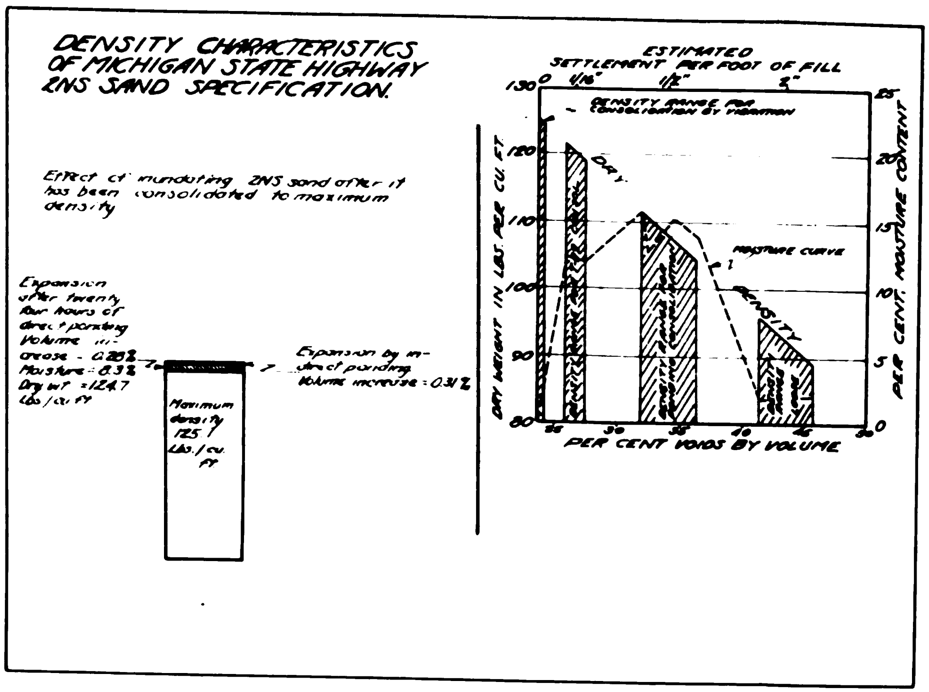
$$\frac{\text{15\% SIZE OF FILTER MATERIAL}}{\text{15\% SIZE OF BASE MATERIAL}} = \frac{8.00}{0.159} = 50.3 \quad \text{(PERMEABLE)}$$

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

GRADATION TEST No. 5



POROUS BACKFILL GRADE 'B' & 'C'



APPENDIX B

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 5-3/4 inch in diameter and the small end shall be not less than 2-1/2 inches. The mold shall be approximately 8-1/2 inches in height and shall have a volume of approximately 1300 cubic centimeters or 0.0459 cubic feet. (See Figure A.)

(b) Balances - A balance or scale of 6000-g. (12 pound) capacity sensitive to 1.0-g or 0.01 pound; and a balance sensitive to 0.10-g.

(c) Drying - Any approved method for drying soil samples.

PROCEDURE

3. (a) A 3500-g 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 the 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 means of the following formulae:

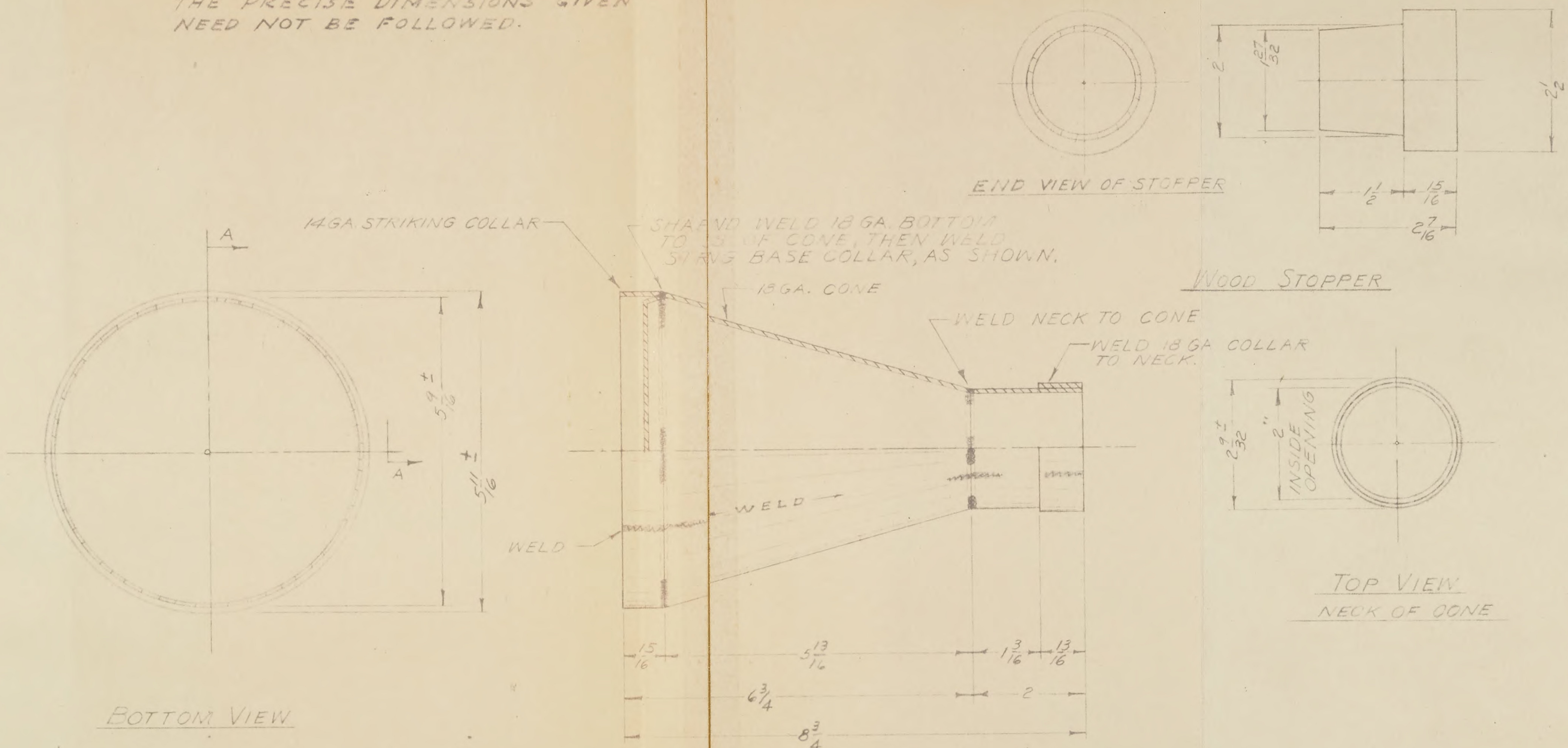
$$\text{Moisture, per cent} = \frac{A-B}{B-C} \times 100$$

where -

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

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

SHOP NOTE:
IN CONSTRUCTING CONES, THE PRECISE DIMENSIONS GIVEN NEED NOT BE FOLLOWED.



SIDE VIEW AND
HALF SECTION A-A

DENSITY CONE

M RIAL: 14 GAGE GALV. SHEET STEEL
STRIKING COLLAR, 18 GA. FOR
ANCE OF CONE.

NOTE: ALL GAGES U.S. STD.
ALL WELDS CONTINUOUS & WATER TIGHT.

MICH. STATE HIGHWAY DEPT. STATE HIGHWAY LAB. ANN ARBOR, MICHIGAN		
DENSITY CONE		
DR. CMK	3-12-48	SCALE 1/2" = 1"
CK. HAGAN	3-12-48	SM 13-1
APP. W.S. Housel	3-19-48	

APPENDIX C

APPENDIX C

Consolidation tests made in model abutment.

Material	Per Cent Moisture	Volume Cu. Ft. Orig.	Volume Cu. Ft. Final	Per Cent Volume Change	Density Lbs./Cu.Ft. (Dry)	Per Cent 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	3.16	15.6			106.0	27.0	Loose
Gravel	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		Ext.vib. of total volume.
	3.26				141.3		Tamped in 4" layers.
Pea	1.60	15.6	13.60	12.8	117.5	34.9	Total vol. vibrated.
Gravel	2.77	15.6	13.55	12.8	119.1		" " "
	4.20				122.1		Tamped in 4" layers.
	0.76				123.1		" " " "
	1.60				102.5	39.0	Loose
	2.77				103.5		"
Natural	4.77	15.6	13.0	16.7	108.6	31.0	Extended vibration.
Sand	4.77				90.2		Loose
Coarse	1.06	15.6	13.55	13.1	122.6		Extended vibration.
Aggregate	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

TEST DATA

TEST DATA
VOLUME CHANGE AND DENSITIES OF DIFFERENT
MATERIALS PLACED IN THE MODEL ABUTMENT BOX

Test No.	Material	Method of Consolidation	Weight of Box and Gravel (Pounds)	Wt. of Box	Weight of Gravel (Pounds)	Gravel Surface Below Top of Box		Volume (Cu.Ft.)		Per Cent Volume Change	Per Cent Moisture	Density Lbs./Cu.Ft. Dry		Remarks
						Before Consol.	After Consol.	Original	Final			Before Consol.	After Consol.	
1	Gravel	Vibrat'n	1,923	425	1,498	4"	6"	11.73	10.00	14.7	3.65	123.0	144.4	After consolidation all tests were saturated with no change in volume resulting.
2	Sand	"	1,932	455	1,477	Full	2.63"	15.60	13.00	16.7	4.77	90.4	108.4	
3	Pea Gravel	"	2,146	480	1,666	Full	2"	15.60	13.55	13.1	2.77	103.9	119.6	
4	Coarse Agg.	"	2,147	475	1,672	Full	2"	15.60	13.55	13.1	1.06	106.1	122.1	
5	Slag	"	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	"	1,893	475	1,418	----	5.00"	---	10.80	--	5.40	---	124.6	
8	Pea Gravel	"	2,031	480	1,551	----	3.75"	---	12.00	--	0.76	---	128.3	
9	Coarse Agg.	"	2,075	475	1,600	----	3.00"	---	12.70	--	1.06	---	124.7	
10	Slag	"	1,659	445	1,214	----	3.50"	---	12.10	--	4.99	---	95.6	

APPENDIX D

Consolidation of bank-run gravel in a bin
of approximately 1000 cu. ft. capacity.

Test No.	Description of Consolidation Method	Dry Densities in Lbs. Per Cu. Ft. Surface	1 Foot Depth	2 Foot Depth	Remarks
1	Loose density	107.5(1)	---	108.5(2)	(1) Ave. of 7 tests (2) " " 5 "
2	Vertical spears or tines on 2-foot layer	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 Min. vibration. Before modification of vibrator.
5	Same as No. 4	108.2	109.7	110.1	20 Min. vibration. Before modification of vibrator.
6	Same as No. 4 and 5. (See remarks)	115.6	113.7	113.5	Vibrator buried under 2' of fill.
7	Tube vibrator dragged down sloping face of fill.	110.9	113.9	115.3	Vibrator modified as shown in Figure 27.
8	Tamped	117.2	111.3	---	
9	Platform vibrator	109.2	110.3	106.0	Original platform vibrator.
10	Platform vibrator. (Modified)	Ave. 109.4 after one pass. Ave. 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' lift and vibrated 20 minutes. From these tests this method appears to be the most effective and adaptfor large volumes of backfill.

Consolidation of sand backfill in metal crib retaining wall.

Test No.	Description of Test	Moisture Content Per Cent	Dry Density Lbs/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	" "	5.3	103.6	Normal "
5	" "	5.8	108.2	" "
6	" "	6.3	102.3	" "
7	" "	5.0	105.5	" "
8	" "	5.3	103.8	" "
9	" "	4.9	102.5	Vibrated - 3" plank base with metal base plate having large corrugations.
10	" "	5.8	99.9	Same as No. 9.
11	" "	6.8	101.6	Same as No. 9 except metal base plate with small corrugations.
12	" "	6.9	95.6	Same as No. 11.
13	" "	5.4	105.5	Vibrated - Plank base and sheepsfoot metal base.
14	" "	5.9	103.3	Same as No. 11.
15	" "	5.1	102.2	Same as No. 9.
16	" "	5.7	104.4	" " " "
17	" "	5.7	109.0	Same as No. 9 except two passes were made.
18	" "	6.8	110.6	3" plank removed for remainder of tests. Base plate with small corrugations.
19	" "	5.6	107.4	Same as No. 18. Vibrated 4 min. one pass at 3600 v.p.m.
20	" "	6.1	104.8	Same as No. 18. Vibrated 5 min. at 2400 v.p.m.
21	" "	8.4	105.3	Same as No. 18.
22	" "	8.2	114.8	Prolonged tamping with sheepsfoot base plate.
23	" "	8.3	101.5	Same as 18, vibrated only 2-1/2 minutes.

Consolidation of Mayfield bridge backfill.

Test No.	Stock pile Density Lbs./Cu.Ft.	Labor-atory Density Lbs./Cu.Ft. (Loose Dry)	Labor-atory Density Lbs./Cu.Ft. (Cone)	Field Density	Shrinkage Stockpile To Fill (Per Cent)	Remarks
1	89.3	96.8	114.2	105.4	18.1	Tests 1,2,3 & 4 were made
2	"	"	"	104.0	16.4	on a 33" lift and were
3	"	"	"	103.0	15.3	taken from depths of
4	"	"	"	94.5	5.8	(2"-7"),(2"-7"),(12"-20")
5	"	"	"	102.0	14.2	&(26"-30").Tests 5,6,& 7
6	"	"	"	101.7	13.8	were made on a 15" lift &
7	"	"	"	104.2	16.7	from depths of (2"-8"),
8	98.8	103.6	113.6	107.0	8.3	(2" - 8") & (12"-20").
9	"	"	"	110.4	11.7	Tests 8,9,10 & 11 were
10	"	"	"	110.6	12.0	made on a 36" lift & from
11	"	"	"	104.7	6.0	depths of (2"-8") (2"-10")
12	"	"	"	110.1	11.4	(12"-20") & (33"-38").
13	"	"	"	116.0	17.4	Test 12,13,14, & 15 were
14	"	"	"	110.2	11.5	made,after vibrating a 12"
15	"	"	"	104.9	6.2	lift,from depths of (2"-
						8") (2"-10") (12"-20") &
						(30"-35").

On this project there was approximately six feet of backfill 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 right through to the top, is consolidated.