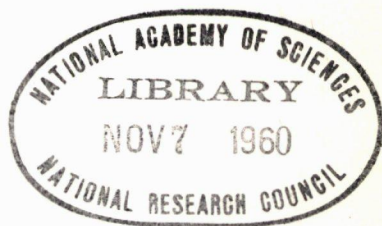


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Bulletin 254

*Soil Compaction
and
Proof-Rolling of Subgrades*



National Academy of Sciences—

National Research Council

publication 763

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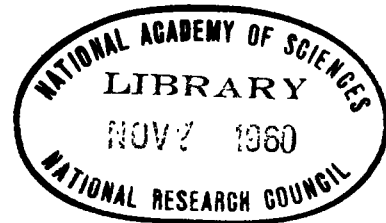
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Full-Scale Compaction Studies at the British Road Research Laboratory

W.A. Lewis, Road Research Laboratory, Department of Scientific and Industrial Research, Harmondsworth, England

Since 1945 the Road Research Laboratory has been carrying out a program of research on soil compaction, in which studies have been made, under controlled conditions, of most types of compaction plant available to the engineer in the British Isles. The object of this work has been to provide data to enable the engineer to select the most suitable type of plant for soil compaction work. As all the tests have been made on the same range of soils using the same test techniques, it has been possible to make a direct comparison between the performances of the various types of plant. The results of many of these tests have already been published (1, 2).

Apart from full-scale tests on existing plant, studies, more fundamental in their nature, have also been made of a number of aspects of soil compaction. This work has included a detailed investigation of impact compaction by rammers (3) and of the effect of wheel load and tire-inflation pressure on the performance of pneumatic-tired rollers (4).

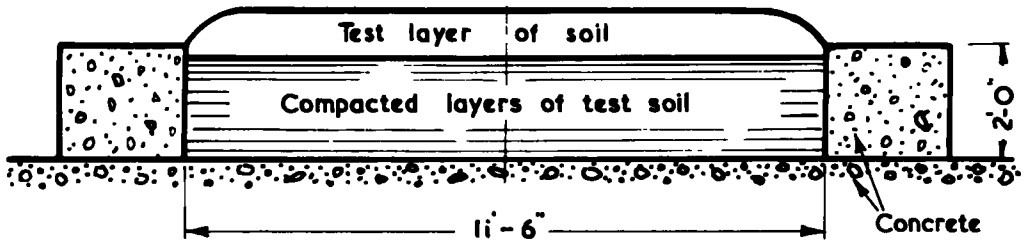
The object of this paper is to give a brief outline of the test installation and techniques employed at the Road Research Laboratory in carrying out the full-scale compaction tests and to give a summary of the main results obtained in the investigations.

TEST INSTALLATION AND SOILS USED

● THE EARLIER investigations of the performance of soil compaction plant were carried out on a circular test track containing five test soils in bays 2 ft deep, 11 ft 6 in. wide and about 45 ft long (1) (Fig. 1). Plant that was not self-propelled was towed by an electric lorry operated by remote control and running on the concrete walls of the soil bays. In this way it was possible to study the performance of the roller under test without introducing any compacting effect of the towing machine.

Although this circular test track was very satisfactory for relatively small types of plant, it was not suitable for testing the large types of compaction equipment, such as the 20- to 50-ton pneumatic-tired rollers, now being used in earthwork construction. These heavier rollers require a greater draw-bar pull than could be provided by the electrically operated towing lorry of the circular test track and, in addition, there was insufficient headroom below the roof for the large rollers. It was also considered that the 2-ft depth of test soil was insufficient for these large rollers in particular.

In view of these and other limitations of the circular track, such as the difficulties introduced in operating machines on a circular path, a special building was constructed in 1954 for carrying out full-scale compaction tests on the largest types of compaction machines (Figs. 2 and 3). This building is approximately 100 ft long and 90 ft wide and provides 16 ft of headroom beneath the roof trusses. The building contains five soil bays, each 35 ft long, 15 ft wide and 3 ft deep. The floor of the bays was left as the natural foundation soil (sandy clay) as this was likely to have elastic properties similar to those of the compacted test soils; the conditions would therefore approximate more closely to those found in the field than if a concrete floor to the bays had been employed.



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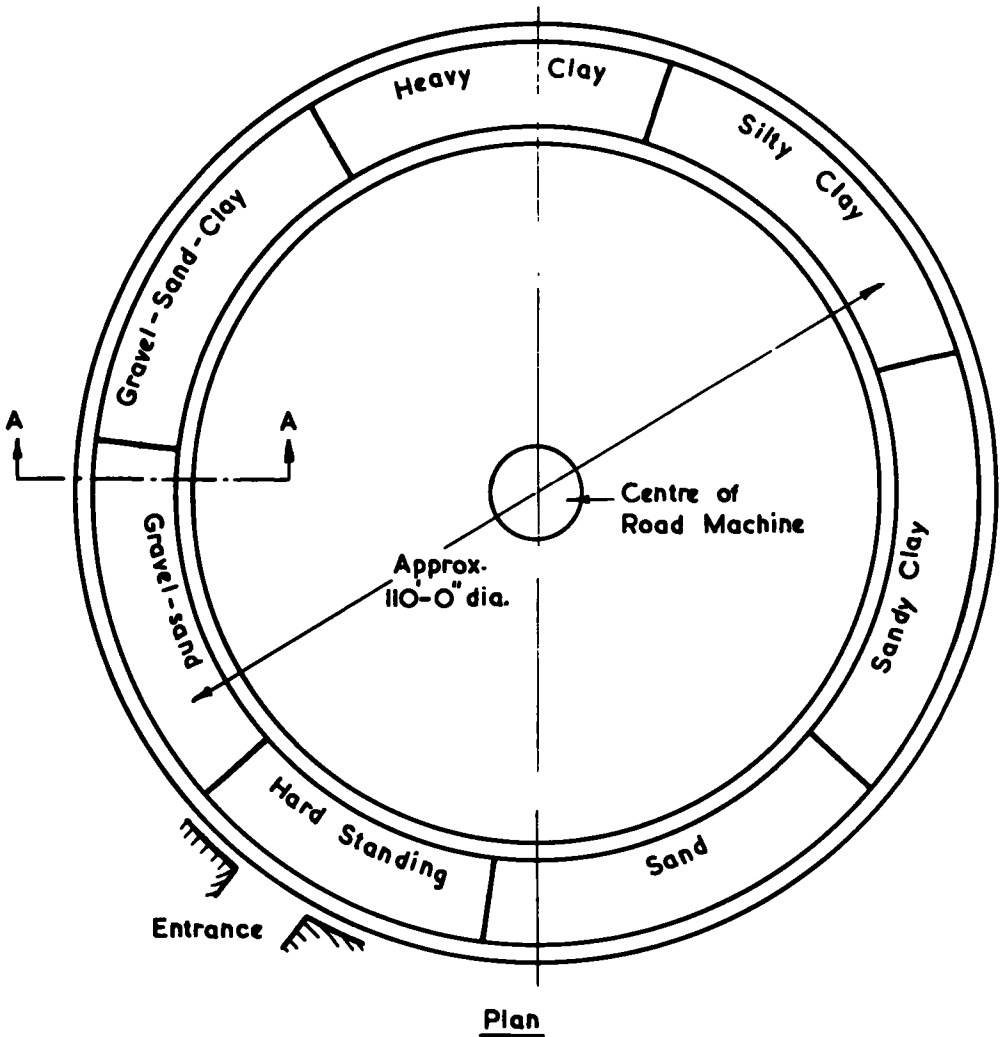


Figure 1. Plan and cross-section of earlier circular track used for full-scale compaction tests.

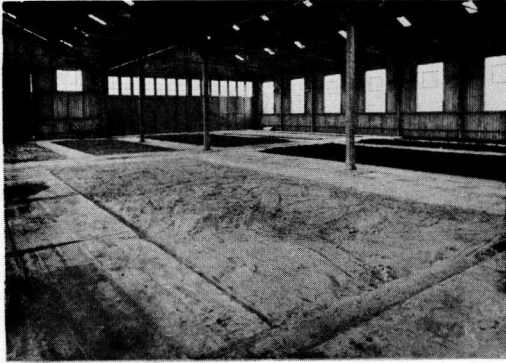


Figure 2. Special building at the Road Research Laboratory for carrying out full-scale tests with the compaction plant.

The five soils used in the bays are a heavy clay, a sandy clay, a well-graded sand, a uniformly graded fine sand, and a gravel-sand-clay. These soils are representative of a wide range of the soils found in Great Britain. The four main soils (heavy clay, sandy clay, well-graded sand and gravel-sand-clay) are almost identical to the corresponding soils of the earlier circular test track but the uniformly graded fine sand is an entirely new soil that was introduced to provide some experience with a difficult material; so far only a few machines have been tested on it. The new building also has two bays containing granular road-base materials (wet-mix graded limestone and wet-mix graded slag) to obtain the performance of plant in the compaction of these types of materials.

Details of the particle-size distributions and plasticity properties of the test soils used in the investigations are given in Figure 4.

FACTORS STUDIED

The three main relations determined in the investigations were:

1. Relation between the moisture content of the soil and the state of compaction (dry density) produced.
2. Relation between the number of passes of the plant and the state of compaction produced.
3. Variation in the state of compaction with depth below the surface of the compacted layer.

The relation between the moisture content of the soil and the state of compaction obtained is a function of the energy provided per unit volume of soil and is therefore dependent on the thickness of layer and the number of passes of the compacting machine employed. In determining this relation, the thickness of layer used was the maximum that still enabled satisfactory states of compaction to be obtained in the lower portions of the layer. A sufficient number of passes of the plant is provided such that the soil is compacted to a condition where no significant increase in the state of compaction can be produced by further passes of the compaction plant. Thus the relations between the moisture content of the soil and the state of compaction obtained in the investigations correspond to the best performance likely with the machine.

In the determining relations between the number of passes of the plant and the state of compaction produced, the same thickness of layer is employed as in the study of the effect of moisture content. Although the moisture content of the soil has an effect on the relation between the number of passes and the state of compaction, it is impracticable to determine this relation for a range of moisture contents for each soil owing to the amount of work involved. In general, therefore, this study is only carried out at one moisture content—usually the optimum moisture content obtained in the previous investigation with the particular machine under test.

The investigations of the variations in the state of compaction with depth below the surface of the layer are carried out with somewhat thicker layers than those employed in determining the previous two relations. This is done to give a clearer picture of the variations in the state of compaction with depth and an indication of the maximum thickness of layer that can be employed in practice. The relation is determined for the condition of the soils compacted to refusal at approximately the optimum moisture contents for compaction by the plant.

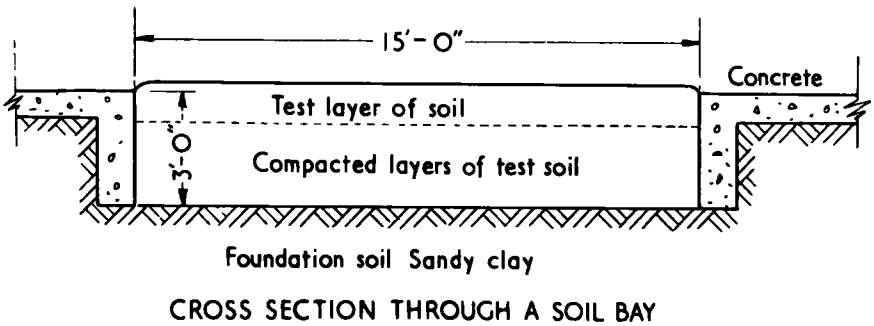
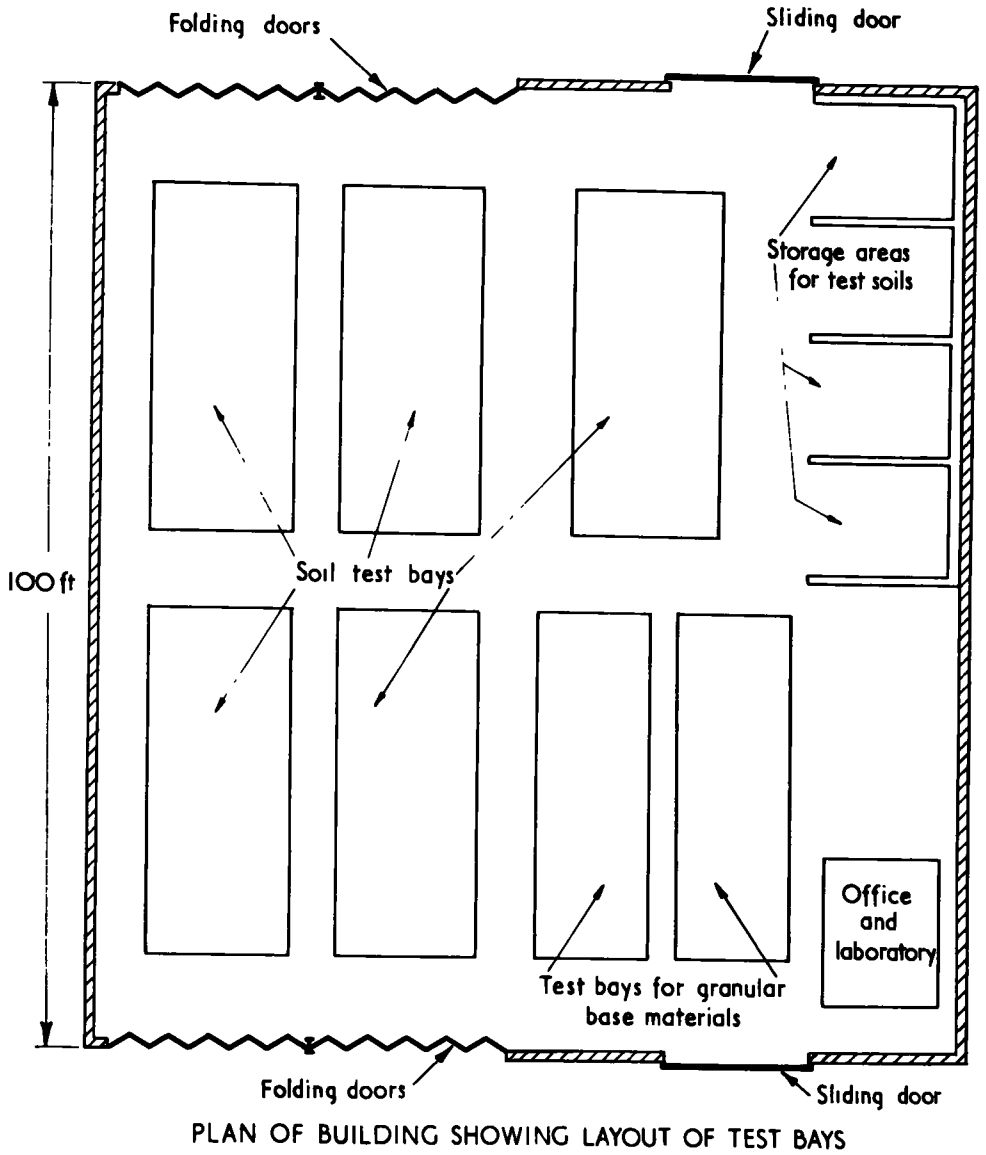


Figure 3. Plan of special building constructed at the laboratory for carrying out full-scale compaction tests and cross-section through a soil bay.

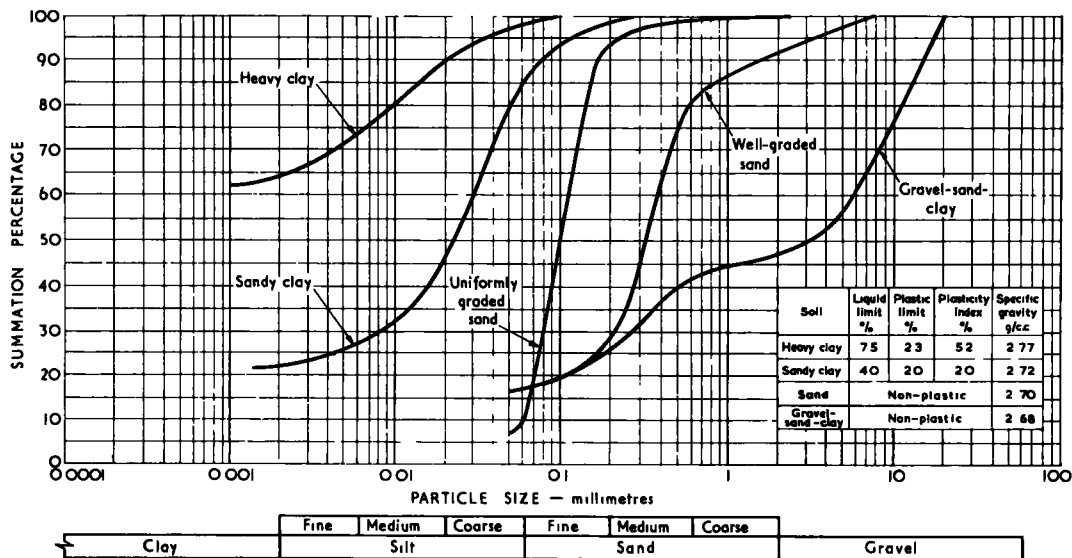


Figure 4. Particle size distributions and results of plasticity and specific gravity tests for the soils used in the investigations.

TEST PROCEDURE

Preliminary Work

Before the main investigations are carried out, a few preliminary tests are made to provide an indication of the thickness of loose layer to be employed in the studies of the relations between the dry density and the moisture content and between the dry density and the number of passes. In most of the tests, loose layers at 9 in. or 12 in. have been employed.

Dry Density and Moisture Content

To determine the effect of moisture content on the dry density of soil when fully compacted by the plant, loose layers of the test soils of appropriate thickness for the particular machine are brought to a suitable low moisture content by aeration using a rotary cultivator. A small amount of water is then added to the soil followed by further mixing by the rotary cultivator to give a uniform loose tilth which represents a state of minimum compaction. This layer is then compacted by a sufficient number of passes of the plant to insure that little further compaction can be obtained (generally 32 passes with rollers and 16 passes with vibrating plate compactors). The dry density of the top 6 in. of compacted soil is then determined by the sand-replacement method (5), ten measurements of dry density being made to obtain an accurate mean value.

The compacted soil is then broken up and its moisture content increased by 1 to 2 percent by spraying. After further mixing, the procedure outlined above is repeated. In this way the relation between the dry density and the moisture content is built up for the test soils for the plant under test. As in the standard laboratory compaction test (5, 6), it is essential to start with the test soil at a low moisture content and to increase the moisture content in stages as otherwise considerable scatter could be introduced in the results owing to the small but significant difference in the moisture properties of the soil in the wetting and drying conditions. After making the dry density measurements with the sand-replacement apparatus, the sand used in the density holes is removed by a powerful vacuum cleaner. This not only saves considerable time but results in virtually the complete removal of the sand which otherwise would tend to produce over a period of time some contamination of the test soils.

Dry Density and Number of Passes of the Plant

A loose layer of the test soil of appropriate thickness for the plant under test is prepared at the desired moisture content (generally the optimum moisture content obtained in the previous test). The layer is then compacted by one or two passes of the plant and the state of compaction is determined as outlined previously. After breaking the soil up, a fresh loose layer is prepared, the moisture content is kept constant by the addition of a small amount of water if required. The procedure is repeated with a progressively increasing number of passes of the plant and in this way the complete relations are obtained.

Dry Density and Depth in the Compacted Soil

Loose layers of the test soils about 20 in. thick are prepared at approximately the optimum moisture content for compaction by the machine under test and are then compacted to refusal. The state of compaction of each 2 to 3 in. in depth in the compacted layer is then determined, ten measurements of dry density being made at each level. After the dry density measurements at each depth are completed, 2 to 3 in. of soil is carefully removed to expose a fresh surface.

RESULTS OF FULL-SCALE COMPACTION TESTS

The results of the full-scale tests on the plant listed in Table 1 represent a considerable amount of data and it is impracticable to give more than a brief summary of the results.

TABLE 1
DETAILS OF THE COMPACTION PLANT

Pneumatic-tired rollers	12-ton	20-ton	45-ton		
Total laden weight (lb)	26,880	44,800	100,800		
Total rolling width (in.)	82	84	93		
Number of wheels	9 (4 front, 5 rear)	9 (4 front, 5 rear)	4		
Average weight per wheel (lb)	2,990	4,980	25,200		
Tire-inflation pressure (lb/sq in.)	36	80	140		
Smooth-wheeled rollers	2 $\frac{3}{4}$ -ton	8-ton			
Total laden weight (lb)	6,160	19,010			
Total rolling width (in.)	51	70			
Diameter and width, front rolls (in.)	34 x 34	42 x 42			
Diameter and width, rear rolls (in.)	36 x 16	54 x 18			
Weight/in. width, front rolls (lb/in.)	80	186			
Weight/in. width, rear rolls (lb/in.)	142	311			
Sheepsfoot rollers	Club-foot	Taper-foot			
Total laden weight (lb)	11,010	10,080			
Size of feet (in.)	4 x 3	2.2 x 2.2			
Foot pressure (lb/sq in.)	115	249			
Length of feet (in.)	7.0	7.7			
Rammers	Frog	Power			
Total weight (lb)	1,350	250			
Diameter of base (in.)	29	9.5			
Approximate height of jump (in.)	12	12			
Vibrating smooth-wheeled rollers	4-cwt	6 $\frac{3}{4}$ -cwt	2 $\frac{1}{2}$ -ton	3 $\frac{3}{4}$ -ton	
Total weight (lb)	480	760	5,400	8,620	
Diameter and width of vibrating roll (in.)	21 x 24	22 $\frac{3}{4}$ x 28	30 x 32	48 x 72	
Static weight/in. width (lb/in.)	20	27	68	119	
Approximate frequency of vibration (cycles/sec)	4,500	4,500	5,000	2,300	
Vibrating plate compactors	4-cwt	13-cwt	14-cwt	1 $\frac{1}{2}$ -ton	2-ton
Total weight (lb)	53	1,480	1,570	3,350	4,480
Approximate area of plate (sq in.)	280	660	570	970	1,700
Width of plate (in.)	15	24	24	30	34
Approximate frequency of vibration (cycles/sec)	1,800	1,200	1,500	1,100	1,050
Track-laying tractors	40-hp	80-hp			
Total weight (lb)	12,840	24,160			
Width of track (in.)	15	20			
Distance between tracks, center to center (in.)	52	72			
Average pressure of tracks (lb/sq in.)	7.3	7.2			

TABLE 2
 MAXIMUM DRY DENSITIES AND OPTIMUM MOISTURE CONTENTS IN LABORATORY AND FULL-SCALE TESTS

Soil Type	Heavy Clay		Sandy Clay		Well-Graded Sand		Gravel-Sand-Clay	
Casagrande Classification	CH		CL		SW		GW	
Avg. Moisture Range for Soils, %	22 - 27		16 - 21		7 - 11		5 - 9	
	Maximum Dry Density (lb/cu ft)	Optimum Moisture Content (%)	Maximum Dry Density (lb/cu ft)	Optimum Moisture Content (%)	Maximum Dry Density (lb/cu ft)	Optimum Moisture Content (%)	Maximum Dry Density (lb/cu ft)	Optimum Moisture Content (%)
British Std. compaction test	99	24	109	16	121	11	129	9
Mod. AASHTO compaction test	116	16	126	12	130	9	138	7
Dietert compaction test	102	23	111	15	109	11	-	-
12-ton pneumatic-tired roller	101	23	111	18	127	10	132	8
20-ton pneumatic-tired roller	106	21	117	15	128	9	139	6
45-ton pneumatic-tired roller	111	19	120	14	131	9	139	6
2 $\frac{3}{4}$ -ton smooth-wheeled roller	96	21	112	17	131	9	137	7
8-ton smooth-wheeled roller	104	20	116	15	132	9	138	7
5-ton club-foot sheepsfoot roller	107	16	118	12	-	-	130	6
4 $\frac{1}{2}$ -ton taper-foot sheepsfoot roller	107	15	118	13	-	-	128	5
2-cwt power rammer	110	18	119	11	129	8	137	6
12-cwt frog rammer	106	17	113	14	128	10	136	7
4-cwt vibrating roller	-	-	-	-	124	11	123	8
6 $\frac{3}{4}$ -cwt vibrating roller	92	28	100	16	127	9	132	8
2 $\frac{1}{2}$ -ton vibrating roller	96	21	-	-	133	7	139	6
3 $\frac{3}{4}$ -ton vibrating roller	106	21	119	14	137	7	145	6
4-cwt vibrating plate compactor	-	-	-	-	128	10	127	9
13-cwt vibrating plate compactor	103	21	116	15	135	8	141	6
14-cwt vibrating plate compactor	87	20	114	16	130	9	137	7
1 $\frac{1}{2}$ -ton vibrating plate compactor	-	-	-	-	129	9	135	8
2-ton vibrating plate compactor	98	17	-	-	128	9	137	7
40-hp track-laying tractor	96	22	-	-	128	10	128	8
80-hp track-laying tractor	99	24	-	-	-	-	126	8

Table 2 gives a comparison of the maximum dry densities and corresponding optimum moisture contents for the four main test soils obtained with the various types of compaction equipment. The results of standard laboratory compaction test (5, 7, 8) have also been included together with an estimate of the average 'equilibrium' or natural moisture content range for the soils in the field for Great Britain.

Although in a few cases there was reasonable agreement between the results obtained with a laboratory compaction test and those obtained with the plant, in general the laboratory compaction tests gave only a poor indication of the maximum dry densities and corresponding optimum moisture contents likely to be obtained with the plant. In any case, the view held at the Road Research Laboratory is that the soil should be compacted in the field close to its equilibrium or natural moisture content rather than at an optimum moisture content obtained in some arbitrary laboratory compaction test. For mass earthworks it is considered that the state of compaction to be achieved in the field with well-graded granular and cohesive soils should be not less than that corresponding to an air void content of 10 percent; that is, to a dry density of at least 90 percent of the saturation dry density.

Table 3 was prepared using the results obtained from the investigations of the relations between the number of passes of the plant and the state of compaction and from the measurements of the variations in the state of compaction with depth. Table 3 shows the possible outputs of the various items of plant tested in compacting soil to a state of compaction corresponding to 10 percent of air voids at the average natural moisture contents. The results obtained on the test soils have been averaged. It must be remembered, however, as indicated in Table 2, that not all the machines operated satisfactorily on all the soils. It has been assumed in the calculations of output that the machines are in operation for 50 min in each hour; no allowance has been made for the time spent in turning machines around, nor for delays due to bad weather. The figures for the cost of operation of the plant are based on average 1958 hire rates for the machines and include an estimate of the operators' wages and the cost of fuel. No allowance

has been made for the cost of transporting plant to and from the site. The operating costs for the pneumatic-tired rollers include the cost of a track-laying tractor of appropriate size for the roller. Because of the limited use of sheepsfoot rollers in Britain, they have been omitted from Table 3.

Table 3 shows that for mass earthworks the most economical items of plant are likely to be heavy smooth-wheeled rollers or pneumatic-tired rollers. Although the output of pneumatic-tired rollers increases with size and loading, the cost of operation also increases because the higher cost of the roller and the necessity of having to employ a more powerful tractor for towing the roller. The resulting cost per cubic yard of compacted soil is not very different for the various sizes of pneumatic-tired roller, with only a slight tendency for the heavier rollers to be more economic. It seems apparent, therefore, that the use of very large sizes of pneumatic-tired roller is unlikely to result in spectacular savings in the cost of compaction of soil in the British Isles as compared with those obtainable with the lighter rollers. Thus the selection of the size and weight of pneumatic-tired rollers is likely to be based more on site conditions, the availability of plant and the compaction requirements, than on the question of the economics of operating the rollers.

The relative outputs and costs of compacting soil (Table 3) can only be considered as a guide since they are based on ideal conditions of working and may be affected by many factors. It is desirable, therefore, that on large earthworks a small compaction trial should be carried out before the construction work to determine the type of plant, the optimum thickness of layer, and the number of passes that will give the desired state of compaction with the lowest cost per cubic yard of fill material.

FUNDAMENTAL INVESTIGATIONS

Apart from the investigations of the performance of compaction plant outlined above, a number of more fundamental studies have been carried out at the Road Research Laboratory. The object of these studies was to provide data for use by manufacturers in the design of improved compacting machines. The principal investigations carried out have been:

1. A study of some of the factors likely to affect the performance of impact compactors on soil (3).
2. An investigation of the effect on the compaction of soil of varying the diameter

TABLE 3
ESTIMATED PLANT OUTPUTS IN COMPACTING SOIL¹

Type of Plant	Approximate Hourly Cost of Operating Plant (including hire, wages of operator and fuel), ^a dollars	Average Output of Plant					Cost per Cu Yd of Compacted Soil, cents ^b	
		Width Compacted by plant, in.	Speed of Rolling, ft per min	Number of Passes Required	Area Compacted per Hour, sq yd	Depth of Compacted Layer, in.		
12-ton pneumatic-tired roller	6.1	82	200	4	1,900	5	260	2.3
20-ton pneumatic-tired roller	8.2	84	200	4	1,940	6	320	2.6
45-ton pneumatic-tired roller	14.3	93	200	3	2,900	10	800	1.8
2 ¹ / ₂ -ton smooth-wheeled roller	2.0	51	180	8	530	5	74	2.7
8-ton smooth-wheeled roller	2.5	70	180	4	1,500	6	250	1.0
2-cwt power rammer	1.2	0.49 ^c	60 ^d	6 ^e	27	6	4.5	27
12-cwt frog rammer	1.8	4.6 ^c	50 ^d	12 ^e	110	12	36	5.0
4-cwt vibrating roller	0.9	24	30 ^d	8	42	3	3.5	26
6 ³ / ₄ -cwt vibrating roller	1.2	28	60	16	49	6	8.2	15
3 ³ / ₄ -ton vibrating roller	4.9	72	120	6	670	6	110	4.5
4-cwt vibrating plate compactor	1.4	15	28	3	65	5	9.0	16
13-cwt vibrating plate compactor	1.7	24	60	4	170	8	37	4.6
14-cwt vibrating plate compactor	1.7	24	42	2	230	6	39	4.4
1 ¹ / ₂ -ton vibrating plate compactor	2.3	30	25	2	170	12	57	4.0
2-ton vibrating plate compactor	2.3	34	27	2	210	12	70	3.3
40-hp track-laying tractor	4.6	30	350	6	810	6	130	3.5
80-hp track-laying tractor	7.3	40	440	6	1,400	6	230	3.2

¹Compacting soil to 10 percent air voids, the average natural moisture content in the British Isles.

²The costs (1958 prices) are those for Great Britain converted into dollars and cents.

³Area of soil compacted by each blow (sq ft).

⁴Average number of blows per minute.

⁵Number of blows.

⁶Hand-propelled.



Figure 5. General view of the equipment used in the investigation of impact compaction. The rammer has been raised to a height of about 5 ft and the operator is about to trip the quick-release mechanism.

ft-lb per sq in. to 8 ft-lb per sq in., had no significant effect on the state of compaction obtained at any moisture content provided the total compactive energy per unit area was kept constant. Thus, within the range of maximum compactive energies per blow that are likely to be adopted from consideration of practical design, it is only necessary to consider the total compactive energy per unit area of rammer. Nevertheless, to avoid overstressing during compaction of soils with a low bearing capacity, it would be necessary to employ a lower compactive energy per blow than would be possible with soils with a high bearing capacity.

Diameter of Smooth-Wheeled Rollers

One of the factors that might be expected to affect the state of compaction produced in soil by smooth-wheeled rollers is the diameter of the rolls. To give an indication of the importance of this factor, an investigation was carried out using a special apparatus in which rolls of various diameters but constant width could be fitted. The rolls used had diameters of 1, 2 and 3 ft, a width of 2 ft, and were loaded to 142 psi width.

The results obtained showed that there was no significant difference between the states of compaction obtained for any given number of passes with the 2- and 3-ft diameter rolls. It was not found possible to operate with the 1-ft diameter roll since the soil built up in front of the roll and prevented rotation. The results suggest, therefore, that it is unlikely that any significant improvement in the compacting performance of smooth-wheeled rollers can be achieved by small alterations in the diameter of the rolls.

of a smooth-wheeled roller (9).

3. An investigation of the effect of wheel load and tire-inflation pressure on the performance of pneumatic-tired rollers in the compaction of soil (4).

Impact Compaction

The equipment used in the tests (Fig. 5) comprised a special rammer working in conjunction with a pile-driving frame suspended from the jib of an excavator. The rammer consisted of a fabricated light-alloy cage, the weight of which could be adjusted from about 1 cwt to about 4½ cwt by bolting steel plates inside the cage. The area and shape of the rammer base could also be altered from a minimum size of 1 sq ft to a maximum size of 2¼ sq ft by bolting on to the base of the rammer special steel plates. The rammer could be lifted to any height up to about 9 ft above the ground and, using a quick-release mechanism, it could be allowed to drop freely on the surface of the soil.

The results obtained in the investigation showed that, for a given size of rammer, the state of compaction produced was a function of the kinetic energy per unit area of the rammer. When the rammer size was increased, to obtain the same state of compaction the compactive energy per unit area of the rammer had also to be increased in proportion to the square root of the area of the rammer. The compactive energy per blow, within the limits of 4.5

Wheel Load and Tire-Inflation Pressure

Three sizes of pneumatic-tired roller were employed in the investigations with wheel loads ranging from 1.3 tons at a tire-inflation pressure of 36 psi to wheel loads of 10 tons at a pressure of 140 psi. The investigation included a study of the effect of speed of rolling on the state of compaction produced. The following main conclusions were drawn:

1. The contact area and contact pressure under the tires, both of which affect the state of compaction, are functions of the wheel load and the inflation pressure.
2. An increase in the wheel load or in the tire-inflation pressure produces an increase in the maximum dry density with a correspondingly lower optimum moisture content.
3. For the compaction of soils similar to the two clay test soils (Fig. 4), there appears to be little point in employing tire-inflation pressures much in excess of 40 to 50 psi with wheel loads of about 5 tons.
4. For the compaction of granular soils in the British Isles there appears to be some advantage in employing the highest tire-inflation pressures and the heaviest wheel load practicable, consistent with avoiding overstressing of the soils.
5. The magnitudes of the wheel load and tire-inflation pressure had very little influence on the general shape of the curves relating the number of passes of the roller with dry density, very little increase in the state of compaction being obtained after 16 passes.
6. In general, about 4 passes of the rollers were necessary to produce states of compaction equivalent to an air-void content of 10 percent.
7. To achieve the greatest output of compacted soil, the highest practicable rolling speed should be employed with, if necessary, a slight increase in the number of passes to compensate for the slight drop in the state of compaction that might result from employing the higher speed.
8. In general, the higher the wheel load and tire-inflation pressure, the higher was the state of compaction at any given depth.
9. The maximum thickness of loose layer that can be employed to achieve a minimum acceptable state of compaction equivalent to an air-void content of 10 percent ranges between 6 to 9 in. for the lightest combination of wheel load and tire-inflation pressure to 14 to 18 in. for the heaviest combination used.
10. Although the output of the rollers increased with size and loading, the cost of operation is also likely to increase at a similar rate and the resulting cost per unit volume of compacted soil is not likely to be very different for the various sizes of roller.

SUMMARY

The Road Research Laboratory has been carrying out a program of research on soil compaction, in which studies have been made under controlled conditions of the performance of most types of compaction plant available in Great Britain. Special installations have been employed for carrying out the tests; the building contains five soil bays, each 15 ft wide and 35 ft long, filled with a 3 ft depth of test soil. The soils, which are representative of a wide range of the soils found in Great Britain, are a heavy clay, a sandy clay, a well-graded sand, a uniformly graded fine sand, and a gravel-sand-clay.

The results of the full-scale tests are summarized and suggest that, for the compaction of mass earthworks, pneumatic-tired rollers and heavy smooth-wheeled rollers are likely to be the most economical plant. In addition to the tests on compaction machines, a number of studies of a more fundamental nature have been made; these include investigations of impact compaction by rammers and of the effect of wheel load and tire-inflation pressure on the performance of pneumatic-tired rollers.

ACKNOWLEDGMENTS

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Proof-Rolling of Subgrades

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Proof-rolling of subgrades is being used by state highway departments to check the adequacy of normal compaction and to correct any deficiencies that may exist. Proof-rolling is an excellent tool, but there is a growing tendency to look on proof-rolling as a panacea that will end all compaction problems. This is a false approach, as proof-rolling will find and correct deficiencies only under certain conditions of moisture. This paper shows the cases where proof-rolling will be effective and those cases where it will not. The paper also contains information that can be used to establish desirable roller characteristics and the amount of rolling that should be used for various conditions.

● THE TERM "PROOF-ROLLING" as used in this paper refers to the application of a few coverages of a heavy rubber-tired roller on a subgrade or other layer of a subbase or base course after the completion of normal compaction. The procedure is used to check the adequacy of the normal compaction and to correct any deficiencies that may exist. In recent months several states have incorporated proof-rolling requirements into their specifications for heavy-duty pavements. Proof-rolling is an excellent tool that will assist materially in the construction of adequate pavements, but there is a growing tendency to look on proof-rolling as a panacea for all compaction problems, even to the extent of relaxing on inspection and construction control testing where proof-rolling is being used. This is an entirely false approach as proof-rolling is not a positive cure-all. The primary purpose of this paper is to show where proof-rolling will correct compaction deficiencies and where it will not. Also, the paper contains information that can be used to establish desirable roller characteristics and the amount of rolling that should be used for various conditions.

BACKGROUND

Heavy rubber-tired rollers have been used to compact buried layers since their development by Porter (1) in the period 1943-1944, and he and others have used the very heavy rollers to proof-test construction on many airfields. The term proof-rolling was introduced into the Corps of Engineers' guide specifications (2) for graded crushed-aggregate base courses in 1957. The specifications required the application of 30 coverages with a 50-ton roller after the layer in question had been compacted to 100 percent of modified AASHO density. The primary purpose of the 30 coverages was to boost the density to the range of 103 to 104 percent of modified AASHO that was considered to be necessary. The application of 30 coverages was not proof-rolling in the sense defined in this paper, but the term seemed apt and has been adopted and used widely.

The application of 30 coverages of proof-rolling which was required on the Columbus Air Force Base, Columbus, Miss., as part of the Corps of Engineers' job specifications was thought to be a definite requirement to preclude any occurrence of deficiencies from lack of compaction. However, the time between completion of normal rolling and the start of proof-rolling was sufficient to permit the base course to dry out to such a degree that the 30 coverages did not produce the anticipated density. Settlement under the accelerated traffic test (3) produced approximately $\frac{3}{4}$ -in. densification within the 10-in. layer of base course. In retrospect, this case is a classic example where proof-rolling did not get the intended job done because of probable strengthening of the base due to cementation as a result of drying and the inability to reintroduce the proper moisture content for compaction.

To understand what happened at Columbus and what can happen in other cases, it is necessary to have a working knowledge of the interrelationships of moisture, density, and strength that exist during and immediately after construction and during subsequent periods when the pavement is being subjected to traffic. These relationships are illustrated in this paper by schematic diagrams rather than by actual presentations of test data. The trends indicated by the schematic diagrams are adequately supported in the literature. References are used to indicate the source of the test data in support of the illustrated trends.

In the discussion, primary emphasis is placed on cohesive materials, and all diagrams are for cohesive materials. The principles, however, apply to both cohesive and cohesionless materials, with the one exception that cohesionless materials can usually lose moisture quickly by drainage as well as by evaporation; therefore, the time element between normal compaction and proof-rolling is more critical for cohesionless materials than for cohesive materials.

RELATIONSHIP OF MOISTURE, DENSITY, AND COMPACTION EFFORT

Figure 1 illustrates the typical relationship between moisture content and density that is obtained in the laboratory compaction test on cohesive soils (4). If samples

are compacted at a range of moisture contents with a given compaction effort, for example 12 blows per layer with the specified hammer, densities similar to those indicated by the small circles (Fig. 1) are obtained. A smooth curve drawn through these points represents the moisture-density relationship for the given compaction effort.

The moisture content at the peak of the curve is termed the optimum because the maximum density for the specified compaction effort is obtained at the optimum moisture content. If the test is made at higher compaction efforts, for example 25 and 55 blows per layer, relationships similar to those shown in Figure 1 are obtained (5, 6). The peaks of the compaction curves can be joined by a straight line, designated the "line of optimums."

This diagram shows that higher densities are produced with higher compaction efforts if the moisture content is right. No material increase in density can be expected from the higher compaction efforts if the moisture content is appreciably above or below optimum.

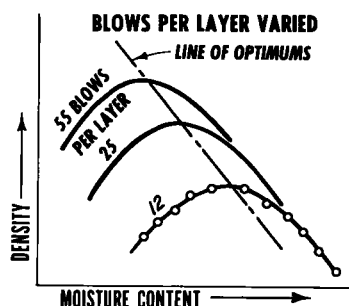


Figure 1. Laboratory compaction.

VARIABLES AFFECTING FIELD COMPACTION

The purpose of requiring adequate compaction during construction is to prevent settlement under future traffic and to obtain higher strength. The next figures illustrate the variables that affect the density that can be obtained during construction with rubber-tired rollers. Incidentally, these same variables, inclusive of the depth to the layer being considered, affect the density produced by traffic during the life of the pavement. Figure 2 shows the effect of tire pressure in producing higher densities (6, 7) and illustrates the effect of tire pressure when all other variables are held constant. Increasing the tire pressure increases the compaction effort and produces a pattern

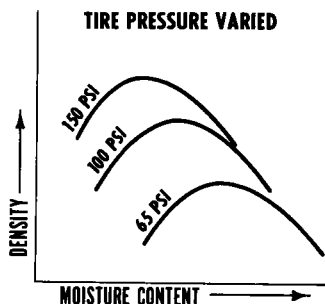


Figure 2. Field compaction.

of moisture-density relationships similar to that produced by increasing the number of blows in the laboratory test. Figure 3 shows the effect of increasing the number of repetitions (6, 7) when other variables are held constant. Increasing the number of repetitions increases the compaction effort and produces higher densities in the same manner as increasing the number of blows in the laboratory test.

Figure 4 shows the effect of total load (8) when tire pressure and repetitions are held constant for the case where the tire is operating directly on the layer being compacted and for the condition where the tire is substantially wider than the thickness of the layer being compacted. Under these conditions the density obtained is not related to the load on the tire, but is related to the tire pressure, which in this case is assumed constant for all three loads. Gross load is a factor when layers at a substantial depth are being considered, which is the case under actual traffic or in proof-rolling. Figure 5 shows the effect of gross load.

The two solid-line curves (Fig. 5) are plots of the maximum theoretical vertical stress with depth (9) for a 50-ton and a 16-ton rubber-tired roller. The tires on the

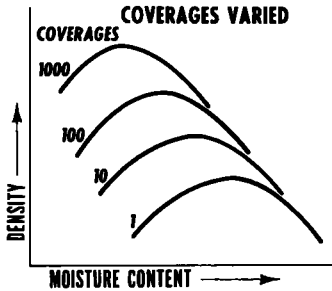


Figure 3. Field compaction.

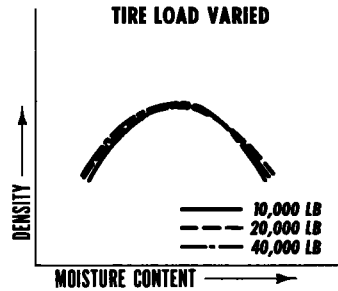


Figure 4. Field compaction.

50-ton roller have a tread width of 16 in. and those on the 16-ton roller have a tread width of only 8 in. Inflation pressure in both cases is 90 psi. The vertical stress produced in the top few inches is approximately the same for each roller, but at the deeper depths the roller with the wider tire produces higher stresses to a greater depth. Vertical stress is related in a general way to compaction effort. The dashed-line curve (Fig. 5) represents the densities produced at various depths in a lean clay soil at a moisture content of 15 percent with 8 coverages of a 50-ton rubber-tired roller with tires inflated to 90 psi (10). The density produced with a rubber-tired roller would vary with the soil type, moisture content, and firmness of the foundation; therefore, the relationship of density to depth should be considered as qualitative rather than quantitative.

Figure 6 illustrates hypothetically the intent of construction specifications and proof-rolling requirements in a given layer in a flexible pavement structure. The solid-line curve indicates the compaction effort that will be produced by traffic. This compaction effort is a function of the wheel load, tire pressure, assembly configuration, repetitions, and depth to the layer. These parameters are too complex to consider individually, but studies made at the Waterways Experiment Station indicate that the CBR relations can be used to interrelate these several parameters. Table 1 gives the relationships existing between the percentage of modified AASHO maximum density specified by the Corps of Engineers and the

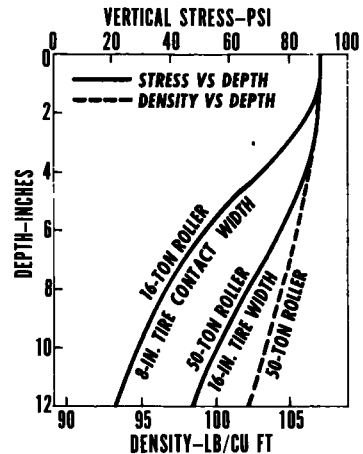


Figure 5. Theoretical vertical stress vs depth.

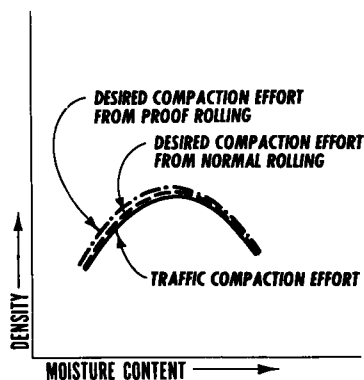


Figure 6. Intent of compaction requirements.

design CBR at the depth to which the specified percentage of compaction is required. Separate values are necessary for cohesive and cohesionless soil. These values were developed for airfield conditions but are probably applicable to highway conditions.

Table 1 shows that cohesive soils should be compacted to 100 percent modified AASHO density down to a depth where the design CBR is 19, to 95 percent density down to a depth where the design CBR is 9, etc. (The value of 100 percent of modified AASHO for cohesive soil is shown primarily to indicate the position of the curve because cohesive soils can seldom be compacted to 100 percent of modified AASHO maximum density.) Table 1 can be used with a CBR design curve for the given loading to establish the compaction that should be obtained to a given layer

during construction. In highway work compaction requirements generally are not varied with depth, and usually one value is specified for the base (for example, 100 percent standard AASHO) and one for the subgrade (for example, 95 percent standard AASHO).

TABLE 1

Compaction Effort Percent of Mod. AASHO Density	Design CBR	
	Cohesive Soils	Cohesionless Soils
100	19	9
95	9	4
90	5	2

With the heavier loads and higher repetitions that are being experienced, it is believed that compaction requirements should be raised, in which case it may prove economical to vary the compaction requirements with depth as is done in airfield work. In any case, it should be the intent of the compaction requirements to produce compaction in each layer during construction equivalent to or slightly higher than will be produced by the actual traffic. The short-dashed curve (Fig. 6) shows the intent of the construction requirements.

For proof-rolling to be effective, it is desirable that it produce a compaction effort which approximates that specified during construction. The dash-dot curve (Fig. 6) illustrates the intent of proof-rolling requirements. Proof-rolling is done with a very few repetitions as compared to highway or airfield traffic. Therefore, the loads and tire pressures must be higher than those which will be imposed on the finished pavement. The compaction effect of the proof-rolling can be compared directly with the compaction effect of traffic by comparing CBR design curves for the roller and the traffic loading. Figure 7 shows CBR curves prepared by the Waterways Experiment Station for a 50-ton roller, 100 psi inflation pressure, for 5 and 30 coverages. The coverage numbers assigned to the curves should not be taken too literally; the lower value should be considered as covering a range of 3 to 8 coverages, the upper value about 20 to 40 coverages. Comparisons of compaction effort can be made directly by plotting the CBR design curve for a specific traffic loading on the diagram. For illustration purposes the Virginia Highway Department CBR design curve (11) for a 20,000-lb wheel load is shown. Even the curve for 100,000-lb 30 coverages lies well above the traffic curve for the 20,000-lb wheel load, indicating that proof-rolling from the surface will not produce the same compaction effect as the traffic. By operating the proof-roller at levels below the finished surface, the same compaction effect in the

underlying layers can be produced as will be produced by traffic operating on the surface. For example, if the design CBR is 5, 30 coverages of proof-rolling at 100,000 lb will produce the same effect as traffic if the proof-roller is operated on a level 4 in. below the finished surface. Similar comparisons can be made for other conditions to select proof-rolling that will approach the compaction effect of full-scale traffic.

EFFECT OF MOISTURE DURING COMPACTION

In actual construction the moisture content cannot be maintained at a precise value, and a range in moisture content will occur. It is desirable that the range be restricted to a zone fairly close to optimum, tending toward the dry side rather than the wet side, (12) because deviations on the wet side may produce a spongy layer which will make compaction of the succeeding layers difficult. Figure 8 shows the desired compaction curve and the desired range of moisture content for compaction for a hypothetical soil. The values of moisture content and density shown on this and succeeding figures are in the general range of those found for lean clay soils. Figure 9 shows the variation of strength with variation in density (6) for materials compacted on the dry side of optimum, at about optimum, and on the wet side of optimum. Specific figures are used in this case to add meaning to the illustration, but the diagrams are schematic. They represent known trends that would be experienced with a lean clay soil. At 13 percent moisture, the CBR immediately

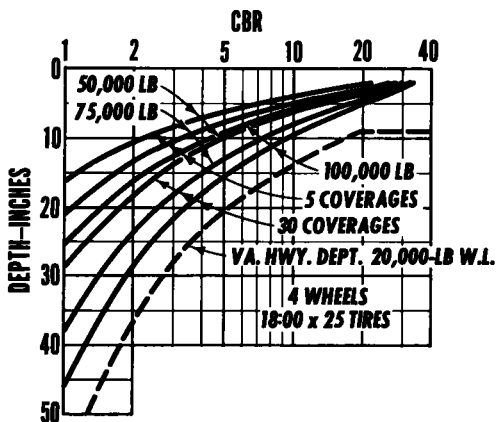


Figure 7. CBR curves rubber-tired roller (50-ton 100-psi tire pressure).

after compaction would be fairly high even at low densities and would be quite high at a density of 105 lb per cu ft (pcf), which is the maximum density that can be obtained from the compaction effort at the percentage of moisture. However, there is a drastic loss in strength when the material takes up moisture. Under conditions similar to those achieved in the soaking tests, the CBR would drop from values of 10 to 20 to values of 1 to 3. At 16 percent moisture, which is approximately optimum, the values immediately after construction would not be quite as high as at 13 percent moisture, but they would be high enough to indicate a firm material. A drop in CBR would also occur in soaking, but the drop would not be as drastic as at 13 percent moisture, and the CBR values after soaking would be in a reasonable range (7 to 10 for a reasonable degree of compaction) after soaking. At 19 percent moisture, the CBR values at less than 104-pcf density would be lower than at the lower moisture contents and there would be little effect of soaking. However, attention is called to the sharp and drastic loss in CBR that would occur with an increase in density (13) beyond a value of about 104 pcf. The decrease would start at about the point where the density crosses the line of optimum.

Figure 9 shows the variation of strength with variation in density (6) for materials compacted on the dry side of optimum, at about optimum, and on the wet side of optimum. Specific figures are used in this case to add meaning to the illustration, but the diagrams are schematic. They represent known trends that would be experienced with a lean clay soil. At 13 percent moisture, the CBR immediately

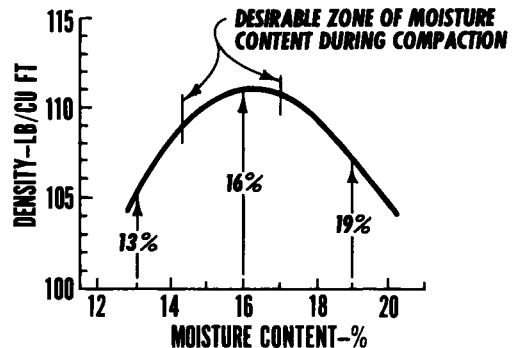


Figure 8. Effect of compaction moisture content.

EFFECT OF PROOF-ROLLING

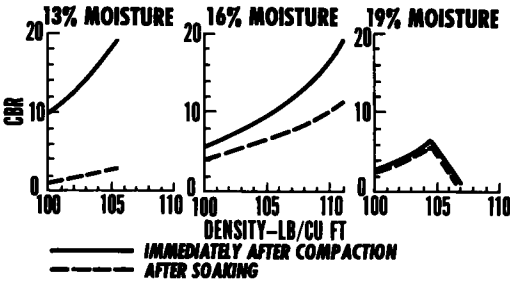


Figure 9.

about at optimum for the compaction effort, the strength will be adequate even after field saturation. In this case, proof-rolling accomplished the intended purpose.

The effect of proof-rolling on a material that is well below optimum (13 percent) moisture during the time that proof-rolling is being accomplished is shown by the circle shown in Figure 11 at 13 percent moisture and 102-pcf density. The density is low because of faulty rolling or for some other reason. The lower moisture content could be the result of the material being placed at a low moisture content or it could be the result of drying between the time of normal compaction and proof-rolling. In cohesionless materials, the time between normal compaction and proof-rolling is very critical because not only does drying occur but a loss of moisture from drainage can also occur. The effect of the proof-rolling in this case is shown by the solid line designated 1 (Fig. 11). Proof-rolling brings the density up to 105 pcf, which is the maximum that can be obtained with that compaction effort. Strength is very high (see plot for 13 percent moisture, Fig. 9) immediately after construction, and the job is apparently one that will stand forever. However, this particular phase of construc-

The first case to be considered is where the moisture content is in the desired range but for some reason the density obtained is well below that specified. This condition (Fig. 10) is illustrated by the circle shown at 16 percent moisture and 102-pcf density. It is assumed that this condition exists just prior to proof-rolling. As proof-rolling is applied, the density increases; and as proof-rolling continues, the density reaches the maximum that can be obtained with the compaction effort. Since the moisture was

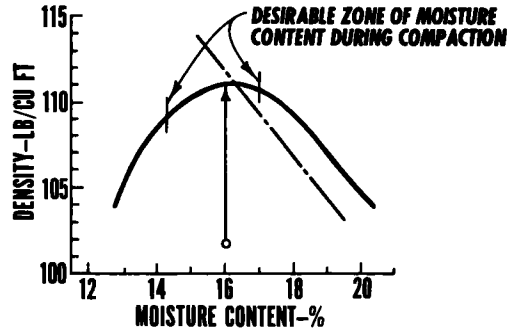


Figure 10. Effect of proof-rolling when moisture content is proper.

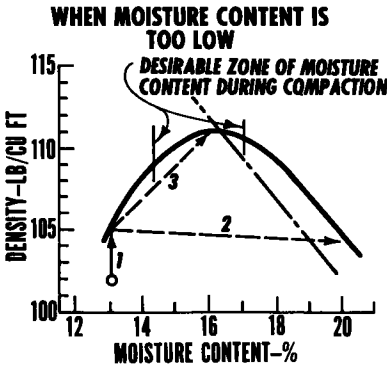


Figure 11. Effect of proof-rolling.

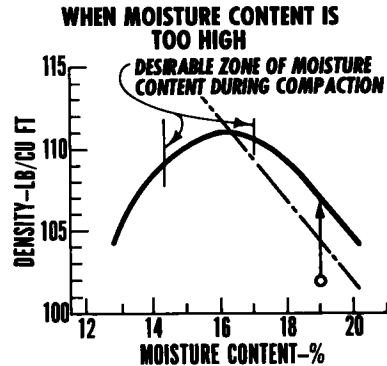


Figure 12. Effect of proof-rolling.

tion may be completed in the late fall, and the road cannot be opened to traffic until the following summer because shoulder work or some other element is not complete. In the meantime, the moisture content increases during the wet winter weather and approaches the compaction curve on the wet side. This change in moisture content is shown by the dashed line labeled 2 (Fig. 11). The line is slanted downward because almost always some swelling will occur upon saturation. A large decrease in strength will occur (see plot for 13 percent moisture, Fig. 9) and the CBR values will be very low, in the order of 1 to 3. With continuous traffic on the pavement, distress in the form of cracking or displacement will develop quickly. In this case, the proof-rolling was wasted and, furthermore it produced a false sense of security.

The case illustrated by the dashed line labeled 3 (Fig. 11) results when the moisture content increases moderately to about optimum while traffic is being applied, with resulting densification. There is a gain in strength as moisture content increases, and the only effect on the structure is settlement in the wheel paths. The density increase is approximately 6 lb, or 5.7 percent. This percentage of densification in a 12-in. layer would produce 0.7-in. settlement, enough to hold water during rains. Again, the proof-rolling did not accomplish the desired results. The deficiencies are not as severe as in the previous case, but the deficiency will require correction.

The circle at 102-pcf density and 19 percent moisture (Fig. 12) shows the effect of proof-rolling on a material when the moisture content is above the desired value for compaction at the time proof-rolling is started. The high moisture content could be caused by the material being placed too wet or by an increase in moisture from rain between the time of normal compaction and proof-rolling or by a ground-water condition. As proof-rolling is applied, the density increases with coverages. By reference to the plot for 19 percent moisture (Fig. 9), it can be seen that at a density 102-pcf density, the strength is fairly low (CBR about 4) but as density increases the strength builds up. However, as the density crosses the line of optimums, there is a drastic loss of strength and the CBR drops to values of 1 to 2. Weaving develops under the roller and cracking occurs in the surface of the layer being rolled. The situation can be corrected by removing the material, drying it out to the proper moisture contents, replacing it, and compacting it properly. Where the moisture increase was due to a ground-water condition, the ground-water condition must be corrected or another design must be employed. In this case, the proof-rolling accomplished something. It located wet material that would subsequently have caused trouble on the road, and in this sense it was not wasted.

Some engineers advocate limiting the proof-rolling so that it will not increase the density of materials on the wet side of the optimum to a point where the decrease in strength occurs (Fig. 13, solid line labeled 1). Proof-rolling increased the density from 102 to 104 pcf, and it was stopped before loss in CBR occurred and weaving developed. Future performance would depend on the rate at which drying occurred and the rate at which traffic was applied. The dashed line labeled 2 (Fig. 13) illustrates the case where no drying occurred before traffic was applied. Traffic would produce densification; and when the density reached the line of optimums, the drastic loss in

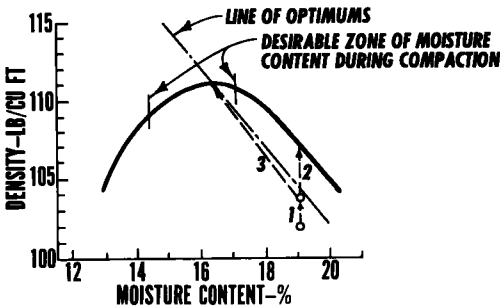


Figure 13. Effect of partial proof-rolling when moisture content is too high.

strength described above would occur and distress from cracking and displacement would develop. If drying occurred at a rate such that the combination of moisture and density did not plot above the line of optimums (dashed line labeled 3, Fig. 13), there would be a gain in strength but settlement would occur which would result in grooving in the wheel paths.

OVERROLLING

Figure 14 shows that moderate overrolling during the proof-rolling procedures is not serious. Overrolling refers to the application of a greater com-

paction effort, through lead and/or repetitions, than is needed. The "normal" curve indicates the compaction effort produced by normal construction rolling, and it is assumed that this duplicates closely the compaction effort that will be produced by traffic on the finished surface. In addition, three other curves represent 5, 30, and 100 coverages with a specified roller. It is recognized that the 100-coverage level is entirely impractical, but it is shown merely as an example of excessive overrolling. Three levels of compaction effort could also be achieved by using three tire pressures or, if substantial depths are being considered, by using 50-, 100-, and 200-ton rollers. As previously demonstrated, the effect produced by the rollers will vary with moisture content. On the dry side, for example 13 percent moisture, the use of 30 or 100 coverages of overrolling would improve the situation and would produce no detrimental effects, whereas the underrolling with 5 coverages would leave much to be desired. At about optimum, 16 percent moisture, the 5-coverage level would leave much to be desired, the 30-coverage level would not be detrimental, but the 100-coverage level would probably push the density far enough above the line of optimum to cause serious loss of strength. Serious overrolling may weaken materials that would not be weakened under traffic. At the high moisture contents, about 19 percent, all three levels of proof-rolling would have about the same effect in that they would cause serious loss of strength.

It appears that excessive overrolling should be avoided both from the standpoint of economics and because excessive overrolling may weaken materials that would not be weakened by traffic. A moderate amount of overrolling would not be detrimental, and it appears desirable to adopt a conservative approach and specify proof-rolling that produces a moderately higher compaction effort than normal rolling.

An exception to the situation described above can occur in a base course or other material which is cohesionless or practically so. Proof-rolling may produce displacement and rutting of the surface of such a material which would not occur under traffic. If the proof-rolling is to be effective, the surface of materials of this type must be stabilized to the extent that it will not displace when rolled.

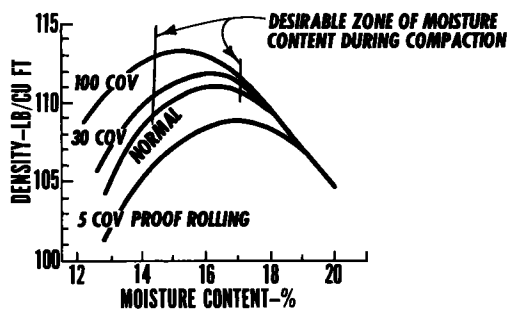


Figure 14. Effect of over-rolling during proof-rolling.

CONTACT PRESSURE VS INFLATION PRESSURE

The true measure of the effect of compaction with a rubber-tired roller is the contact pressure between the tire and the layer on which it operates. Measurement of the contact pressure is fairly difficult. Without electronic pressure gages, the average contact pressure is the only value that can be obtained feasibly. This average value is obtained by jacking up the roller wheel, coating a portion of the tread of the roller tire with mimeograph ink or some similar material, and lowering the tire on a sheet of paper resting on a firm surface. The print on the paper gives the area of tire contact under a static condition. This area is divided into the load to obtain an average contact pressure. Studies made at the Waterways Experiment Station with small electronic pressure gages and those made by others with other types of gages show that the contact pressure between a tire and the layer it is operating on is not uniform, and there are areas where the pressure is considerably higher than the average and also areas where the pressure is lower than the average. The pattern varies considerably with the firmness of the surface and is different for the rolling and for the static condition. The difficulty of measuring the average contact pressure and the variation that occurs in actual contact pressure within the tire print area are such that measurement of the tire print does not appear warranted. Inflation pressure can be measured readily, and this measurement is adequate for construction control purposes.

CONCLUSIONS

The following conclusions covering proof-rolling when properly specified are warranted:

1. If the moisture content is in the proper range for compaction, proof-rolling will correct compaction deficiencies.
2. If the moisture content is well on the wet side of optimum, proof-rolling will locate this condition and thus permit correction of the condition during construction.
3. If the moisture content is on the dry side of the proper range for compaction, proof-rolling gives a false sense of security, because the layer looks firm and hard; but as moisture increases, the layer will either lose strength drastically or will compact further under traffic.
4. Moderate overrolling during proof-rolling will not be detrimental, but excessive overrolling should not be applied since it may weaken materials that would not be weakened by traffic.

Based on these considerations, it should be apparent that proof-rolling is not a panacea to end all compaction problems and replace inspection. Proof-rolling is an excellent tool that will assist in overcoming compaction deficiencies and will locate wet materials, but it cannot replace good inspection and proper construction-control testing.

ACKNOWLEDGMENTS

This paper is based primarily on data collected by the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, as a part of the studies on development of flexible pavement design criteria for Air Force and Army airfields. Acknowledgment is also made to Edward C. Cothran, Executive Director, Carolina Asphalt Association, for suggesting this paper.

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Discussion

W. H. CAMPEN AND L. G. ERICKSON, *Omaha Testing Laboratories*—In the first part of the paper the authors have shown the relationships of density and moisture with strength in a very clear manner. In particular they have shown that the ultimate strength or CBR value is controlled principally by the amount of water which can be taken up by the soil; the amount of water that can be taken up depends on the voids, which in turn are controlled by density. Therefore, in order to evaluate dependable strength, the soil should be compacted to specified density at its corresponding moisture content on the wet side of the curve. For instance, if it is desired to place a given soil at 95 percent maximum density and the moisture-density curve shows a water content of 19 percent at this point, on the wet side, the CBR should be determined under these conditions.

The second part of the paper deals with proof-rolling. Foster and Turnbull say in effect that proof-rolling can be used to accomplish either of two things: additional densification or the detection of weak or soft areas. The writers are in agreement on these points, but wish to point out that the actual application is very difficult.

To begin with, the tire pressure and total load should be varied not only in accordance with the load carrying capacity of the total layered system under construction, but also for each layer of the system. No doubt this can be done, but it will require a great deal of correlation. The authors indicate that these factors should be considered but no details are included in the paper.

As far as additional densification is concerned, it seems that the pneumatic-tired rollers might be used in conjunction with sheepsfoot or vibrating steel rollers as the layered system is constructed from the bottom up. However, it is believed that for highway construction the sheepsfoot roller (for plastic soils) and the vibrating steel rollers (for cohesionless soils) should suffice. There is an airfield constructed under the direction of the Corps of Engineers where 105 percent of maximum density, by the modified method, was obtained by the use of vibrating rollers.

Concerning the use of heavy pneumatic-tired rollers for the detection of weak or soft areas; it is believed that its possibilities are good provided that the sizes of rollers are commensurate with the strength of the layers on which they are applied. To repeat, the size of the roller should be such as not to overload the layer being tested. Incidentally, experience has shown that heavy sheepsfoot and steel rollers, patrols, and loaded trucks will detect weak subgrades if the layers being constructed are at optimum moisture. In making such tests the criterion should be the amount of deflection. Heavily loaded trucks have been used since 1936 to test natural subgrade and superimposed layers.

CLOSURE, W. J. Turnbull and Charles R. Foster—The discussion is appreciated, and the authors find no significant difference of opinion but would like to call attention to the following items:

The writers' first paragraph states "they have shown that the ultimate strength or CBR value is controlled principally by the amount of water which can be taken up by the soil." This is an over-simplification of the problem. The future strength will be influenced as much by the molding moisture content as by the amount of moisture taken up by the soil. The authors feel that the effect of moisture content at the time of compaction on future strength has not been given the attention it deserves in soil compaction.

The authors object mildly to rewording statements to infer "that proof-rolling can be used to accomplish either of two things: additional densification or the detection of weak or soft areas." The first conclusion states that proof-rolling will correct com-

paction deficiencies; that is, produce additional densification only if the moisture content is in the proper range during the period of time the proof-rolling is being applied. The detection of layers deficient in strength should, of course, be accomplished with a roller which will not overload the soil.

It is agreed that the tire pressure and total load should be varied in accordance with each layer, and the authors proposed a method of determining the necessary tire pressure, load, and number of repetitions (the latter a variable that the writers failed to mention) in the paper. Figure 7 of the paper illustrates the proposed method.

There is complete agreement that density should be obtained during construction rather than waiting and attempting to obtain it with proof-rolling. In fact the very purpose of the paper is an attempt to show "that proof-rolling is not a panacea to end all compaction problems and replace inspection."

Hydraulic Fill Compaction

OLAF STOKSTAD and KENT ALLEMEIER, Michigan State Highway Department

The subject of fill compaction has rightly received special emphasis in connection with the improvement of pavement smoothness. The problem of building fills over deep peat marshes presents conditions which defeat conventional compaction efforts.

This paper deals with the use of flowing water as a compacting and stabilizing agent. The technique described involves introducing water into the "end dump" fill by means of jets to develop a temporary condition of flotation (quicksand) progressively along the length of the marsh fill.

The procedure described obtains embankment movement and settlement during jetting operations, which would otherwise take place over a period of years.

● THE LAST 25 years have seen great progress in the development of techniques for embankment compaction. This development includes both equipment for compacting work and the yardstick necessary for measuring the degree of compaction. As a result of the evolution in compaction testing, it has become possible to specify results to be obtained rather than methods to be used in embankment construction.

Nevertheless, some problems remain. One of these stems from conditions which surround the building of embankments across deep deposits of peat and other marsh materials. Deep peats (more than 10 or 15 ft) never compact within reasonable time limits to the point where overlying embankments will stop settling. If such movement cannot be tolerated, it becomes necessary to improve the situation by sinking the fill to the marsh bottom. This operation is greatly facilitated by loading the peat to the point of failure, which is generally accomplished by end-dumping the embankment material to some surcharge grade as the filling progresses across the marsh (Fig. 1).

This technique of fill construction does not permit the proper mechanical compaction of the fill as it is being placed. Thus, road settlement may be experienced in the deep marsh crossings even though complete peat displacement has been obtained. The slow and continuing fill compaction with associated volume change, which takes place over the years following construction becomes a threat to traffic safety and the source of an expensive maintenance nuisance.

In geographic areas characterized by numerous marsh deposits, the problem of marsh fill compaction has received much thought. Acceleration of the compaction process has been accomplished by procedures involving vibration or the use of water. Michigan's experience with vibration has been limited to a technique employing explosives as a source of energy. Generally, the use of water in a fill jetting operation has proved to be faster, more flexible, and more effective. This discussion, therefore, is limited to a description of jetting procedures (hydraulic fill compaction) which are serving greatly to reduce a serious design, construction, and maintenance problem in Michigan—a problem which has become especially acute with greater earth moving capacity and the exploitation of this capacity in connection with an accelerated building program.

Historically, the jetting of marsh fills is an outgrowth of an attempt to soften peat trapped below an embankment and thus facilitate peat displacement. In this procedure, an unreasonable time lag was experienced between the beginning of pumping operations and the beginning of accelerated fill settlement. A study of this phenomenon revealed that settlement action did not begin until the condition of saturation had extended into the fill above the peat. With jets placed in the peat, the water tended to cut channels through the peat and escape before saturating either the peat or fill. Lifting the jets out of the underlying peat so as to pump water directly into the embankment brought

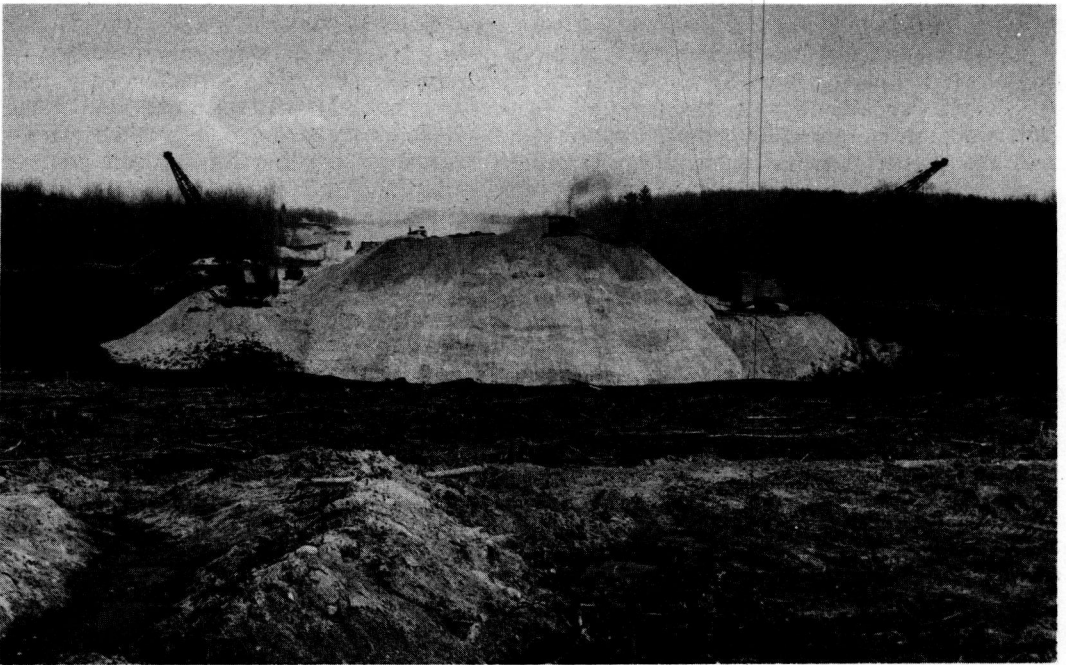
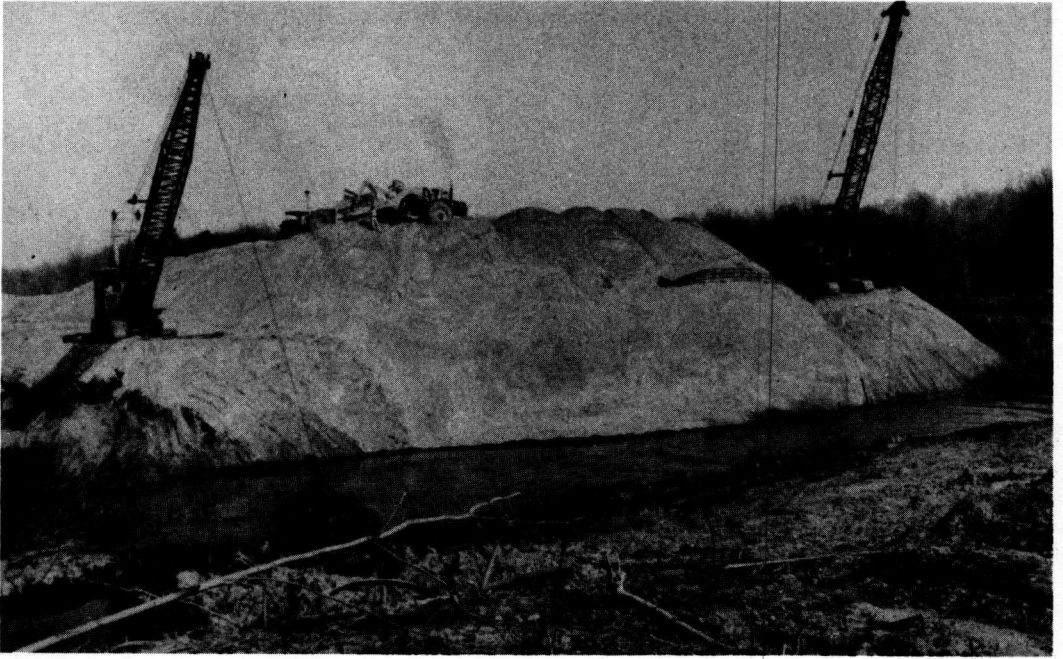


Figure 1. Peat displacement and embankment construction to surcharge grade by end dump method before treatment by hydraulic fill compaction (marsh 45-ft deep).



Figure 2. Extensive settlement cracking.



Figure 3. Settlement cracking and jet spacing.

almost immediate action. Thus, hydraulic fill compaction may be described as a procedure to compact and stabilize deep marsh fills through the action of flowing water introduced into the fill under pressure by means of a system of jets. The objective is accomplished mainly by developing a temporary condition of flotation (quicksand) in the granular fill progressively along the length of the entire marsh embankment. The procedure appears to produce improved embankment stability through a twofold influence: first, the flowing water seems to rearrange soil grains into a somewhat more compact arrangement, and second, the temporary "quick" condition greatly reduces the tendency of a granular fill to bridge or arch over areas of deficient embankment

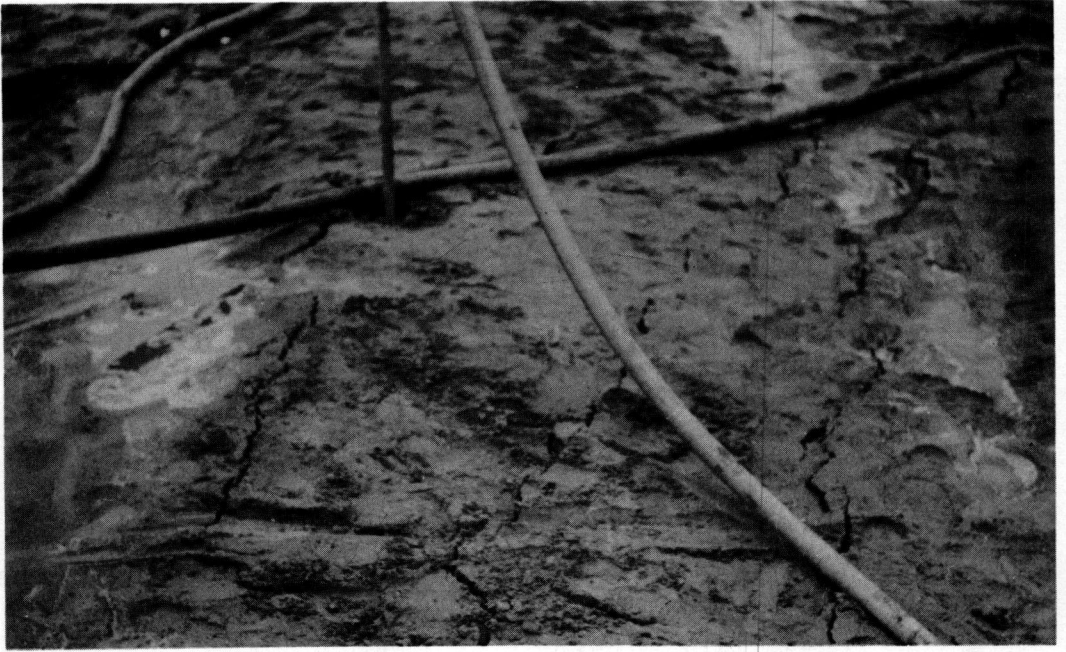


Figure 4. Crack pattern showing readjustment of embankment.



Figure 5. Jet pattern and distribution setup.

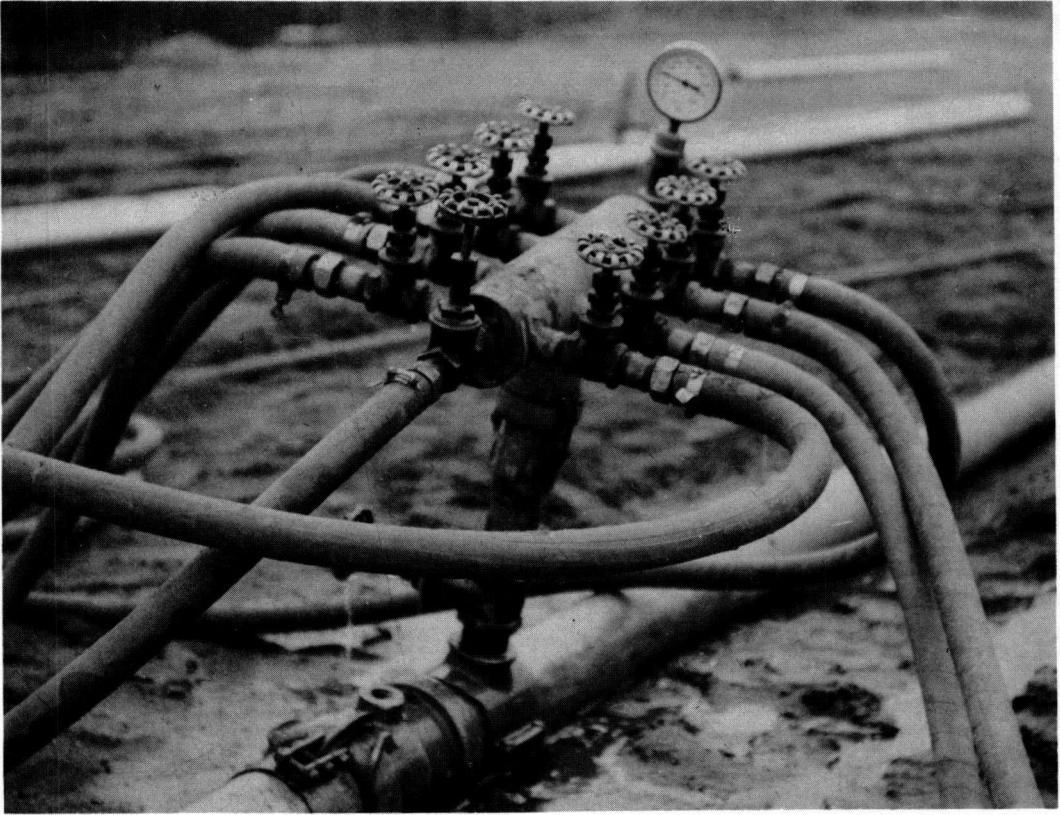


Figure 6. Typical tee with pressure gage.



Figure 7. Truck-mounted 1,000-gpm pump.

support. The first action results in added compaction; the second results in additional peat displacement and a more stable fill cross-section.

The jetting procedure has three serious limitations:

1. It demands that a substantial supply of water be conveniently available. A well which might be completely ample for a farmstead will not provide the rate of flow needed for a marsh fill stabilizing project. Likewise, a small pond is quickly pumped dry. A small stream capable of supplying at least 1,000 gal per minute will serve. This dependence on an adequate source of water would be a serious handicap except for a recent development in pipelines designed for irrigation purposes. A lightweight aluminum pipe with a very effective snap-on coupling arrangement used in this field permits tapping of water sources as much as two miles distant.
2. Hydraulic fill compaction requires the use of soil materials which permits a fairly free movement of water through the embankment. Free draining sands are the most suitable. Clay soils are definitely not suitable. Fill compaction by water, therefore, cannot be used except in locations where sand or sand and gravel are available for embankment construction.
3. Hydraulic compaction of fills cannot be considered equivalent to methods where layer construction permits proper compaction as filling progresses. Working with marsh fills often involves sand bodies more than 100 ft wide and 75 ft deep, as compared to the units of 6- or 8-in. layers tested in conventional compaction control operations. As yet, no attempt has been made to check the amount of compaction actually obtained by pumping water into a large embankment of sand. The evidence of fill compaction is visual and consists of simple subsidence or of shear failures as illustrated in Figures 2, 3, and 4. Thus, embankment stability and compaction can be roughly determined by observing fill behavior and can be controlled by varying the position of jet points, by timing of jet operations, and by varying the volume of water pumped.

Experience has shown that no two embankments react exactly the same. For that reason, the operation requires the good judgment of an experienced supervisor to obtain satisfactory results without costly and sometimes dangerous slides.

A properly planned hydraulic compaction project infers proper preliminary studies of the marsh, beginning with a classification of marsh materials. After fill materials have been placed and before hydraulic compaction operations begin, it is essential to obtain cross-sections of the embankment to determine its position with respect to the marsh bottom. Such sections are needed at 100-ft intervals and sometimes at 50-ft intervals if the marsh is short or has an irregular bottom. These sections are determined by wash borings.

NORMAL OPERATING PROCEDURES

Jets are spaced at 10-ft intervals, both parallel and perpendicular to the centerline (Figs. 2, 3, and 5). The embankment is jetted from shoulder line to shoulder line with occasional jets placed in the slope if original borings indicate that this is desirable. The jets consist of $\frac{3}{4}$ -in. extra-strength pipe cut into 10-ft lengths, and connected to the water line by 1-in. high-pressure rubber hose. A gage valve is incorporated in each hose connection to permit absolute control of each individual jet.

The jets, operating at a pressure of from 50 to 75 lb per sq in., are forced to within 5 ft of the bottom of the fill material. They are left at this elevation only for the period of time required to inject a volume of water previously calculated to be sufficient to saturate the fill material. The volume may be controlled by a water meter. At the end of this time, they are lifted 5 ft and left at this elevation as before. The operation is repeated until the jets are at the surface and the fill is completely saturated. The normal pumping time per 5 ft of fill depth averages approximately 30 minutes. This time varies depending on the texture of the fill material and the number of jets operating. Medium to fine sands require more pumping time than more coarse textured materials. Where a stony fill material has been used, penetration of the jets is often facilitated by preboring with a truck-mounted power auger.

Marsh areas having inclined clay bottoms may be subject to serious embankment slides if the jetting process is used. Consequently, this type of marsh requires that jetting operations be given special study and be conducted so as to keep the fill under control at all times.

An average jetting crew consists of approximately 15 laborers, hired locally by the road contractor, in addition to the pump operator and the supervisor. A crew of this size will operate from 35 to 40 jets simultaneously.

Cost records show that jetting costs for a modern dual-lane embankment (94 ft from center to center) ranges between \$500 to \$1,000 per station (100 ft), depending upon the depth of marsh, height of fill, and texture of the back fill material. From 35 to 100 ft of dual embankment can be jetted in a 9-hr day.

CONCLUSIONS

Experience with hydraulic fill compaction demonstrates that pumping water into a peat marsh fill under controlled conditions has an immediate stabilizing effect on the embankment. The adjustments and movements obtained would otherwise take place slowly over a period of years and result in detrimental pavement settlement. Because of the success of the operation and because of an expanded road program, the Michigan State Highway Department has added a second set of jetting equipment. The additional equipment will permit the treatment of all end-dump marsh fills of 10 ft or more in depth.

Rapid Determination of Liquid Limit of Soils by Flow Index Method

H. Y. FANG, Soils Engineer, AASHO Road Test, Highway Research Board

This paper presents a new approach to liquid limit determinations by a one-point procedure. Necessary equations, charts and tables, developed on the AASHO Road Test and easily usable by technicians, are presented.

● THE ROUTINE testing of large numbers of soil samples in highway construction and maintenance requires the use of rapid methods for determining standard characteristics. The liquid limit determination for soils is one of the most important of these tests.

The standard method for determining the liquid limit of soils (1) requires that at least three random trials be made, each at a different moisture content. The moisture contents are plotted as the ordinates on an arithmetical scale and the number of blows as abscissae on a logarithmic scale. A flow curve is then drawn and the liquid limit is taken as the moisture content corresponding to the point where the 25 blow line intersects the flow curve.

The more rapid one-point procedure for determining the liquid limit has been given considerable attention in recent years, and the American Association of State Highway Officials has adopted it as an alternate test procedure. It was initiated by the Corps of Engineers (2) and later extended by the Washington State Highway Department (3) and the Bureau of Public Roads (4).

According to the Corps of Engineers method, it was assumed that the flow curve plotted on the logarithmic paper was a straight line and could be expressed by

$$LL = W_n \left(\frac{N}{25} \right)^{\tan B}$$

in which LL = liquid limit;

N = number of blows;

W_n = moisture content at N blows; and

$\tan B$ = slope of the flow curve when plotted on logarithmic paper.

The Washington State Highway Department method was based upon relationships between the different slopes of flow curves obtained for soils with different liquid limit test values. This was called the "Washington Chart Method."

The Bureau of Public Roads method was derived from a family of typical flow curves (from the Washington State Highway Department), and made use of the assumption that the convergence of the flow curves with the zero moisture content axis was a constant. This assumption made it possible to derive the following equation for such a family of curves:

$$\frac{LL}{W_n} = \frac{\log N_a - \log 25}{\log N_a - \log N}$$

N_a is the point of convergence of a family of flow curves intersecting on the zero moisture content axis. The problem was further simplified by arbitrarily moving the point of convergence to the zero moisture content axis. This point was so far from the 25 blow line that the consequent changes in the slopes of the flow curves were negligible within the 17 to 36 blow range used.

This paper presents another approach based on work done on the AASHO Road Test, and making use of published data from the Washington State Highway Department.

In Figure 1, the flow index is defined as the slope of the flow curve. It can be expressed by the formula:

$$\text{Flow Index } (I_f) = \frac{LL - W_n}{\log N - \log 25}$$

From this relationship the liquid limit may be determined by the following equation:

$$LL = W_n + I_f \log \frac{N}{25}$$

The term $I_f \log \frac{N}{25}$ is called the moisture correction factor. If N is less than 25, the value of this term will give a negative sign. Table 1 shows the moisture correction factor computed for various flow indexes, and Figure 2 shows the moisture correction factor versus number of blows for various flow indexes.

Effect of variations in the flow index on the value of the liquid limit can be measured by the method of differentials. The equation immediately above can be ex-

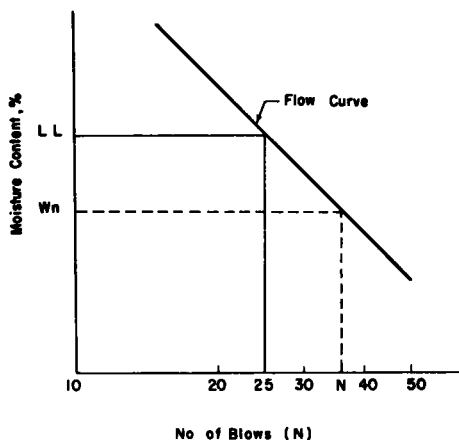


Figure 1. Flow curve on semi-logarithmic plots.

pressed in differential form as:

$$\Delta LL = \log \frac{N}{25} \Delta I_f$$

in which ΔLL is the percent of moisture change in the value of liquid limit for changes in ΔI_f . Calculations of ΔLL for various N and ΔI_f are shown in Figure 3.

Indications from Figure 3 are that the maximum error in liquid limit associated with an error 2.5 in flow index would be 0.4 percentage points, if the number of blows in the one-point test was between 17 and 36.

Figure 4 shows the flow index—liquid limit relationship plotted from data obtained from the Washington State Highway

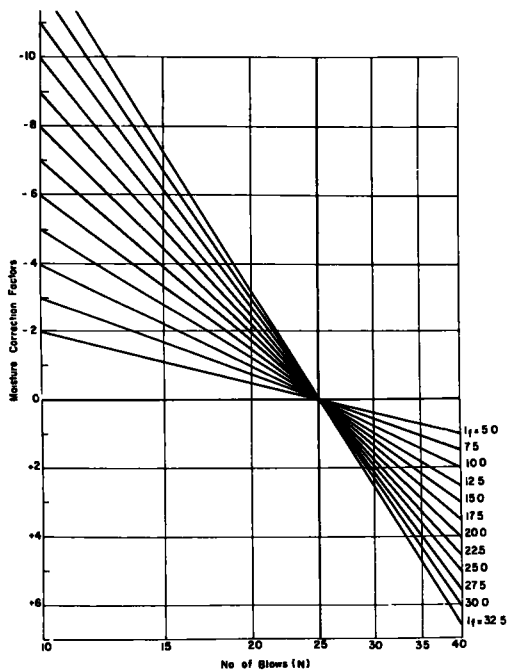


Figure 2. Moisture correction factors vs. No. of blows for various flow indexes.

TABLE 1
MOISTURE CORRECTION FACTOR FOR VARIOUS FLOW INDEXES

* Moisture Correction Factor = $I_f \text{Log} \frac{N}{25}$												
N \ I _f	5.0	7.5	10.0	12.5	15.0	17.5	20.0	22.5	25.0	27.5	30.0	32.5
17	0.84	1.25	1.67	2.09	2.50	2.92	3.34	3.76	4.18	4.59	5.01	5.43
18	0.72	1.07	1.43	1.79	2.14	2.50	2.86	3.22	3.58	3.93	4.29	4.65
19	0.60	0.89	1.19	1.49	1.78	2.08	2.38	2.68	2.98	3.27	3.57	3.87
20	0.48	0.73	0.97	1.21	1.46	1.70	1.94	2.18	2.42	2.67	2.91	3.15
21	0.38	0.57	0.76	0.95	1.14	1.33	1.52	1.71	1.90	2.09	2.28	2.47
22	0.28	0.41	0.55	0.69	0.82	0.96	1.10	1.24	1.38	1.51	1.65	1.79
23	0.18	0.27	0.36	0.45	0.54	0.63	0.72	0.81	0.90	0.99	1.08	1.17
24	0.09	0.14	0.18	0.22	0.27	0.32	0.36	0.40	0.45	0.50	0.54	0.58
26	0.08	0.13	0.17	0.21	0.26	0.30	0.34	0.38	0.42	0.47	0.51	0.55
27	0.16	0.25	0.33	0.41	0.50	0.58	0.66	0.74	0.82	0.91	0.99	1.07
28	0.24	0.37	0.49	0.61	0.74	0.86	0.98	1.10	1.22	1.35	1.47	1.59
29	0.32	0.48	0.64	0.80	0.96	1.12	1.28	1.44	1.60	1.76	1.92	2.08
30	0.40	0.59	0.79	0.99	1.18	1.38	1.58	1.78	1.98	2.17	2.37	2.57
31	0.47	0.70	0.93	1.16	1.40	1.63	1.86	2.09	2.33	2.56	2.79	3.02
32	0.54	0.80	1.07	1.34	1.61	1.87	2.14	2.41	2.68	2.94	3.21	3.48
33	0.61	0.91	1.21	1.51	1.82	2.12	2.42	2.72	3.03	3.33	3.63	3.93
34	0.67	1.01	1.34	1.68	2.01	2.35	2.68	3.02	3.35	3.69	4.02	4.36
35	0.72	1.08	1.44	1.80	2.16	2.52	2.88	3.24	3.60	3.96	4.32	4.68
36	0.79	1.19	1.58	1.98	2.37	2.77	3.16	3.56	3.95	4.35	4.74	5.14

* The moisture correction factors for number of blows (N) less than 25 should be preceded by a minus (-) sign.
If the number of blows is above 25, the moisture correction factors should be plus (+).

Department (3) and the AASHTO Road Test. From this relationship, it is indicated that the flow index increases uniformly with the liquid limit. (Organic soils are not included.)

Table 2 has been prepared from the data shown in Figure 4 for flow index intervals of 2.5, and may be used with Table 1 to compute liquid limit values.

Using the procedures outlined in this paper, laboratory personnel may easily and accurately estimate liquid limit values from one-point test data. Proper flow index values corresponding to the moisture content (W_n) coincident to the number of blows

used in the one-point test are selected from Table 2, provided the number of blows is not less than 17 nor greater than 36. The liquid limit value is determined by applying the proper correction factor from Table 1 to the moisture content obtained in the one-point test.

It is believed that this procedure is an improvement over previously used methods in that it is derived directly from the flow curve and the observed relationship between the slope of the flow curve and the liquid limit for various soils. These relationships are simple and allow use of the procedure by the laboratory technician through the

TABLE 2
SELECTION OF FLOW INDEX

One pt moisture cont. (range of no. of blow between 17 to 36) (W_n)	Flow Index (I_f)
20.0 — 25.0	5.0
25.0 — 30.0	7.5
30.0 — 37.5	10.0
37.5 — 45.0	12.5
45.0 — 52.5	15.0
52.5 — 60.0	17.5
60.0 — 67.5	20.0
67.5 — 75.0	22.5
75.0 — 82.5	25.0
82.5 — 90.0	27.5
90.0 — 97.5	30.0
97.5 — 105.0	32.5

Equation : $L.L. = W_n + I_f \text{Log} \frac{N}{25}$

Example :

Given : a soil with a moisture content (W_n) = 50 % and number of blows (N) = 20.

From table 2, the selection of flow index (I_f) = 15

From table 1, the moisture correction factor = - 1.46

Therefore, the liquid limit (L.L.) = $50 - 1.46 = 48.54$.

application of simple correction factors presented in tabular form.

ACKNOWLEDGMENTS

The writer wishes to express sincere appreciation to W.N. Carey, Jr., Chief Engineer for Research, and J.F. Shook, Materials Engineer, AASHO Road Test, Highway Research Board, for helpful suggestions on the paper.

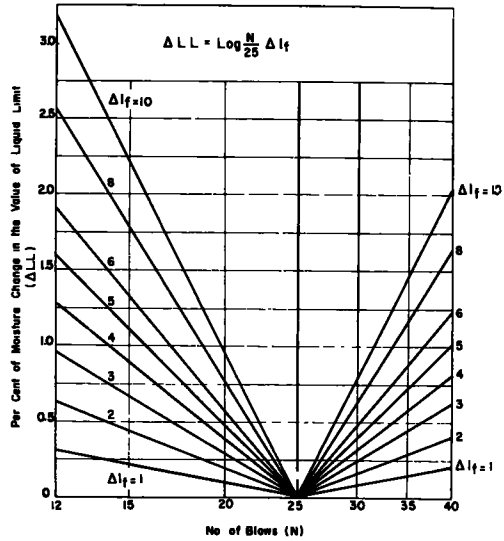


Figure 3. Percent of moisture change in the value of liquid limit vs. No. of blows for change in flow index.

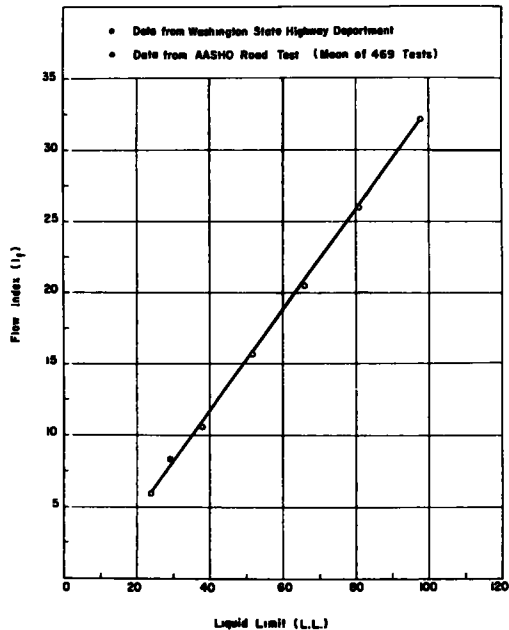


Figure 4. Flow index vs. liquid limit.

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Discussion

C. M. JOHNSTON, Soils Research Engineer, Bureau of Public Roads (Retired)—Mr. Fang's paper is well written. His procedure is well thought out and technically correct. He has added to the knowledge of the soil testing profession by giving us a tabular method for calculating the liquid limit, to add to the three methods listed in the American Association of State Highway Officials Alternate Standard Method for Liquid Limit of Soils, that is, the nomographic chart, the multicurve chart and the slide rule with special "blows" scale. His method appears to qualify for the AASHO Alternate Method, since the last clause in the section regarding calculations reads, "or by any other method which produces equally accurate liquid limit values." The author includes proof that the maximum variation to be expected by his procedure is 0.4 percentage unit, which is as accurate as any of the three accepted methods.

It may be that his method was an "improvement over previously used methods" for the soils and personnel on the AASHO Test Road, but for general use the writer prefers the slide-rule method and would use the tabular one only for personnel who found the slide rule difficult.

For the record, the Washington State Highway Department engineers did not know of the existence of the Corps of Engineers' method when they were developing their method. Neither did the Soils Research Laboratory of the Bureau of Public Roads, as the writer was in charge of the investigation of the Washington State method and the report of the slide-rule method. The Arlington Research Station of the Bureau of Public Roads first learned of the Corps of Engineers' method in August 1956, when the Denver Division office forwarded a copy of the nomograph with a reference to its source. At that time the Research Laboratory had been using the slide-rule method for five years.

CLOSURE, H. Y. Fang—The comments and review of the literatures contributed by Mr. Johnston are much appreciated.

The selection of the flow index using Table 2 gives a maximum possible deviation of 2.5 in the flow index and a deviation of about 0.4 points in the value of liquid limit. This may be somewhat too large an error in the liquid limit determination. To reduce the error, an expansion of Table 2 for selection of the flow index is needed. Since Table 2 has been prepared from the data shown in Figure 4, it is possible to use the following relations for determining the proper flow index with more precision:

$$I_f = 0.36 Wn - 3$$

When the above relationship is used, the deviation of flow index will be reduced from 2.5 to 1.

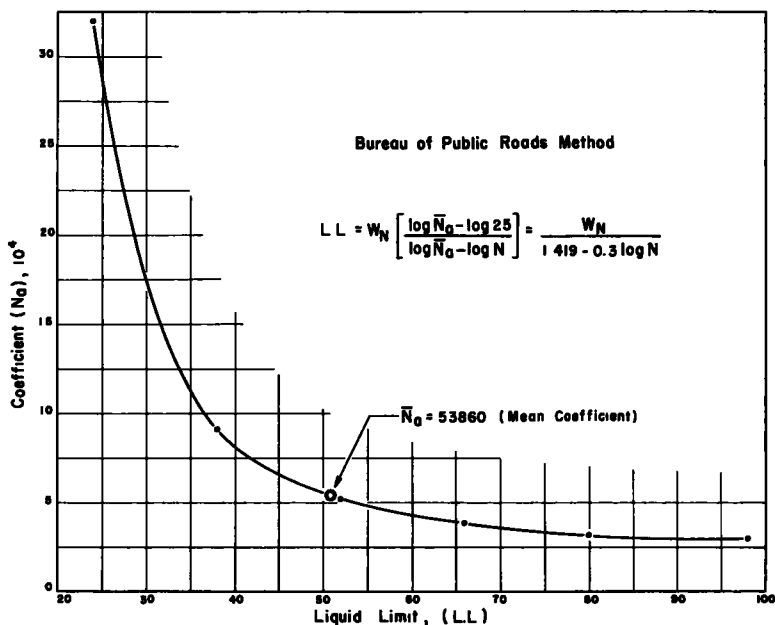


Figure 5. Coefficient (N_a) vs. liquid limit.

Using the published data from the Washington State Highway Department (3) for the evaluation of the Bureau of Public Roads Method, a relationship between coefficient (N_a) and liquid limit was found (shown in Figure 5). It is indicated that the coefficient (N_a) is very sensitive when the liquid limit is small (say lower than 35 or 40). On the other hand, when the liquid limit is larger than 40, the coefficient (N_a) has little effect. Therefore, the use of coefficient (N_a) as a measurement for the liquid limit is limited.

Since the mean coefficient (shown in Figure 5 $\bar{N}_a = 53860$) is used in the Bureau of Public Roads method, the deviation from mean to the true coefficient will be markedly increased when the liquid limit is below 40. Many of the embankment soils used in highway construction and maintenance are classified as A-4 and A-6, with a maximum liquid limit of 40. For these reasons, from a practical point of view, the writer believed that the AASHTO Road Test method is an improvement over previously used methods.

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