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SECTION OF MOUNT VERNON MEMORIAL HIGHWAY ON COLUMBIA ISLAND, D. C.

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

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STABILIZATION BY DRAINAGE OF MUCK AND SAND FILL

Reported by C. A. HOGENTOGLER, Senior Highway Engineer, and E. A. WILLIS, Assistant Highway Engineer, Division of Tests, United States Bureau of Public Roads

COLUMBIA ISLAND, on which a section of the Mount Vernon Memorial Highway, 4,000 feet in length, has been constructed, is a body of land which was built up from the waste material resulting from the hydraulic dredging operations performed in deepening the Potomac River Channel between the Highway Bridge and Georgetown, D. C. In many places the material pumped in was extremely unstable and, in the condition in which it was encountered on the island, was unfit for use as a highway subgrade. The measures employed in improving the stability of those portions of the grade which were unsatisfactory furnish excellent examples of some of the fundamental principles of highway drainage and illustrate the behavior of a class of soil frequently encountered in highway construction.

At the time it was desired to start grading operations on Columbia Island a section about 300 feet in length between stations 10 + 00 and 13 + 00 (see fig. 1) was covered with water and was so unstable that a person attempting to walk along the center line would sink in to his knees.

FIGURE 1.—LOOKING NORTH FROM STATION 13+00, ALONG THE CENTER LINE, BEFORE DRAINAGE

The remainder of the grade, between stations 13 + 00 and 50 + 00, was perfectly dry on the surface and was firm underfoot. It was, consequently, assumed to be in a stable condition and equipment was moved in. This step revealed the presence of soft undersoil in many places.

The power shovel was bogged down in several instances (see fig. 2) even though it was traveling over timber mats and the trucks soon broke through the harder surface crust. It was evident that stabilization would have to be effected before construction could be attempted.

PERMEABLE SAND AND IMPERMEABLE MUCK COMPOSE SOIL OF ISLAND

Preliminary observations revealed that, in general, the island consists of only two distinct types of soil. One is a clean, permeable river sand. The other is a silty river bottom muck, almost impermeable in a compact state but having relatively high shrinkage properties which, under the action of evaporation, produced large cracks and fissures which would act as water carriers. The character of this muck is shown in Figure 3. Mixtures of the two soils were, of course, encountered.

FIGURE 2.—WET AREA AT STATION 20+00. POWER SHOVEL WAS STUCK AT THIS POINT

Table 1 gives the mechanical analysis and the subgrade soil constants of each of the two soils. The nature of these test constants and the methods of obtaining them are discussed in a paper entitled, "Subgrade Soil Constants, their Significance, and their Application in Practice," Part I, PUBLIC ROADS, vol. 12, No. 4, June, 1931; and Parts II and III, PUBLIC ROADS, vol. 12, No. 5, July, 1931. The classification of subgrade soils according to their characteristics into groups A–1 to A–8 is also discussed in these articles.

The test results shown in Table 1 indicate that the sand found on Columbia Island has the properties of the A–3 subgrade group. The muck is an A–8 subgrade. The significance of these groupings may be understood from the following quotations: [1]

Group A–3.—Coarse material only, no binder. Lacks stability under wheel loads but is unaffected by moisture conditions. Not likely to heave because of frost nor to shrink or expand in appreciable amount. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements.

Group A–8.—Very soft peat and muck incapable of supporting a road surface without being previously compacted.

[1] PUBLIC ROADS, vol. 12, No. 4, June, 1931, p. 105.

TABLE 1.—*Analysis of soils*

MECHANICAL ANALYSIS

Soil	Percentage of particles				
	Coarse sand 2.0 mm to 0.25 mm	Fine sand 0.25 mm to 0.05 mm	Silt 0.05 mm to 0.005 mm	Clay smaller than 0.005 mm	Colloids smaller than 0.001 mm
Sand	39	51	8	2	1
Muck	1	4	59	36	11

TEST CONSTANTS

Soil	Liquid limit	Plasticity index	Shrinkage		Moisture equivalent		Specific gravity
			Limit	Ratio	Centrifuge	Field	
	Per cent	Per cent	Per cent		Per cent	Per cent	
Sand	21	0	------	------	10	21	2.65
Muck	56	25	[1] 34	1.4	[2] 64	44	2.70

[1] Shrinkage limit of muck in undisturbed state is 43. [2] Water-logged.

The values of the subgrade soil constants characteristic of these two groups are stated as follows: [2]

Group A–3.—Grading: Effective size not likely to be less than 0.10 millimeter.

Constants: Liquid limit not appreciably greater than 35; no plasticity index; no significant shrinkage limit; centrifuge moisture equivalent less than 12.

Ability of sands to resist sliding when wet indicated as follows: Liquid limits of 10 to 14 signify beach and other rounded sands which slide easily; liquid limits of 30 to 35 indicate rough angular particles which do not slide easily. In addition, liquid limits when lower than field mositure equivalents indicate materials which flow under partial saturation; when equal to the field moisture equivalents, the liquid limits indicate average sands which flow under full hydrostatic uplift. Liquid limits greater than field moisture equivalents indicate rough-grained sands which flow only when in a state less consolidated than that represented by the field moisture equivalent.

Group A–8.—Grading: Not significant.

Constants: Liquid limit greater than 45; plasticity index generally less than those indicated by curve 3; shrinkage limit indicated approximately by curve 6; centrifuge moisture equivalent between curves 9 and 10; field moisture equivalent likely to be greater than those indicated by curve 12.

Water-logging in the centrifuge test is characteristic of the mucks containing clay and colloids, whereas very high equivalents without water-logging are characteristic of peat not more than slightly decomposed.

The curves referred to in this quotation are those of the soil identification chart, which is shown in PUBLIC ROADS, vol. 12, No. 5, July, 1931, page 135. The equations of the curves are given on page 137.

Table 1 shows that the sand has a liquid limit of 21 which is about the value possessed by the great majority of sands. A lack of the soil constituents productive of cohesion and shrinkage is indicated by the fact that the sand has no plasticity index and no significant shrinkage limit. Since this material has no significant shrinkage limit it will not shrink or expand with varying moisture contents which do not materially exceed the liquid limit, and field moisture equivalent values of 21. Such excessive moisture contents are not ordinarily encountered in the field.

The liquid limit and the field moisture equivalent of the island sand are the same. This further indicates the average character of the sand, which will flow only when completely saturated.

[2] PUBLIC ROADS, Vol. 12, No. 5, July, 1931, pp. 136 and 137.

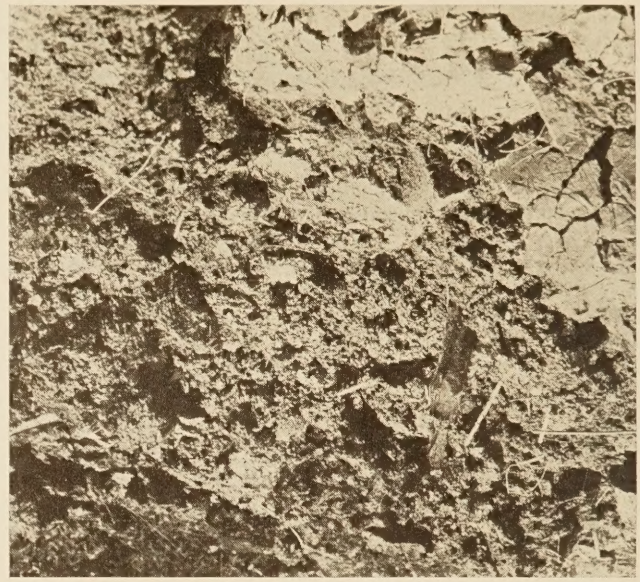


FIGURE 3.—CHARACTER OF MUCK ENCOUNTERED. MATERIAL AT THIS SPOT DISTURBED WITH DYNAMITE IN EFFORT TO HASTEN DOWNWARD PERCOLATION INTO UNDERLYING SAND LAYER

The constants for the muck shown in Table 1 agree with those previously specified for soils of the A–8 group containing colloids. For a liquid limit of 56 a soil should have, according to the relationship indicated by curve 3, $\left(\text{P. I.} = \frac{\text{L. L.} - 14}{1.60}\right)$, a plasticity index of 26. The plasticity index of the muck is 25, which shows that this muck has about the maximum plasticity likely to be found in A–8 soils.

According to curve 6 $\left(\text{S. L.} = \frac{\text{L. L.} + 86}{4.1}\right)$ the shrinkage limit should be 35. According to test it is 34. The centrifuge moisture equivalent should fall between values indicated by curve 9 (C. M. E. = 0.72 L. L.) and curve 10 $\left(\text{C. M. E.} = \frac{\text{L. L.} - 14}{0.55}\right)$. These values are respectively 40 and 76. The island muck has a centrifuge moisture equivalent of 64.

The field moisture equivalents of A–8 soils are likely to exceed those indicated by curve 12 $\left(\text{F. M. E.} = \frac{\text{L. L.} - 10}{1.03}\right)$. The field moisture equivalent of a soil having a liquid limit of 56, as defined by this relationship, is 45. The field moisture equivalent of this particular muck is 44.

Water-logging in the centrifuge indicates the presence of colloidal material. According to the mechanical analysis, Table 1, 11 per cent of the soil is smaller than 0.001 mm.

It is thus apparent that the two materials encountered are fairly representative of the subgrade soil groups to which they belong.

The test constants in this case clearly indicate the character of the muck. In addition to the water-logging in the centrifuge test, the relatively low field moisture equivalent of 44 combined with a high shrinkage limit of 35 suggests the presence of organic matter in the colloidal state. Also the high plasticity combined with the relatively high shrinkage limit suggests the presence of diatoms. It is clear, therefore, that this muck consists primarily of inorganic silt with some clay, colloidal organic matter, and diatoms.

STABILIZATION INVOLVES REDUCTION OF MOISTURE CONTENT AND ADDITION OF MATERIAL TO FILL

Generally, mucks of this character, more than any other soils, refuse to be stabilized by drainage and at the same time suggest the necessity of drainage. This is because, first, they are impermeable with respect to the flow of gravitational water and second, they have a very great capillary lift which causes them to contain abnormally large amounts of water even at appreciable distances above the ground-water elevation.

Thus, in the present case, open ditches were found to be ineffective for lowering the elevation of water in pockets or surface depressions less than 1 foot away and the moisture content at approximately 4 feet above the ground water elevation was found to be as much as 100 per cent,[3] generally, and in some cases more.

In order to comprehend fully the significance of this high moisture content, one should remember that a soil mass consists of soil particles and pores; furthermore, that when a soil shrinks or expands, only the volume of the pores changes, the volume of the soil particles remaining constant.

It is generally recognized that the important decrease in supporting value caused by increase in the moisture content of a soil does not occur gradually, but instead occurs abruptly when the moisture content exceeds the plastic limit of the soil. Therefore, treatment to be effective for stabilizing the muck under discussion must reduce the moisture content at least below the plastic limit or 31 per cent.[4] (56 − 25 = 31, Table 1.)

In the present instance, with the muck containing as much as 115 per cent of moisture, which is not uncommon for muck soils, the voids ratio is about 3.1 ($\frac{1}{100}\times$ specific gravity × moisture content) and this means that the pore space is about 3.1 times the volume of the soil particles.

In the stable sand, as indicated by the field moisture equivalent of 21 and a specific gravity of 2.65, the pore space is only about 0.6 and in the muck at the plastic limit is only about 0.84 as great as the volume of the soil particles. Thus, it can be seen that the reason for the lack of stability in the muck is simply a low percentage of soil particles. This low percentage of soil particles in the muck, as compared with the sand, is illustrated in Figure 4, A. Therefore, to stabilize the muck by drainage or any other means, its pore volume must be greatly reduced, and this in turn causes a like reduction in the volume occupied by the muck mass.

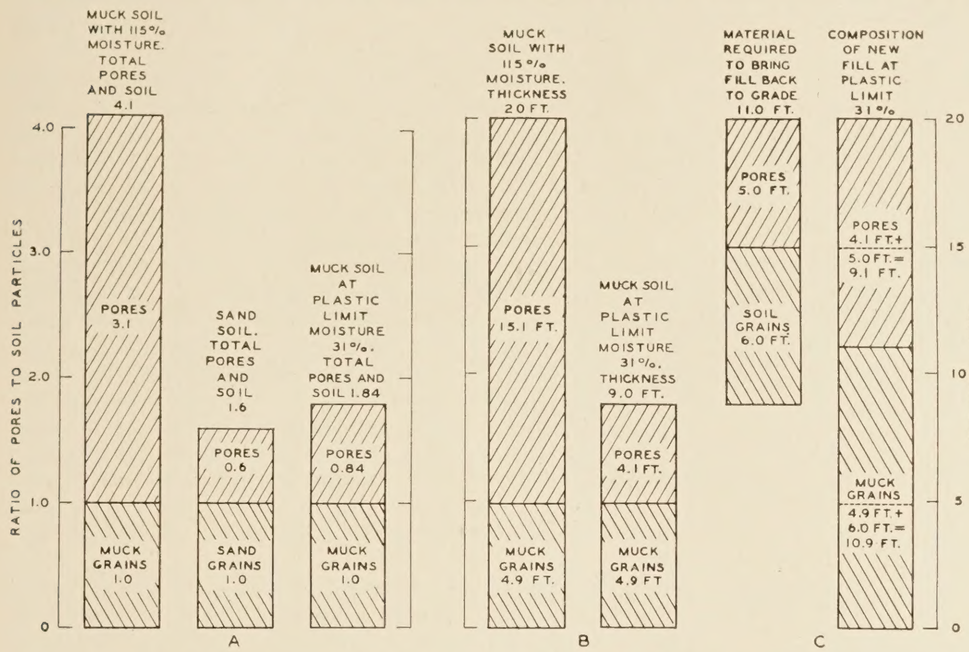


FIGURE 4.—DIAGRAM ILLUSTRATING AMOUNT OF ADDITIONAL MATERIAL REQUIRED, ON BASIS OF 20-FOOT DEPTH OF MUCK SOIL, WHEN MOISTURE CONTENT OF MUCK IS REDUCED FROM 115 PER CENT TO PLASTIC LIMIT, 31 PER CENT

If, for instance, the bed of muck containing 115 per cent of moisture were 20 feet thick it would consist, as shown in Figure 4, B, of 4.9 feet of solid soil particles and 15.1 feet of pores. In order to reduce the pore ratio of the unstable muck to that at the plastic limit the thickness of the muck layer would have to be reduced from 20 to 9.0 feet. (4.9 × 1.84.)

In order to bring the area back to original grade, a fill of 11.0 feet (20 − 9.0) would be required. And if the voids ratio of 0.84 were maintained throughout the fill, an additional thickness of solid soil equal to 6.0 feet (as shown in Figure 4, C) must be provided. This would require, according to cut measure, a sand layer approximately 9.6 feet (1.6 × 6.0) or a new muck layer 24.6 feet (4.1 × 6.0) thick.

So much for the additional material required by the stabilization of the muck. Let us now consider the practical aspects of the problem.

COMPRESSION TEST DATA INDICATE REQUIRED REDUCTION OF MOISTURE CONTENT

Figure 5 gives the results of a compression test which was run on a sample of the muck which had been manipulated. The shrinkage limit value of 43 for the material in the undisturbed state as compared with 34 for disturbed soil (see Table 1) indicates that the support actually furnished by the muck in its natural position might be somewhat higher than is indicated by Figure 5. However, the values shown are of the safe side. Grading operations and the vibrations caused by traffic tend to destroy the natural structure of the material. For this reason the values shown in Figure 5 are believed to be fair. They indicate that under a load of 1,000 pounds per square foot the moisture content will be reduced from 65 per cent to 57 per cent. Under a load of 3,000 pounds per square foot the moisture content will be reduced to 48 per cent; and under 6,000 pounds per square foot the moisture content will be reduced to but 41 per cent or 10 per cent above the stabilization point.

[3] Moisture content based on the weight of oven-dried soil.

[4] It should be noted that the shrinkage limit of the muck soil is higher than the plastic limit. This is due to the spongy properties of the muck and indicates that moisture when present even in amounts as low as the shrinkage limit could cause the soil to lose its stability if manipulated. In contrast to the muck soils, plastic clays of the compressible A-6 variety are usually more stable at the shrinkage limit than at the plastic limit moisture content.

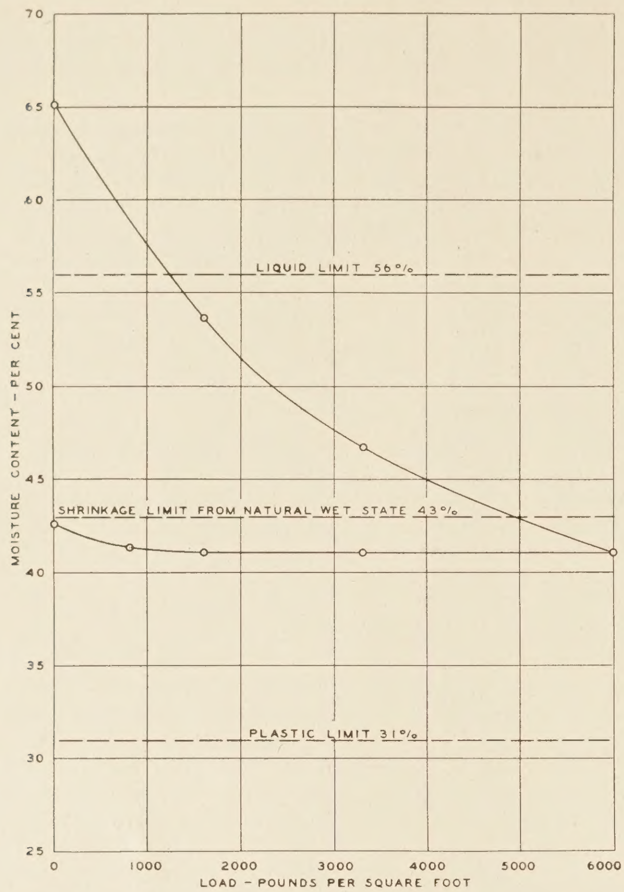


Figure 5.—Compression Test of Muck Soil in Disturbed State

These test data reveal information on two important points. On one hand they show that in order to stabilize a layer of the soil of indefinite thickness by drainage, its moisture content must be reduced below 31 per cent, and furthermore, that it would require a pressure much greater than 6,000 pounds per square foot actually to squeeze the water out. On the other hand, they show that if this muck serves as a base for a new top layer of soil it will support 1,000 pounds per square foot, or an 8-foot thickness of gravel, even with 57 per cent of moisture; and that with a moisture content of 61 per cent it will support a 4-foot layer of ground.

If only the character of the material were known, and not the conditions under which it exists in place, a comparison of the results likely to be furnished by drainage and by covering would suggest covering to be the more logical procedure in uniform deposits of great depth where the ground water table is high. In stratified deposits, however, the removal by drainage of water which is the source of detrimental capillarity may serve to very quickly stabilize the surface muck layer. Columbia Island, according to a subgrade survey, was found to be an excellent representation of the type of stratified soil profile in which such stabilization could be accomplished quickly and economically.

SOIL SURVEY INDICATED CHARACTER OF DEPOSITS AND DISTRIBUTION OF MOISTURE

In connection with this survey borings made every 50 feet, along and on both sides of the center line where trouble was encountered, brought to light several interesting conditions not usually found in natural soil deposits. The soil profile is shown in Figure 6. The sand and muck occurred in layers and pockets varying from an inch to several feet in thickness. In places wet sand occurred above dry muck; in others, dry sand over saturated muck; in others, wet muck over wet sand; and in still others, dry muck over wet sand.

Finally, in several instances perched water tables were encountered. All of the free water surfaces were above the elevation of the river; but in certain places free water occurred in several layers vertically above each other with relatively dry material in between.

The comparatively small pockets of saturated sand and muck presented no particular difficulties. The prevailing condition, particularly in the area between stations 10 + 00 and 13 + 00, where the soil was the least stable, was that of a layer of saturated muck about 6 feet thick underlain by a bed of saturated sand.

A consideration of the characteristics of this material furnished important information. Those test results which showed how the fill would shrink if the moisture content were to be reduced from 115 to 31 per cent, also showed that if this soil had a chance to dry out by evaporation it would crack to such an extent that it would transmit water freely. It could be readily seen that if the water were taken away from the bottom of the muck, evaporation from the top would dry out the soil. These two factors pointed in only one direction and that was to drain out the sand layer and let the characteristic properties of the muck do the rest.

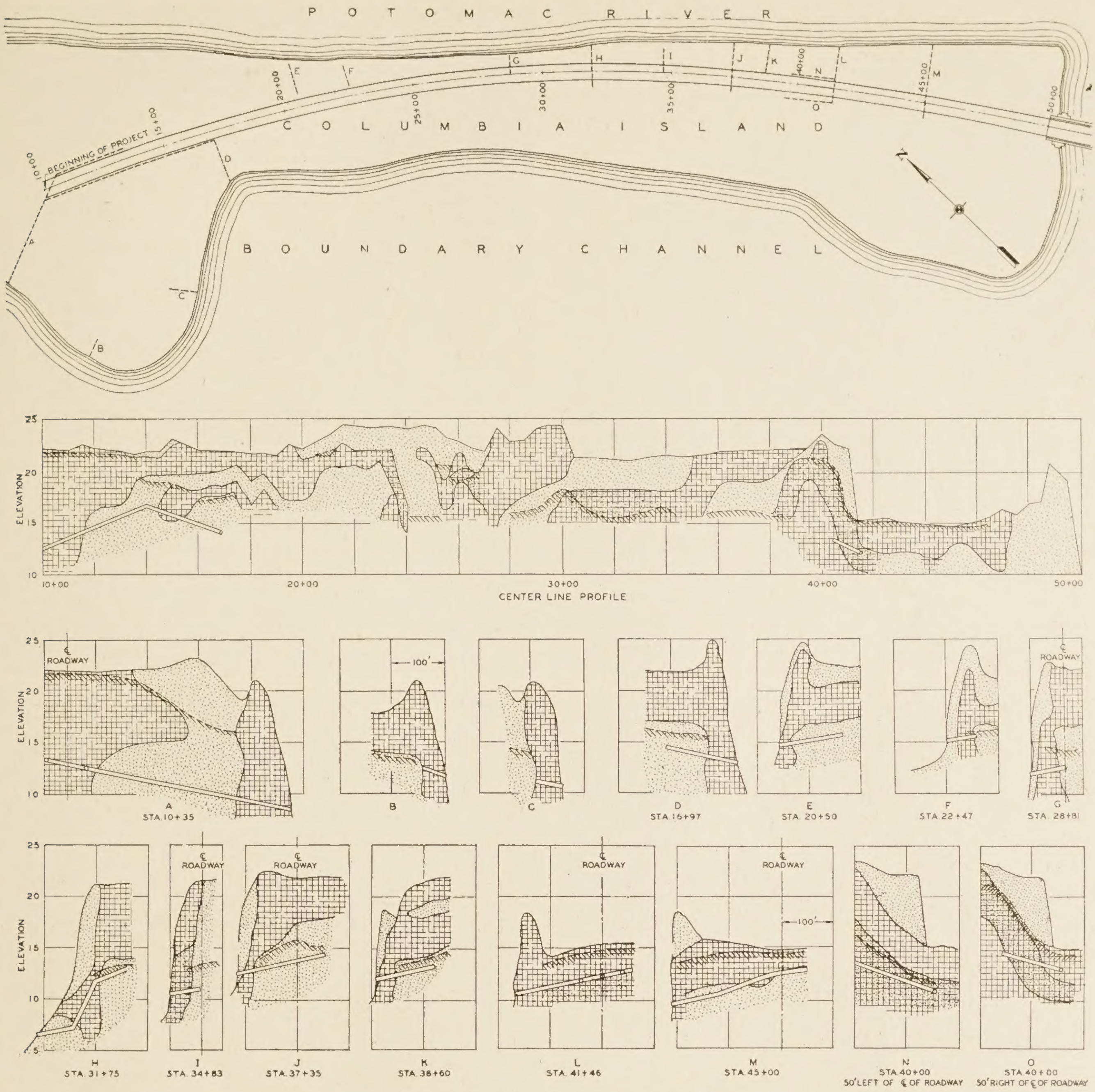
Knowledge of the manner in which the island was formed was very helpful in selecting the location and depth at which the drains should be placed. The island was built up over a period of years from the material dredged out of the river channel. It was thus constructed in a series of "lifts."

Each lift was formed by building a bank or dike around an area and pumping in sand and muck from the river bottom. The area inclosed by the dike formed a settling basin and the excess water was allowed to escape over a timber spillway.

Great pains were taken to make the dike impervious as this reduced the danger of washouts. Wherever seepage through the banks was noticed, soft muck was spread over the inner faces of the dikes with hoes until the dikes were effectively sealed. In this manner impermeable walls were built up around the whole island.

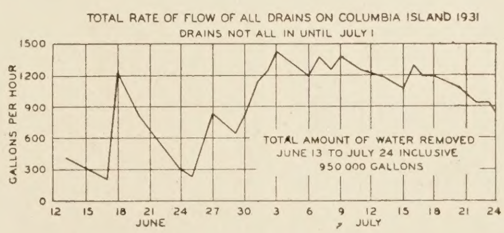
The coarse material naturally settled out of suspension faster than the fine material and was deposited nearer to the end of the discharge pipe. This action had the effect of separating the coarse from the fine material and several instances were encountered where a large amount of sand was surrounded by impermeable muck forming a reservoir.

Since the island was formed in successive lifts with a considerable time interval between the placement of each lift, there was time for each one to become more or less consolidated and covered with vegetable growth. Consequently, a layer of slightly decomposed organic matter was often encountered and a difference in permeability was noticed between successive lifts even where the material appeared to be the same. Furthermore, the material from which the different lifts were formed was often obtained from different parts of the river, with the result that successive layers often consisted of widely different materials.



VOLUME OF WATER FLOWING FROM DRAINS JUNE 30

DRAIN	GALS PER HOUR	DRAIN	GALS. PER HOUR
A *	281	I	5
B	NOT MEASURED	J	27
C *	281	K	12
D	79	L	105
E	NONE	M	84
F	11	N	NOT MEASURED RUN INTO "L"
G	32	O	NOT MEASURED RUN INTO "L"
H	372	* MEASURED JULY 1	



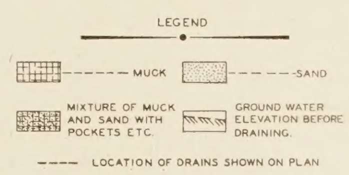


FIGURE 6.—MAP AND SOIL PROFILE OF COLUMBIA ISLAND

PLAN OF DRAINAGE DESCRIBED

In general, the drainage plan consisted of three essential operations, namely: (1) Removal of free surface water, (2) removal of brush and vegetation, and (3) tapping of the impermeable dike and placing drains in such a manner as to remove all underground water which was trapped in layers or pockets.

The surface water was removed from the wet area between stations 10 + 00 and 13 + 00 in an attempt to dry it up enough to make it possible to work in this area. Shallow surface ditches (figs. 7, A and 8, A) were dug from the dikes back toward the center line. A great quantity of water was carried off in this manner and as soon as the water on the surface was removed

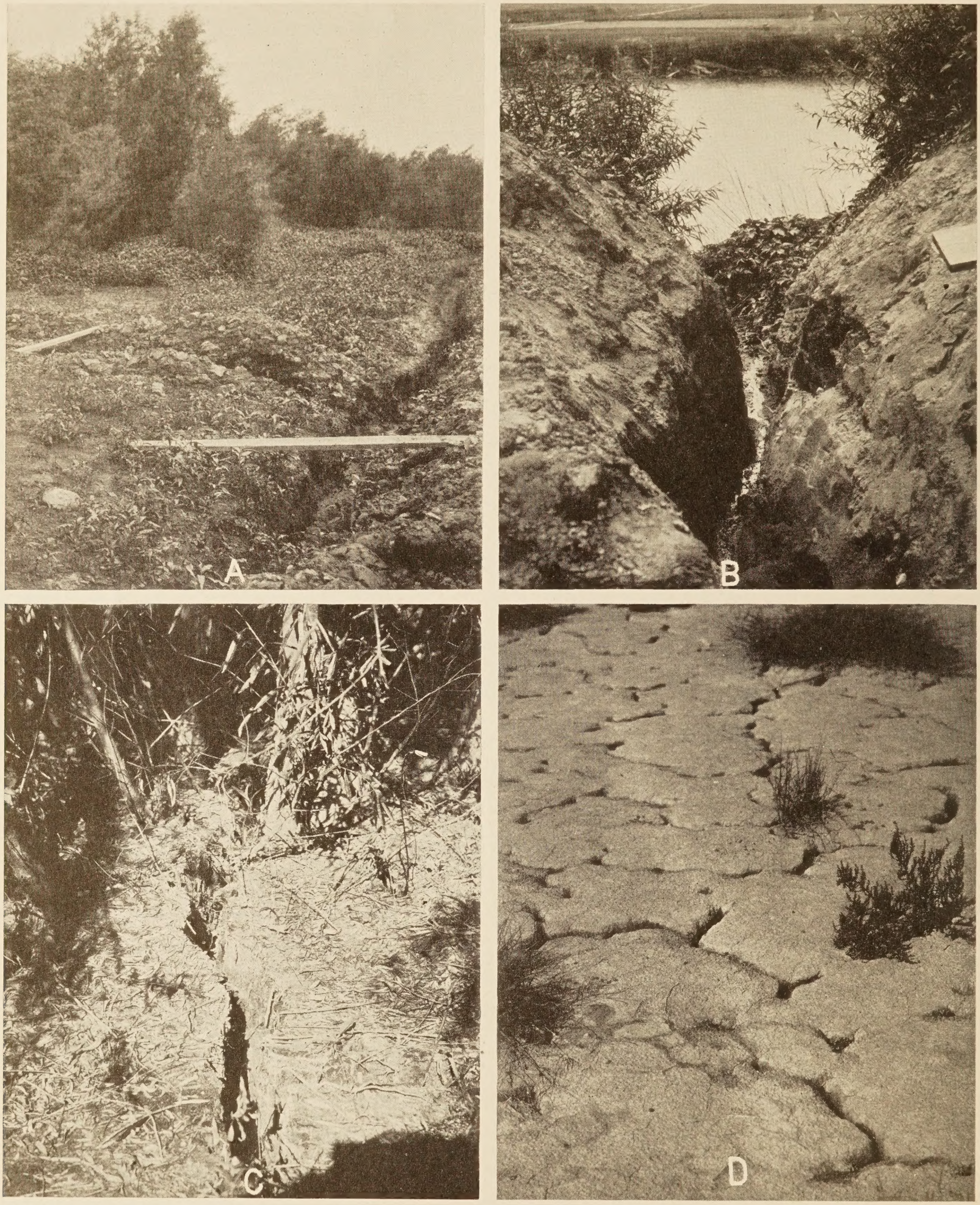


Figure 7.—Steps in the Drainage of Columbia Island

A.—Ditches for removing surface water.
B.—Cut through dike. The ditch as shown barely tapped the underground water. Unstable material prevented deepening ditch until it had been timbered.
C.—Typical shrinkage crack in wet area after drainage.
D.—Shrinkage cracks in soil after drainage.

a crust formed. This crust was not very thick, but as it dried out it began to shrink and crack as illustrated in Figure 8, B, indicating that the soil would eventually become more stable if the free water were removed from it.

All underbrush and vegetation was cut off the right of way. This step very materially aided evaporation and speeded up the drying-out process.

In disposing of the underground water the first and most important step was to cut through the impermeable dikes, as shown in Figure 7, B. This step alone would, in time, have removed a great deal of the free water from the soil.

As it was necessary to begin grading operations as soon as possible, tile drains were installed in such a manner as to tap the water pockets and dry them up rapidly. The location and depth of the drains depended on existing conditions as determined by borings.

Where saturated sand was surrounded by muck the problem was simply to locate the lowest spot in the sand pocket and run a drain to that point.

In instances where wet muck was found over dry sand, drainage was accomplished by cutting a trench through the muck to the sand. The material close to the trench would begin to drain and as it dried out cracks were formed. These cracks, illustrated in Figure 7, C and D, extended back from the ditch and served as water carriers and so the whole area was gradually drained.

In the most common condition encountered, that of a layer of wet muck at the surface resting on a bed of saturated sand, tile drains were so placed that they would take the water out of the sand. With the removal of this water through cuts in the dike, the muck would have slowly dried out. Tile drains connecting these outlets with lateral drains along the roadway materially hastened the stabilizing action.

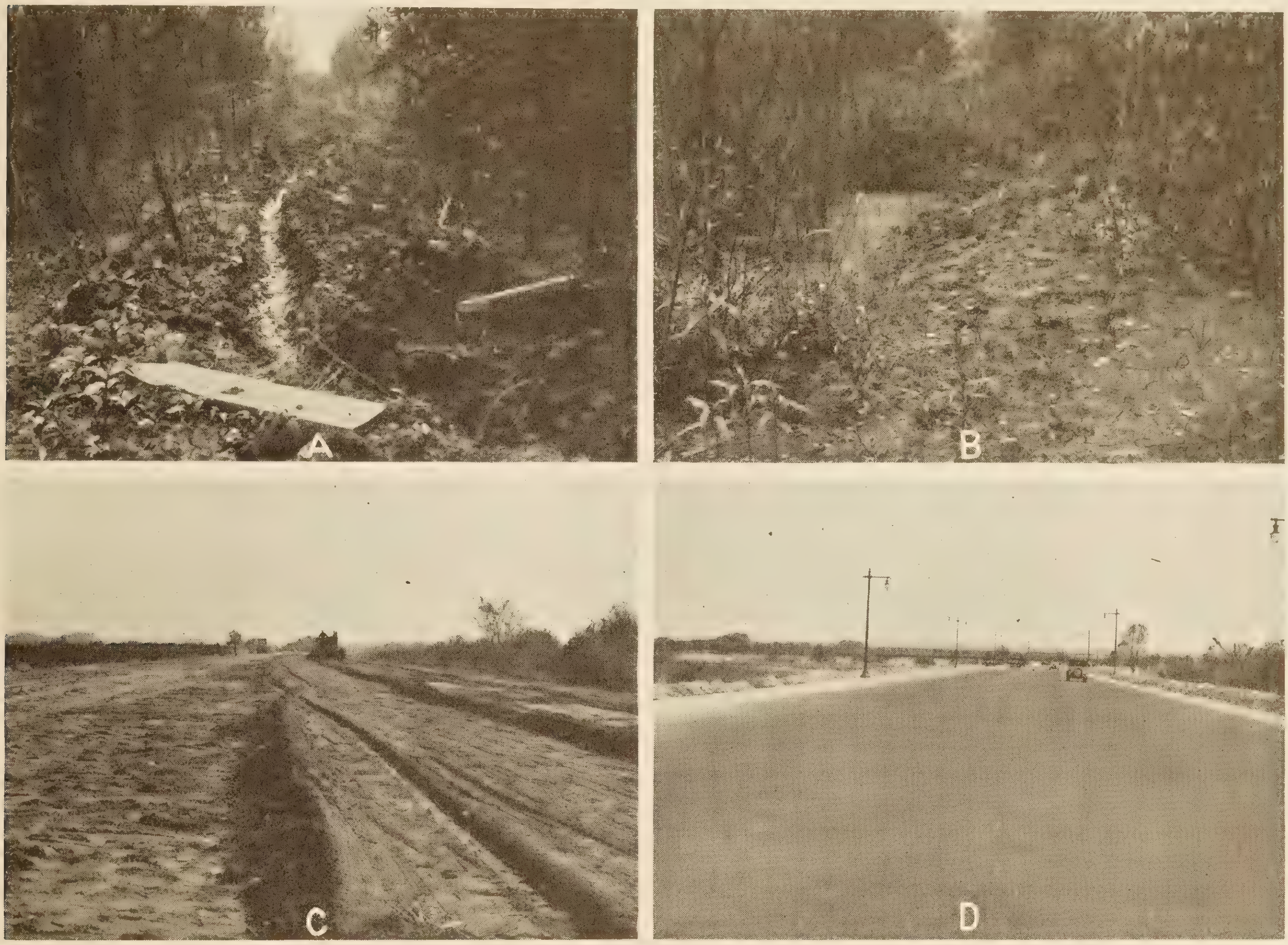


FIGURE 8.—PHOTOGRAPHS TAKEN AT STATION 10+00, AT DIFFERENT STAGES OF THE WORK

A.—Ditch for removing surface water, looking north from station along center line.
B.—Looking north from station after surface water had been removed but before underbrush had been cleared away and underground drainage had been installed.
C.—Constructing the fill after drainage, looking south from station.
D.—The completed road.

DRAINAGE OPERATION SUCCESSFUL IN CONSOLIDATING FILL

The quantity of water which has been removed from the island can be estimated from the data shown in Figure 6. Several of the drains have been carrying water continuously for nearly a year. Further evidence as to the amount of water removed may be obtained from a comparison of the thickness of the muck layer before and after drainage. Borings made

(Continued on p. 72)

THE SEGREGATION OF WATER IN CONCRETE PLACED IN DEEP FORMS

By F. H. JACKSON, Senior Engineer of Tests, and W. F. KELLERMANN, Associate Materials Engineer, United States Bureau of Public Roads

IT IS well known that during the continuous placing of concrete in deep forms such as columns, piers, abutments, retaining walls, etc., there is a tendency for water within the concrete to rise as the form is filled. This results in an accumulation of excess water in that portion of the concrete at and near the upper surface of each continuously placed section with the consequent formation of layers of relatively weak, porous concrete at the fill planes. It is a matter of common observation that such concrete, when exposed to the weather, deteriorates much more rapidly than the adjacent concrete at the bottom of the next lift. Typical examples of this action are shown in Figure 1. It has been generally assumed that this difference in the rate of weathering is due in large part to excessive water, generally known as water gain, although little or no data have been published which show quantitatively either the actual effect of water gain on strength and durability or the relative influence of various factors such as grading of aggregates, consistency of concrete, area of section, method of placing, tamping, etc., on the extent to which water gain takes place.

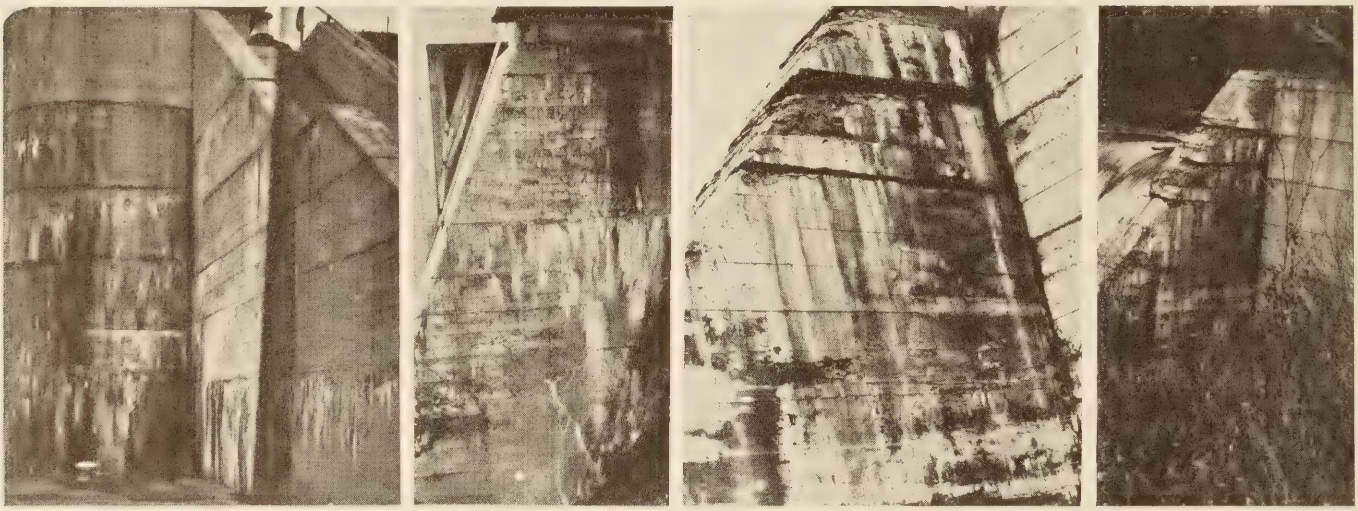


FIGURE 1.—TYPICAL EXAMPLES OF DISINTEGRATION AT FILL PLANES

The problem appeared to the bureau to be of sufficient interest to justify a preliminary series of tests under certain job conditions to determine the extent to which water gain developed and its effect upon the quality and uniformity of the concrete.

It was not possible to cover a wide range in test conditions; and the applicability of the data is restricted to the conditions under which it was obtained. While forms of different heights were used, they were all of one width, 8 inches. Methods of placing included both spading and vibrating. The fine aggregate used was a relatively fine sand. The consistency of the concrete was varied from 2 to 8 inches.

DESCRIPTION OF TESTING PROCEDURE

The tests were made by investigating the variations in composition and strength of concrete in a series of 8 by 48 inch vertical wall columns cast as units and ranging in height from 3 to 12 feet. Four different heights of column were used, 3 feet, 6 feet, 9 feet, and 12 feet. One side of each form was provided with a series of ports equipped with closely fitting doors, spaced about 2 feet apart vertically. From 1 to 1½ hours after placing, depending upon consistency, a sufficient quantity of the fresh concrete was obtained from each of these ports to permit the fabrication of one 6 by 12 inch cylinder as well as an analysis of the fresh concrete. A diagrammatic view of one set of columns is shown in Figure 2. This view shows the location of the ports (indicated by circles) and also the location from which cores were subsequently drilled from the hardened concrete (indicated by crosses). A ⅞-inch vertical dividing strip divided each column into two 8 by 24 inch sections, as indicated. This division was made in order to permit the withdrawal of samples of wet concrete without disturbing the concrete from which the cores were to be drilled.

A total of 33 columns were cast. With the exception of series K, which consisted of one 12-foot column only, they were constructed in sets of four, one of each of the four heights being cast on the same day. Various consistencies were employed, ranging from 2 to 8 inch slump, as well as two methods of consolidating, hand spading and vibrating with an electric vibrator attached to the form. Table 1 gives the consistency and method of compacting employed in each series of four columns.

TABLE 1.—*Consistency and method of compaction used in each series*

Series	Average slump in inches	Method of compaction	Series	Average slump in inches	Method of compaction
A	4	Spaded.	F	8	Spaded.
B	8	Do.	G	5	Vibrated.
C	4	Vibrated.	H	2	Do.[1]
D	3	Do.	K[2]	([3])	Do.
E	4	Spaded.			

[1] Sides and ends of forms also spaded.
[2] 12-foot column only cast.
[3] Slump of concrete in the lower 1 foot, 2 inches. Slump of concrete in remaining 11 feet, 4 inches.

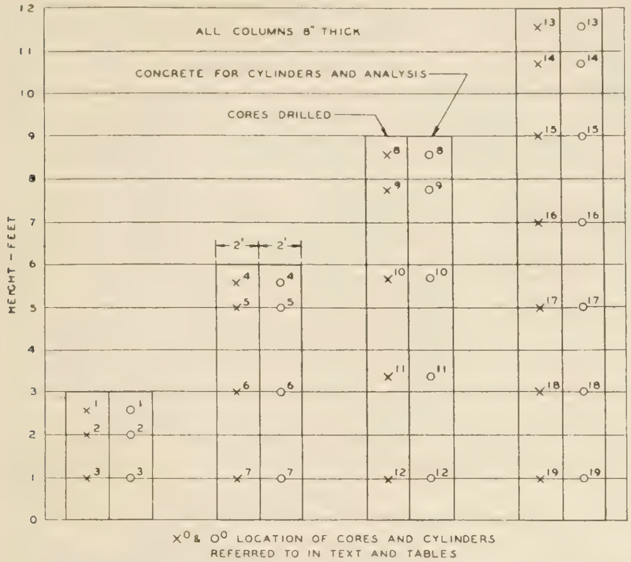


FIGURE 2.—LOCATION OF CONCRETE SPECIMENS TAKEN FROM COLUMNS

A 1:2:4 dry-rodded volume mix, the aggregates being Potomac River sand and gravel, was used throughout the series. The results of tests on the cement are given in Table 2 and the results of tests on the aggregates in Tables 3 and 4. It will be noted that a relatively fine sand having a fineness modulus of 2.49 was employed. This is somewhat finer than the concrete sands employed in general in the Northern States, but is similar to the grading of many natural sands in other parts of the country.

TABLE 2.—*Tests of cement*

Fineness, percentage retained on 200-mesh sieve	7.8
Time of set (Gillmore)—	
Initial	2 hours 35 minutes
Final	6 hours 00 minutes
Soundness	Satisfactory
Normal consistency, per cent	24

TENSILE STRENGTH

[Pounds per square inch, 1:3 Ottawa sand mortar briquets]

1 day	3 days	7 days	28 days
180	290	345	425
170	315	430	420
155	305	370	425
Average 170	305	380	425

The concrete was mixed one minute in a 1-sack mixer. Water was added at the mixer until the required slump was obtained and this amount plus the free water in the aggregates was taken as the total from which the water-cement ratios at time of mixing (shown in fig. 11) were computed.

The forms were placed on a grade so that the top of the 12-foot forms was only about 5 feet above the top of the 3-foot forms A scaffold was built from the mixer to the forms and the concrete hauled in buggies and dumped directly into the forms, each buggy depositing a layer of concrete about 9 inches in depth. The buggies were dumped in such a way that the concrete from the left side of the buggy went into that section of the form from which concrete for analysis and cylinders were taken, while that from the right side of the buggy went into that section from which the cores were drilled. By this method of manipulation any segregation in the buggies was divided between the two sections of the forms. Cylinders were made from concrete taken from the mixer as the columns were being cast. In Figure 3 a general view of the project is shown.

TABLE 3.—*Tests of fine aggregate*

Sieve analysis:	
Total retained on—	Per cent
No. 4 sieve	1
No. 8 sieve	10
No. 16 sieve	20
No. 30 sieve	40
No. 50 sieve	83
No. 100 sieve	95
Fineness modulus	2.49
Silt and clay, per cent	2.8
Apparent specific gravity	2.65
Weight per cubic foot (dry-rodded), pounds	100

TABLE 4—*Tests of coarse aggregate*

Sieve analysis:	
Total retained on—	Per cent
1¼ inch sieve	0
¾-inch sieve	5
⅜-inch sieve	65
No. 4 sieve	97
Fineness modulus	6.67
Apparent specific gravity	2.55
Weight per cubic foot (dry-rodded), pounds	102
Percentage absorption—	
30 minutes	.85
24 hours	.91

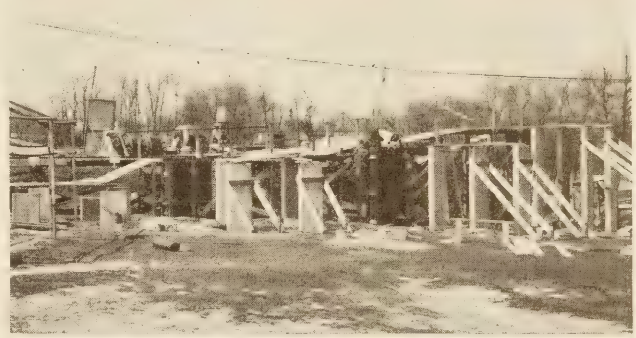


FIGURE 3.—GENERAL VIEW SHOWING HARDENED COLUMNS OF DIFFERENT HEIGHTS, AND FORMS BEING RESET

There were two methods used for compacting the concrete, spading and vibrating. The spading method was as follows: After each buggy was dumped the sides and ends of the forms were spaded with a sidewalk scraper with 1-inch holes punched in it to permit the fine material to come in contact with the form. The concrete was then cut with a vertical movement of the spading tool 18 times, 6 times in the center, and 6 times each in two places about midway between the center and the ends. Two men spaded simultaneously, one man spading the right section while the other man spaded the left. (See fig. 4.) In all cases the spading tools were driven down into the previous layer of concrete. This procedure was continued until the form was filled, after which the top was struck off and wet burlap applied. The method of vibrating was as follows: The electric vibrators were attached to the forms by clamping them to the 2 by 4 inch battens as shown in Figure 5. Each vibrator consisted essentially of an electric motor carrying an eccentric weight on the shaft and operating at a frequency of 3,600 vibrations per minute. On the 3-foot forms one vibrator, clamped in one position near the top, was used. On the 6-foot

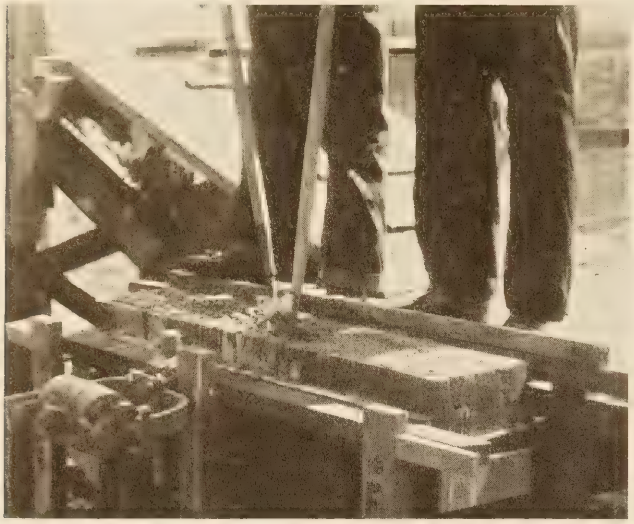


Figure 4.—Showing Method of Simultaneous Spading and Vibrating

forms one vibrator was used but was moved from near the bottom to a position about two-thirds up, as the form was filled. On the 9-foot forms two vibrators, placed on opposite sides, were used. They were staggered as to height and the lower vibrator was moved up to a position near the top as the forms were filled. On the 12-foot forms (for series C and D), two vibrators were used in a manner similar to that used on the 9-foot

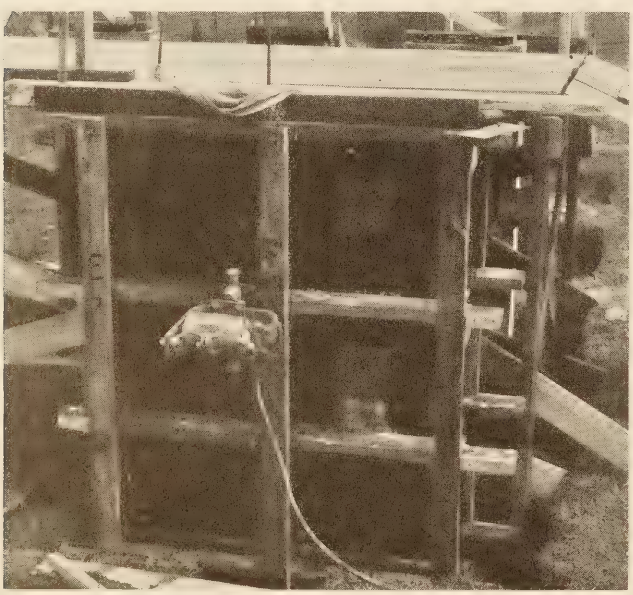


Figure 5.—Vibrator Clamped to Batten of 6-Foot Form

forms. In series G, H, and K, both vibrators were placed on the same side. This was done in order to permit the installation of pressure cells for measuring the pressure exerted by the wet concrete. The results of tests for distribution of pressure was published in Public Roads, vol. 12, No. 1, March, 1931. In series C the vibrators were run only a sufficient length of time (5 to 20 seconds) after each buggy was dumped to level off the concrete. In the other vibrated series the vibrators were run continuously while the forms were being filled. In series H the forms were spaded on the sides and ends in addition to being vibrated. This was done in order to eliminate as far as possible the pitted faces of the vibrated columns. Figure 6 shows this condition. The column shown in the center background was vibrated while those in the right and left foreground were spaded. The tests indicated that spading the sides of the forms during vibration materially reduced the amount of surface pitting. In all cases wet burlap was applied as soon as the forms were filled as in the spaded series.

It was found necessary to brace and tie the form very securely when using the vibrators in order to avoid springing the forms.

Series K (12-foot column only) was placed in order to study the effect of excessive vibration. No samples were taken for analysis or cylinders fabricated but cores were drilled as in the other series. In this series the vibrators were placed on opposite sides of the forms at the same elevation and moved up with the level of the concrete as the form was filled. By this method the form was given considerably more vibration than in the case of any other series.

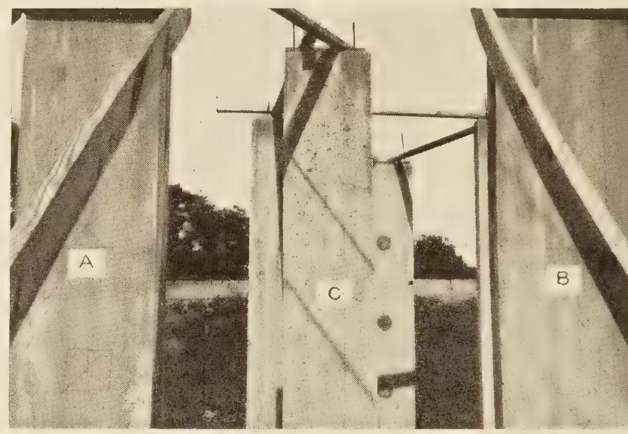


Figure 6.—Showing Pitted Surface on Top of Center Column, a Condition Typical of Vibrated Columns. Columns to Left and Right Spaded

The concrete for analysis was taken to the laboratory and the water-cement ratio and cement aggregate ratio determined by the method described by Prof. W. M. Dunagan in the Journal of the American Concrete Institute, December, 1929. The cylinders were made and stored adjacent to the columns. After 24 hours both the column and cylinder forms were removed, and no further curing provided.

At an approximate age of 4 months the columns were placed in a horizontal position and cores drilled at the points indicated in Figure 2.

All cylinders and cores were then capped and stored in the open until an age of 6 months, when they were tested for compression after immersing in water for 24 hours. Both cylinders and cores were tested in a 200,000-pound universal testing machine. The cores were approximately 5.7 inches in diameter and 8 inches high and a correction factor was applied to make the strength correspond to a specimen having a height equal to twice its diameter.

RESULTS OF INVESTIGATION DISCUSSED

Segregation determined by analysis of fresh concrete.—In Table 5 will be found the volumes of cement to total combined aggregates and the water-cement ratios for each sample of concrete taken from the forms before setting. The method used in making these determinations, as proposed by Professor Dunagan, consists of weighing the sample of concrete in air and in water, then separating the cement, fine and coarse aggregate, by washing through suitable screens and reweighing each in water. From these weights and the specific gravities of the materials, the weights in air are computed. The sum of these weights subtracted from the original weight gives the weight of water. Knowing the weight of water, cement, fine and coarse aggregate, the mix may be readily computed. In these tests no attempt was made to separate the fine and coarse aggregate so that the mix determinations are given as one part of cement to parts of combined fine and coarse aggregate by volume. A No. 100 sieve was used to separate the cement from the aggregate, correction being made for the percentage of cement retained and percentage of sand passing the same sieve. In working with the fine and coarse aggregate combined, it was necessary to use one specific gravity. Inasmuch as the gravity of the sand was somewhat higher than that of the gravel, a weighted value was used assuming the sand and gravel to be present in the proportion of 1 to 2.

The values given in Table 5 are shown for the 6 and 9 foot columns in Figure 7 and for the 12-foot columns in Figure 8. Values for the 3-foot columns are not shown in plotted form.

TABLE 5.—*Segregation as determined by analysis of fresh concrete*

Sample No.	Parts of total combined aggregate to 1 part of cement								Water-cement ratio, cubic feet of water to 1 sack of cement							
	Series								Series							
	A	B	C	D	E	F	G	H	A	B	C	D	E	F	G	H
3-FOOT COLUMNS																
1	5.2	5.4	5.4	4.8	4.8	5.3	5.6	6.2	0.85	1.00	0.83	0.85	0.81	0.89	0.93	0.80
2	7.6	5.8	5.9	5.6	5.9	6.2	6.2	6.2	.98	.96	.88	.83	.87	.95	.91	.84
3	5.4	5.8	6.2	6.4	6.2	6.4	5.1	5.8	.76	.91	.87	.85	.91	.96	.82	.79
6-FOOT COLUMNS																
4	5.2	4.8	5.2	4.6	5.3	5.3	----	6.2	.82	.98	.83	.79	.87	.91	----	.86
5	6.0	5.8	6.0	5.9	6.0	5.7	----	6.1	.82	.93	.97	.81	.81	.87	----	.89
6	5.7	5.8	5.4	5.7	5.5	6.1	6.2	5.0	.79	1.02	.77	.84	.83	.95	.91	.68
7	5.7	5.8	5.5	5.8	6.0	6.0	5.9	5.6	.79	.92	.83	.83	.83	.83	.87	.79
9-FOOT COLUMNS																
8	5.3	4.7	5.5	5.0	5.5	5.3	5.7	6.0	.80	.92	.83	.84	.86	.93	.83	.79
9	5.3	5.6	5.1	5.5	5.9	6.1	5.8	6.5	.79	.93	.84	.86	.84	.92	.88	.84
10	5.5	5.6	5.3	5.4	5.3	5.4	6.1	5.8	.77	.96	.85	.82	.79	.84	.86	.70
11	5.6	5.1	5.4	5.7	5.3	5.7	6.1	5.1	.82	.83	.82	.82	.75	.92	.79	.74
12	5.4	5.8	5.8	5.9	5.5	5.6	6.6	5.4	.75	.88	.86	.87	.80	.87	.86	.73
12-FOOT COLUMNS																
13	4.8	4.1	5.0	5.4	4.7	5.0	5.6	6.4	.79	.92	.79	.79	.78	.86	.85	.89
14	5.3	5.1	5.2	5.8	5.6	5.7	6.5	5.1	.81	.98	.80	.76	.83	.87	.81	.69
15	5.8	6.1	5.1	6.0	5.4	5.5	6.1	5.8	1.06	.94	.78	.95	.77	.84	.87	.71
16	5.6	5.4	5.5	5.5	5.3	5.6	6.7	6.1	.89	.85	.80	.75	.79	.86	.85	.77
17	5.8	5.9	5.3	5.6	5.4	6.1	6.3	6.3	.82	.87	.80	.76	.75	.96	.83	.80
18	5.5	5.8	5.3	5.6	5.4	5.7	5.7	6.3	.82	.91	.78	.74	.75	.89	.78	.72
19	5.4	5.6	5.4	5.7	5.7	6.0	6.0	5.9	.86	.84	.81	.76	.81	.90	.76	.68

Before discussing these data, attention should be called to the fact that individual rather than average results are shown. Each plotted point in Figures 7 and 8, therefore, represents the result of a single test on one sample of concrete taken from the indicated portion of the structure. Too great significance should not be attached to individual test results as such. Instead, an effort should be made to ascertain what, if any, general trends appear to exist, as regards (1) the relative amount of cement in terms of the volume of concrete and (2) the relative amounts of water in terms of the volume of cement (the water-cement ratio).

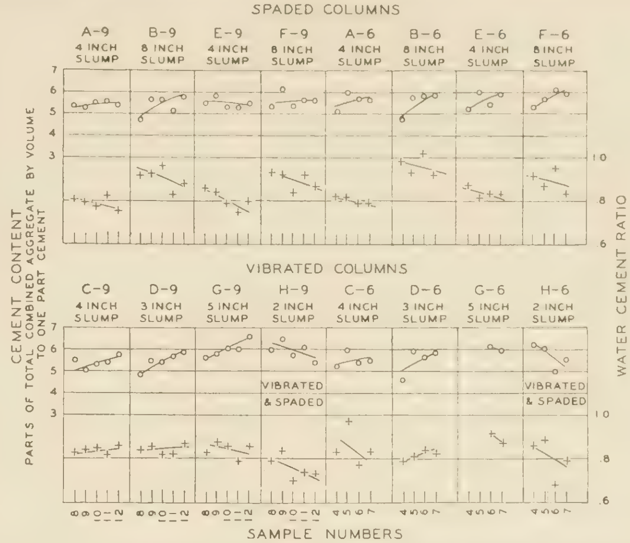


FIGURE 7.—RESULTS OF TESTS TO DETERMINE UNIFORMITY OF COMPOSITION OF CONCRETE IN 9 AND 6 FOOT COLUMNS

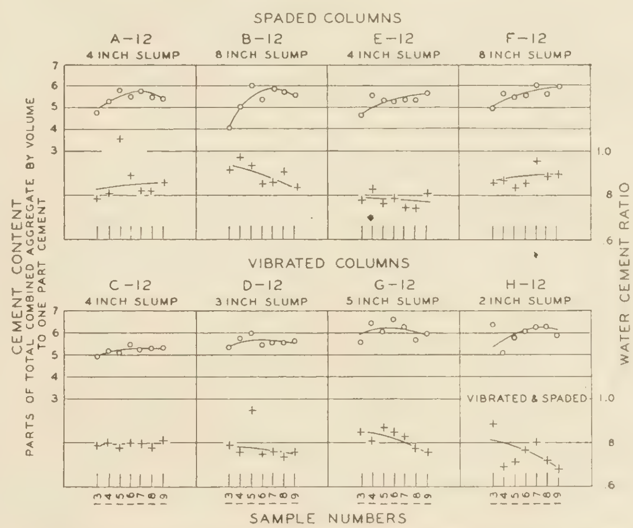


FIGURE 8.—RESULTS OF TESTS TO DETERMINE UNIFORMITY OF COMPOSITION OF CONCRETE IN 12-FOOT COLUMNS

From an examination of the variations in cement content, as plotted in the upper portions of each diagram in Figures 7 and 8, it will be observed that there is a distinct tendency for the cement content to increase near the top of the columns. In 17 out of 24 cases, the sample at the extreme top showed more cement than any other position. This, however, is only a very general relation, the individual values not being very consistent except in a few cases, such as columns D–9 and G–9. In columns H–6 and H–9 the tendency is in the other direction. In still other cases, the individual values vary quite widely without any apparent reason. In spite of the many individual discrepancies, however, there seems to be a general tendency along the line indicated; that is, a somewhat richer mix at the top of the columns. The effect of this variation in cement content on strength is discussed in the next section of this report.

Referring now to the variations in water-cement ratios, as plotted in the lower part of each diagram in Figures 7 and 8, we find that a slight tendency toward increased water content at the top of the columns is shown in some cases. The individual results, however, vary quite widely. Here again we are confronted with the fact that each point represents but one test result, with a consequently increased tendency for experimental errors to affect the results.

It is reasonable to suppose that water gain, if occurring at all, would develop in series B and F, the two sections in which 8-inch slump concrete, spaded in the forms, was used. In Figures 7 and 8 it will be seen that for series B and F there is, with the exception of column F–12, a general tendency in all cases for water to increase toward the top. In only one case, however, F–9, is the maximum water found in the very top sample. In general, it may be said that, in so far as the segregation test was able to measure, there was no serious water gain in any of these columns. This conclusion is verified by visual inspection made during the placing of the columns. No marked water gain was noted in any case.

Segregation determined by strength tests.—As previously noted, concrete strength specimens were obtained from various portions of each column, both by molding 6 by 12 inch cylinders from samples of concrete taken from the ports and by drilling cores from corresponding locations in the columns. Tests on these specimens may also be compared to tests on samples of concrete taken just after mixing and before placing in the forms.

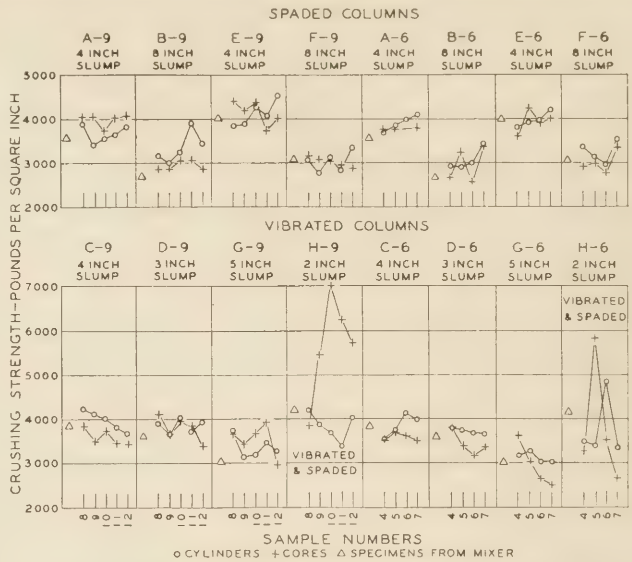


Figure 9.—Results of Tests to Determine Uniformity of Strength of Concrete in 9 and 6 Foot Columns

The results of these strength tests are shown in detail in Table 6 and are plotted for the 6, 9, and 12 foot columns in Figures 9 and 10. Here also, attention

Table 6.—*Strength tests of concrete. Values given are compressive strengths in pounds per square inch*

Specimen No.	Series A		Series B		Series C		Series D		Series E		Series F		Series G		Series H		Series K
	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cylinders	Cores	Cores
3-FOOT COLUMNS																	
1	3,450	3,780	2,480	3,290	4,130	4,070	3,870	3,590	3,790	3,260	2,810	2,820	3,210	3,460	4,330	4,350	--------
2	3,180	3,700	2,620	2,710	3,630	3,150	3,750	3,540	3,720	3,900	2,710	3,120	3,180	3,080	3,440	3,500	--------
3	3,080	3,100	2,680	2,200	3,500	3,030	3,700	3,610	3,610	3,370	2,960	2,860	3,140	2,880	4,240	3,930	--------
Average	3,236	3,526	2,593	2,733	3,753	3,416	3,773	3,580	3,706	3,510	2,826	2,933	3,176	3,140	4,003	3,926	--------
6-FOOT COLUMNS																	
4	3,710	3,790	2,940	2,650	3,570	3,500	3,780	3,780	3,800	3,600	3,380	2,910	3,180	3,630	3,500	3,280	--------
5	3,830	3,780	2,910	3,210	3,780	3,680	3,730	3,380	3,930	4,220	3,120	3,000	3,260	3,060	3,400	5,850	--------
6	4,000	--------	3,000	2,550	4,150	3,620	3,660	3,160	3,990	3,920	2,980	2,780	3,000	2,640	4,890	3,510	--------
7	4,100	3,800	3,410	3,380	4,000	3,500	3,650	3,350	4,210	4,000	3,520	3,360	3,040	2,500	3,370	2,680	--------
Average	3,910	3,790	3,065	2,947	3,875	3,575	3,705	3,417	3,982	3,934	3,250	3,012	3,120	2,957	3,790	3,830	--------
9-FOOT COLUMNS																	
8	3,930	4,010	3,180	2,870	4,230	3,840	3,900	4,100	3,850	4,400	3,080	3,160	3,730	3,650	4,200	3,860	--------
9	3,410	4,040	2,980	2,870	4,110	3,500	3,670	3,630	3,900	4,170	2,790	3,080	3,140	3,420	3,870	5,440	--------
10	3,560	3,760	3,250	3,020	4,030	3,740	4,010	3,960	4,250	4,370	3,110	3,010	3,190	3,670	3,690	7,040	--------
11	3,670	4,010	3,920	3,040	3,800	3,460	3,720	3,830	4,080	3,710	2,840	2,910	3,460	3,910	3,360	6,220	--------
12	3,840	4,070	3,420	2,850	3,680	3,430	3,930	3,370	4,550	4,000	3,350	2,870	3,270	2,950	4,040	5,710	--------
Average	3,682	3,978	3,350	2,930	3,970	3,594	3,846	3,778	4,126	4,130	3,034	3,006	3,358	3,520	3,832	5,654	--------
12-FOOT COLUMNS																	
13	3,960	4,070	2,740	3,080	3,950	4,120	3,850	4,280	4,310	4,100	3,050	3,250	2,990	3,170	3,710	4,770	3,920
14	3,610	4,340	2,620	2,840	3,460	2,990	4,380	4,290	4,270	4,670	2,860	3,100	3,270	3,680	4,200	5,640	3,240
15	3,820	3,950	2,870	2,650	4,000	3,570	4,170	3,860	4,680	4,500	3,360	3,210	3,110	3,070	4,060	4,320	2,900
16	3,450	3,740	2,950	2,850	4,110	3,870	4,560	3,930	4,350	4,010	3,280	3,310	2,920	3,040	3,860	4,050	2,970
17	3,320	3,580	3,010	2,760	3,910	3,930	3,430	3,720	4,450	4,140	3,420	2,980	3,490	2,790	3,620	5,380	3,460
18	3,720	3,590	2,760	2,950	3,570	3,820	3,830	3,390	4,400	4,010	3,200	3,170	3,670	2,810	3,790	5,980	4,330
19	3,650	3,780	3,250	2,820	3,800	3,800	3,960	3,330	3,820	3,690	3,780	3,040	3,500	3,050	4,150	6,670	6,630
Average	3,647	3,864	2,886	2,850	3,828	3,728	4,025	3,828	4,326	4,160	3,278	3,151	3,278	3,087	3,913	5,258	3,921
Grand average all heights	3,647	3,827	3,000	2,873	3,864	3,612	3,871	3,689	4,103	4,002	3,137	3,049	3,250	3,182	3,880	4,851	--------
SPECIMENS TAKEN FROM MIXER																	
	3,450	--------	2,670	--------	3,800	--------	3,280	--------	3,630	--------	2,430	--------	2,930	--------	4,000	--------	--------
	3,660	--------	2,670	--------	3,830	--------	3,820	--------	4,330	--------	3,000	--------	2,910	--------	4,360	--------	--------
	--------	--------	--------	--------	3,800	--------	3,610	--------	3,920	--------	2,670	--------	3,170	--------	4,180	--------	--------
Average	3,555	--------	2,670	--------	3,810	--------	3,570	--------	3,960	--------	2,700	--------	3,003	--------	4,180	--------	--------

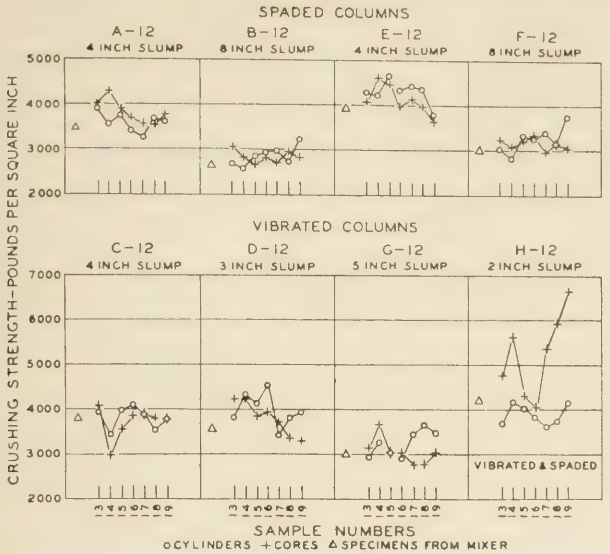


FIGURE 10.—RESULTS OF TESTS TO DETERMINE UNIFORMITY OF STRENGTH OF CONCRETE IN 12-FOOT COLUMNS

case, that is, series H, is there serious disagreement. In this series, in which the concrete was both vibrated and spaded, the tests on the cores gave in general much higher strengths than those on the cylinders. This series contains the driest concrete used in the experiment, 2-inch slump, and it is possible that under these conditions vibration consolidated the concrete sufficiently to increase the strength of core specimens considerably beyond the results obtained on the cylinders. In no other series did the cores from the vibrated concrete show appreciably higher results than the cylinders. As a matter of fact the average core strengths, as shown by Table 6 and in Figure 11, were somewhat lower. The conclusion is reached, therefore, that, in general, vibration did not increase the crushing strength of the concrete. Also, there appears to be no general tendency for either increased or decreased strength in the tops of the columns. This checks the results of the segregation tests and further indicates that no serious segregation developed under these conditions.

In Figure 11 are shown the average results of tests for strength as well as the average water-cement ratios

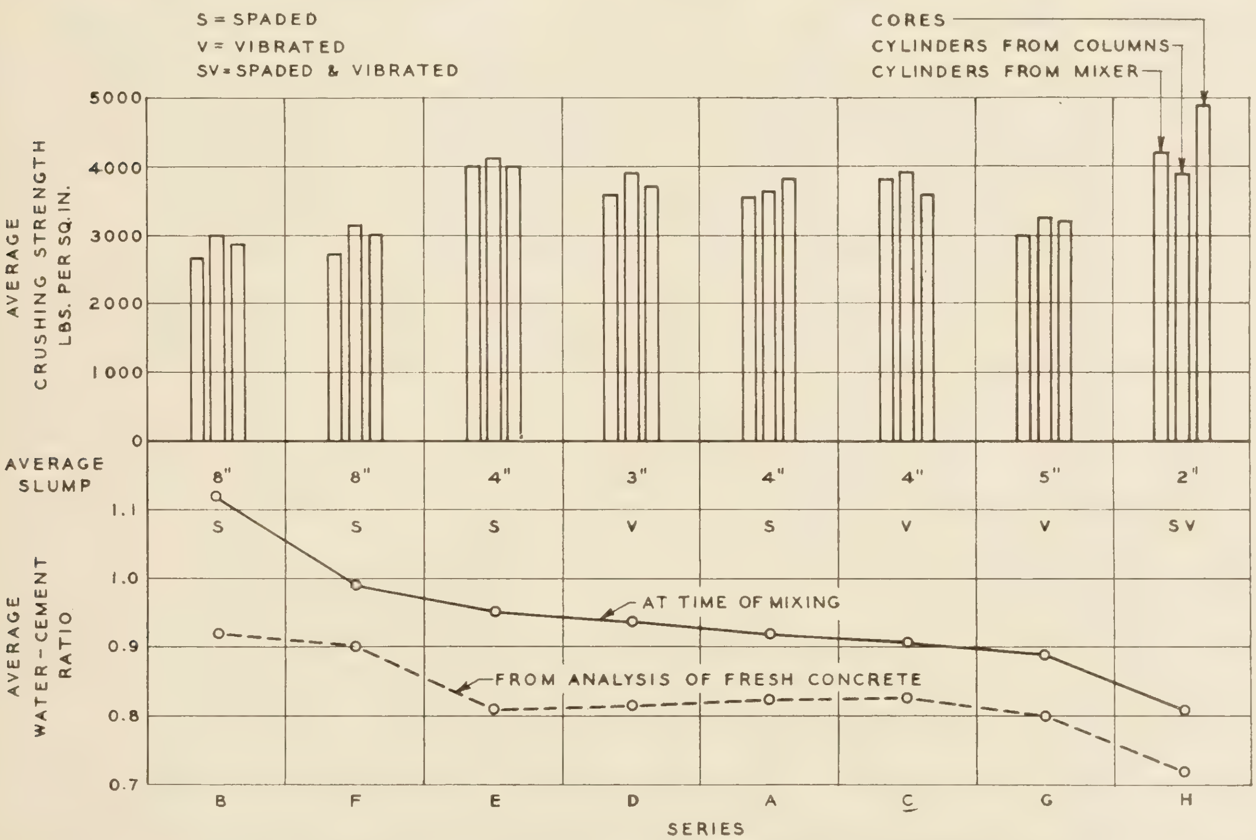


FIGURE 11.—RELATION BETWEEN STRENGTH OF CONCRETE TAKEN FROM MIXER AND THAT TAKEN FROM COLUMNS. FIGURE ALSO SHOWS WATER-CEMENT RATIOS OBTAINED AT TIME OF MIXING AND THOSE OBTAINED FROM ANALYSIS OF FRESH CONCRETE

should be called to the fact that results of tests on individual specimens are shown and that therefore the same degree of concordance should not be expected as when average results representing a number of individual tests are given.

In spite of this, the results seem to be reasonably concordant. Attention may be called especially to the agreement between the results of tests on the cylinders as compared to the corresponding cores. In only one

for each series of columns. With the exception of the values for water-cement ratio as determined at the mixer (that is, the amount of water that went into the concrete with proper allowance for moisture in the aggregates) the data are taken from Tables 4 and 5. Each value represents the average of all of the individual determinations made on each of the four columns making up the series. With one or two exceptions, then,

(Continued on p. 72)

GASOLINE TAXES, 1931

Tax earned on motor vehicle fuel, etc., refunds, disposition of fund, and gallons taxed, during full calendar year, 1931

[From reports of State authorities]

State	Gross tax assessed prior to deduction of refund	Exemption refund (deducted from gross tax)	Net tax earning on motor vehicle fuel [1]	Other receipts, under tax law (licenses, etc.)	Grand total earning (tax and other receipts)	Disposition of grand total earning, according to law: Collection and administration cost [2]	Construction and maintenance on rural roads: State highways [3]	Construction and maintenance on rural roads: Local roads [3]	State and county road bond payments [4]	On city streets	For other than highway purposes	Tax rate, 1931: Cents per gallon, Jan. 1	Cents per gallon, Dec. 31	Date of rate change	Gasoline, or other fuel for motor vehicles, taxed: Net gallons taxed	Percentage change [5]
Alabama	$7,197,474		$7,197,474		$7,197,474	$16,644	$2,204,117	$3,248,655	$1,728,058			4	5	7/28	162,670,816	−5.7
Arizona	3,595,902	$391,614	3,204,288	$32	3,204,320	(6)	2,227,256	977,064				4	5	1/30	64,701,865	−3.1
Arkansas	6,956,952	508,903	6,448,049		6,448,049	[7] 20,000	[7] 200,000	787,846	[8] 5,282,399	$107,804	[9] $50,000	5	6	2/26	110,579,175	−14.0
California	44,588,156	4,724,519	39,863,637		39,863,637	[10] 124,632	26,467,700	13,233,850			[11] 37,455	3	3		1,328,787,915	14.3
Colorado	7,059,749	805,411	6,254,338		6,254,338	[12] 67,379	4,330,871	1,670,479		185,609		4	4		156,358,455	1.8
Connecticut	4,732,734	[13] 4,741	4,727,993	48,759	4,776,752	(14)	4,776,752					2	2		236,399,661	5.9
Delaware	1,141,921	69,860	1,072,061		1,072,061	(15)	533,881		[16] 538,180			3	3		35,735,365	5.8
Florida	14,986,170		14,986,170	32,050	15,018,220	[17] 32,050	5,781,048	423,555	[18] 5,781,047		[19] 3,000,520	6	7	8/1	235,057,035	3.5
Georgia	13,313,500		13,313,500		13,313,500	4,200	8,872,867	2,218,217			[20] 2,218,216	6	6		221,891,668	−0.6
Idaho	2,953,992	355,626	2,598,366	[21] 11,061	2,609,427	[22] 30,355	2,325,833		[23] 242,594		[24] 10,645	5	5		[25] 51,967,321	−4.5
Illinois	30,495,596	1,429,911	29,065,685		29,065,685	106,834	19,305,901	9,652,950	(26)			3	3		968,856,165	5.8
Indiana	19,258,819	1,224,266	18,034,553	8,972	18,043,525	66,942	13,482,437	3,370,609		1,123,537		4	4		450,863,830	5.1
Iowa	12,378,724	1,451,135	10,927,589		10,927,589	62,956	6,372,372	4,492,261				3	3		364,252,984	3.2
Kansas	11,383,674	3,312,789	8,070,885		8,070,885	(27)	6,270,885	1,800,000				3	3		269,029,495	−11.5
Kentucky	8,810,130		8,810,130	2,864	8,812,994	30,987	8,782,007					5	5		176,202,606	4.7
Louisiana	9,398,107	324	9,397,783		9,397,783	62,000	4,791,951		2,664,276		[28] 1,879,556	5	5		187,955,663	1.7
Maine	4,609,316	226,588	4,382,728	[29] 56,647	4,439,375	32,772	2,203,302	2,203,301				4	4		109,568,209	6.6
Maryland	7,796,063	365,061	7,431,002		7,431,002	20,549	5,868,362			1,467,091	[30] 75,000	4	4		185,775,062	6.3
Massachusetts	15,573,815	267,439	15,306,376		15,306,376	(31)	12,535,626	902,267		1,868,483		2	3	5/1	558,555,950	5.8
Michigan	23,908,116	2,075,769	21,832,347	[32] 40,293	21,872,640	54,639	14,707,047	4,070,661	3,000,000	(33)	[34] 40,293	3	3		727,744,907	0.7
Minnesota	12,542,814	1,472,655	11,070,159		11,070,159	(35)	7,380,106	3,690,053				3	3		369,005,304	6.9
Mississippi	5,882,264		5,882,264	[36] 127,172	6,009,436	[37] 11,281	2,901,625	2,871,310			[38] 225,220	5	5½	11/1	115,640,338	−14.9
Missouri	9,475,871	269,307	9,206,564		9,206,564	49,189	9,157,375		(39)			2	2		460,328,204	6.6
Montana	3,801,888	783,734	3,018,154		3,018,154	27,147	2,970,346		20,661			5	5		60,363,076	2.6
Nebraska	9,176,004	79,756	9,096,248		9,096,248	11,250	6,813,749	2,271,249				4	4		227,406,207	0.4
Nevada	905,533	127,615	777,918		777,918	(40)	777,918					4	4		19,447,944	15.2
New Hampshire	2,720,395	63,252	2,657,143		2,657,143	(41)	1,992,857		664,286			4	4		66,428,585	6.3
New Jersey	17,141,415	16,783	17,124,632	48,215	17,172,847	18,600	11,636,747		427,500	5,000,000	[42] 90,000	3	3		570,821,076	4.4
New Mexico	2,664,704		2,664,704	20,873	2,685,577	[43] 60,110	1,686,034		928,933		[44] 10,500	5	5		53,294,084	−2.0
New York	31,422,861	878,800	30,544,061	45,095	30,589,156	[45] 50,000	22,904,367	6,107,831			[46] 1,526,958	2	2		1,527,203,055	6.2
North Carolina	14,371,270	346,967	14,024,303		14,024,303	9,370	4,379,478	2,639,474	[47] 6,400,509		[48] 595,472	5	6	4/1	249,609,024	−0.4
North Dakota	3,149,024	1,118,786	2,030,238	2,217	2,032,455	[49] 33,237	1,332,351	666,175			[50] 692	3	3		67,674,591	3.1
Ohio	41,272,028	1,943,975	39,328,053		39,328,053	107,956	22,061,305	9,805,024		7,353,768		4	4		983,201,323	6.1
Oklahoma	11,674,218	8,786	11,665,432		11,665,432	71,552	7,249,008	2,416,302			[51] 1,928,570	4	5	3/25	252,483,145	−16.5
Oregon	6,935,821	748,903	6,186,918		6,186,918	18,300	6,168,618		(52)			4	4		[53] 155,032,787	0.05
Pennsylvania	32,929,982	477,305	32,452,677	[54] 735,913	33,188,590	[55] 765,125	23,423,209	5,563,743	3,436,513			3	3		1,081,755,912	16.5
Rhode Island	1,963,128	70,493	1,892,635	3,640	1,896,275	(56)	1,422,206		474,069			2	2		94,631,763	9.3
South Carolina	7,274,440	28,451	7,245,989		7,245,989	(57)	4,796,512	1,207,665	[58] 1,241,812			6	6		120,766,492	1.4
South Dakota	5,157,175	1,762,500	3,394,675		3,394,675	[59] 48,602	3,346,073		(60)			4	4		84,866,877	−3.1
Tennessee	11,461,023		11,461,023		11,461,023	57,305	4,209,658	2,994,786	[61] 4,199,274			5	[62] 7	12/19	206,707,098	−3.6
Texas	33,462,890	2,948,332	30,514,558		30,514,558		22,885,918				[63] 7,628,640	4	4		762,863,942	3.3
Utah	2,311,734	2,507	2,309,227	451	2,309,678	4,214	2,305,464		(52)			3½	4	5/12	60,362,698	0.4
Vermont	1,966,544		1,966,544		1,966,544	(64)	1,562,638		278,906		[65] 125,000	4	4		49,163,593	4.6
Virginia	12,210,867	765,652	11,445,215		11,445,215	(66)	8,011,651	3,433,564				5	5		228,904,309	6.2
Washington	12,313,127	1,280,665	11,032,462		11,032,462	(67)	8,587,166	2,445,296				3	5	4/1	244,529,589	1.1
West Virginia	5,665,218	278,001	5,387,217	8,005	5,395,222	14,777	2,054,573		3,325,872			4	4		134,680,421	0.5
Wisconsin	16,471,048	690,867	15,780,181	(68)	15,780,181	20,469	8,830,273	4,514,493	[69] 1,797,646	617,300		2	4	4/1	431,504,707	3.8
Wyoming	1,587,014		1,587,014		1,587,014	5,894	1,129,841	395,279	56,000			4	4		39,675,361	9.7
District of Columbia	1,740,022	13,726	1,726,296		1,726,296					1,726,296		2	2		86,314,800	7.9
Total			536,397,458	1,192,259	537,589,717	2,117,317	354,017,281	100,073,959	[70] 42,488,535	19,449,888	19,442,737	Weighted average rate, 3.48 cents			15,407,650,452	4.4

[1] Net gasoline tax earned after deduction of refunds allowed by law.

[2] Many States pay collection cost from other State funds, and such are noted. Administration cost here includes balances allocated to reserve funds for administrative purposes, and amounts are noted.

[3] Since this table covers the calendar year earnings, but not the actual collections during the year, these columns are not comparable with similar columns in the financial tables, F-1 and F-4, issued by this bureau, which cover different periods fixed by State and local agencies; also certain funds are allocated to bond payments shown in next column.

[4] Payments are for State highway bonds, except as noted.

[5] Shows per cent increase or decrease (−) compared to net gallons reported in previous year.

[6] Paid from State highway budget, $16,556.

[7] Estimates reported.

[8] Includes payments on county bonds, $3,395,828.

[9] For expenses of motor-vehicle department.

[10] Includes $61,857 for next 6 months' expenses, in reserve.

[11] For expenses of transportation license tax division.

[12] Includes all expenses of State inspector of oils, and special expenses for shale-oil investigation in cooperation with U. S. Bureau of Mines.

[13] Excludes tax on 11,254,626 gallons exempted when purchased.

[14] Expenses of $30,000 paid from motor-vehicle fees.

[15] Paid from State treasury funds.

[16] Includes $349,355 as payments on county road bonds.

[17] Includes $11,690 gasoline tax reserve fund, assigned to administration.

[18] Payments on county road bonds.

[19] Consists of $200,000 to permanent fund for State buildings of higher learning, $1,917,773 for county schools, and $882,747 to State general revenue fund.

[20] To an equalization fund for public schools.

[21] Includes $10,645 collected on 212,905 gallons sold for airplanes.

[22] Includes $17,007 to reserve for refunds.

[23] To State treasury note redemption fund.

[24] To an aviation fund, being amount of aviation gas tax.

[25] Excludes 212,905 gallons sold to airplanes and taxed.

[26] County bond payments from gasoline tax included in local roads.

[27] Paid from State general fund, $15,000.

[28] Consists of $939,778 for State board of eduction, and $939,778 for boards of commissioners of ports of New Orleans and Lake Charles Harbor.

[29] Consists of 1 cent tax on all sales of gasoline not used by motor vehicles.

[30] To conservation department for oyster propagation.

[31] Paid from State general fund, $22,500.

[32] Consists of taxes on gas used in aeronautics and aeronautic licenses.

[33] Amount for city streets reported under State highways.

[34] Consists of $38,108 from tax on gas used in aeronautics assigned to State aeronautic fund, and $2,185 collected from licenses to State general fund.

[35] Paid from State general fund, $13,000.

[36] Special taxes collected in two counties for sea-wall financing.

[37] Includes $5,881 to State accounting department for auditing.

[38] For sea-wall protecting road. Derived from extra gas taxes in Harrison County (2 cents) and Hancock County (3 cents) shown in other receipts, $127,172; and the remainder from State highway system share of gas tax receipts.

[39] Amount assignable to sinking fund for bond payments, included in State highway fund column.

[40] Paid by tax commission which collects taxes: Amount not reported.

[41] Paid by motor-vehicle department: Amount not reported.

[42] For inland waterways under Department of Commerce and Navigation.

[43] Includes balances in suspense fund and operating fund of $28,318.

[44] Loans to motor-vehicle department and public-auditing department.

[45] Paid from State general fund, $75,463. Amount shown is for reserve to pay refunds.

[46] To New York City general fund.

[47] Includes $1,381,146 payments on county road bonds.

[48] For State highway patrol and administration expenses of State revenue department.

[49] Includes $8,237 to reserve for refunds.

[50] Dealers' license fees credited to State general fund.

[51] Consists of $1,000,000 special emergency relief for destitute, and remainder for distribution of field and garden seed.

[52] Payments reported from motor-vehicle fees, instead of assigning a pro rata share to gasoline taxes.

[53] Includes approximately 3,118,610 gallons of distillate taxed at 3½ cents per gallon.

[54] Including expenses of department of revenues prorated to gasoline taxes.

[55] Consists of delinquent collections of previous years, penalties, and fines.

[56] Paid from motor-vehicle fees, $15,737.

[57] Paid from State tax commission appropriation: Amount not reported.

[58] Includes all payments on State highway bonds, as pro rata share not reported in disposition of motor-vehicle receipts.

[59] Includes $31,602 to reserve for refunds.

[60] All State highway bonds retired on Jan. 15, 1931, by funds previously accumulated.

[61] Includes county road bond payments $2,089,184.

[62] A tax of 6 cents was effective July 1, 1931 to Dec. 19, 1931.

[63] For free-school fund.

[64] Paid from motor-vehicle department appropriation, estimated $2,000.

[65] Transfer to motor-vehicle department.

[66] Paid by State appropriation, $14,096.

[67] Paid from motor-vehicle fund, $10,000.

[68] Dealers' license fees paid into general funds.

[69] Payments on county road bonds.

[70] Includes payments on county road bonds, $27,451,735, on State highway bonds, $14,794,206 and in note fund, $242,594.

(Continued from p. 63)

50 feet to the right of the center line at station 17 + 00 before drainage revealed a depth of muck of 6 feet. Nine months later this had consolidated to 4.75 feet. This represents a loss of 54,450 cubic feet per acre.

Figure 8 shows a series of photographs taken at or near station 10 + 00 at different stages of the work, from the construction of a surface drainage ditch to the completion of the road surface.

As a result of the drainage this soil, which a few months before failed to support the power shovel on rafts, carried heavily loaded material trucks and grading was completed without difficulty.

Thus, this soil was improved by the very means which might have been the least effective under other circumstances.

The method employed was successful in this instance only because of the peculiar topographic features encountered in combination with certain physical properties possessed to such a marked degree by the muck soil. The decision as to proper methods of stabilization was possible only as a result of thorough knowledge of all the factors involved.

(Continued from p. 69)

each value represents the average of nineteen individual tests. This chart indicates quite clearly that the crushing strength of the concrete as determined by cylinder tests on samples taken as the mixer was discharged approximated very closely the actual average strength of the concrete in the column, as determined either by molding cylinders of concrete taken from the forms or by means of cores drilled from the hardened concrete.

The data also indicate that the average net water-cement ratio as determined by the Dunagan analysis approximately one hour after the concrete had been placed was about 0.1 less than the net water-cement ratio at the time of mixing.

The results of strength tests of cores on the one 12-foot column (series K) are shown in Figure 12. As previously explained this column was vibrated, but in a more severe manner than any other series. The concrete going into the bottom of this form (represented by core 19) appeared dry, so the consistency was changed from a 2-inch slump to a 4-inch slump after about 2 feet of concrete had been placed. While this change in water-cement ratio would naturally result in a higher strength at the bottom of the column than at the top, the difference in strength is greater than would be expected from this cause alone. The severe vibration is probably responsible for some of this difference for it will be observed that the strength increases from a point 9 feet from the bottom to a maximum at a point 1 foot from the bottom. It also increases from the 9-foot mark as the top is approached. In series H the curve for the 12-foot column has the same general trend. (See fig. 10.) It would appear from the above that the strength of the concrete is not appreciably affected by compacting it either by spading or moderate vibration as was done on series A to series G, inclusive. When the two methods are combined, or when the vibration is of a more severe nature, as in series H and series K, a marked effect is produced.

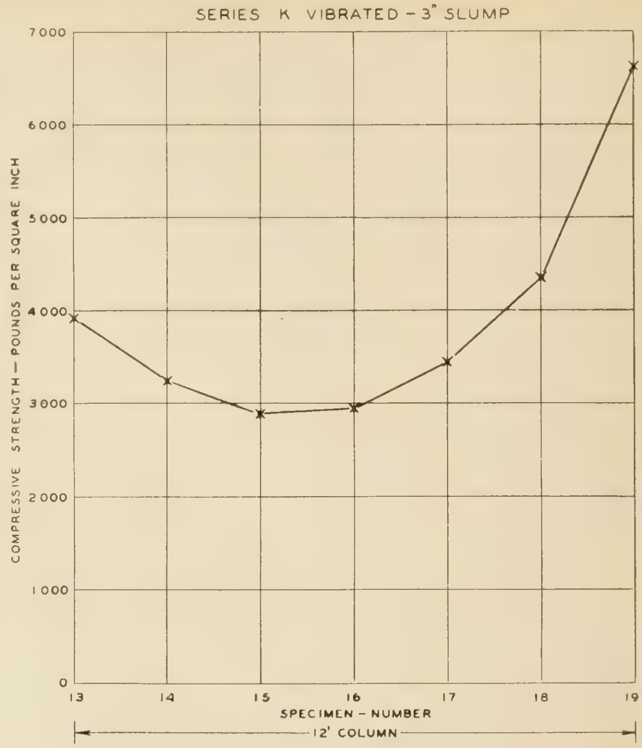


Figure 12.—Compressive Strength of Cores Obtained From Series K, Which was Subjected to Severe Vibration

CONCLUSIONS

The conclusions derived from this study are naturally restricted in their application to conditions comparable to those under which the tests were conducted. These conditions include a range in slump from 2 to 8 inches; placing by both spading and vibratory methods; forms 8 inches in width; and a fine aggregate of relatively fine sand. The conclusions are:

1. No serious segregation of concrete or water gain took place. This conclusion applies for a range in slump varying from 2 to 8 inches and for both the spading method and the vibratory method of placing. In this connection, attention is called to the fact that forms only 8 inches in width were employed in these tests and also that a relatively fine sand was used. Both of these factors probably contributed considerably to the absence of serious segregation.

2. With the exception of the one series of tests in which the concrete was both spaded and vibrated (series H), the method of placing by vibration produced concrete having about the same average strength and uniformity as similar concrete placed by spading.

3. A combination of spading and vibration appeared to increase the strength of the concrete considerably beyond the strength obtained by either spading or vibrating alone.

4. With the exception of one series of tests (series H), the average strength of concrete as determined by cores drilled from the columns was approximately the same as the strength of the concrete determined by tests on samples taken as the mixer was discharged.

5. The net water-cement ratio taken approximately one hour after the concrete had been placed averaged about 0.1 less than the net water-cement ratio at time of mixing.

The bureau wishes to acknowledge the courtesy of the Electric Tamper & Equipment Co., Ludington, Mich., for the loan and operation of the electric vibrators used in the work.

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ROAD PUBLICATIONS *of the* BUREAU OF PUBLIC ROADS

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Report of the Chief of the Bureau of Public Roads, 1925.

Report of the Chief of the Bureau of Public Roads, 1927.

Report of the Chief of the Bureau of Public Roads, 1928.

Report of the Chief of the Bureau of Public Roads, 1929.

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Report of a Survey of Transportation on the State Highways of Vermont. (1927.)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927.)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928.)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928.)

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States. (1930.)

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

MAY 31,1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	MILEAGE			Estimated total cost	Federal-aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
		$	$				$	$				$	
Alabama	2,393.9	1,812,011.04	884,109.47	74.1		74.1						5,469,694.24	Alabama
Arizona	1,232.8	2,367,513.11	1,701,787.22	117.1	27.9	145.0	513,875.38	386,896.72	8.9	19.4	28.3	852,053.43	Arizona
Arkansas	1,960.1	2,537,870.43	1,169,880.01	64.8	45.3	110.1	395,313.38	197,045.12	17.3		17.3	2,144,072.23	Arkansas
California	2,301.1	10,489,263.29	4,781,591.52	224.3	38.6	262.9	3,005,968.60	1,345,216.58	53.9	17.0	70.9	151,622.60	California
Colorado	1,583.4	4,457,020.37	2,331,213.18	183.5	50.3	233.8	203,412.66	114,236.51	8.1	.5	8.6	2,097,113.09	Colorado
Connecticut	282.0	4,034,661.02	1,467,917.99	29.4		29.4	563,726.40	263,977.96	7.7		7.7	450,266.59	Connecticut
Delaware	345.2	808,757.25	404,378.62	34.0		34.0	162,192.00	81,096.00	12.4		12.4	167,140.82	Delaware
Florida	617.5	4,232,614.05	1,935,392.29	111.2		111.2	410,013.79	205,006.88	14.9		14.9	2,797,004.58	Florida
Georgia	3,062.8	6,488,979.48	3,031,546.68	153.5	171.2	324.7	800,640.68	397,410.33	14.5	22.2	36.7	1,284,326.84	Georgia
Idaho	1,474.4	2,301,263.46	1,336,548.68	117.4	80.7	198.1	912,173.80	551,669.33	19.9	164.2	184.1	753,666.09	Idaho
Illinois	2,640.2	25,039,271.01	11,663,206.91	748.9	32.8	781.7	4,558,209.19	1,933,042.38	146.9	8.8	155.7	1,224,027.26	Illinois
Indiana	1,788.5	10,089,248.33	5,001,315.41	324.4	16.1	340.5	2,928,955.25	1,447,252.23	139.7	4.4	144.1	277,921.19	Indiana
Iowa	3,376.4	2,550,856.28	1,190,093.28	148.6	14.2	162.8	2,428,843.53	1,141,055.93	107.1	39.8	146.9	468,655.99	Iowa
Kansas	3,670.8	4,815,729.58	2,353,680.03	292.8	29.9	322.7	2,300,589.40	1,112,906.07	85.9	228.6	314.5	871,135.30	Kansas
Kentucky	1,885.0	2,553,780.56	1,132,511.46	137.9	4.0	141.9	1,241,059.97	519,790.62	43.1	84.9	128.0	1,337,204.72	Kentucky
Louisiana	1,566.7	7,471,657.46	3,511,759.18	86.9	10.6	97.5						847,317.82	Louisiana
Maine	708.4	2,504,257.11	1,062,976.98	50.4		50.4	1,940,190.50	895,232.96	75.7		75.7	162,080.43	Maine
Maryland	776.8	590,183.00	223,974.80	20.5		20.5	1,222,453.98	573,117.51	41.7	6.2	47.9	187,216.51	Maryland
Massachusetts	814.8	7,767,729.70	3,206,471.58	73.7	.1	73.8						566,527.21	Massachusetts
Michigan	2,048.5	8,573,575.70	3,934,374.90	393.4	33.1	426.5	691,240.18	336,705.00	35.6	7.5	43.1	2,859,371.63	Michigan
Minnesota	4,031.9	8,104,776.42	2,713,578.48	162.0	216.0	378.0	4,022,137.30	1,265,722.90	58.6	101.0	159.6		Minnesota
Mississippi	1,797.0	4,157,313.62	2,039,029.74	170.3	75.4	245.7	44,671.02	22,335.51	6.4		6.4	5,210,985.59	Mississippi
Missouri	2,960.4	4,545,016.92	2,036,567.15	144.6	38.0	182.6	3,738,561.17	1,546,441.89	80.3	62.2	142.5	513,591.48	Missouri
Montana	2,704.1	4,660,149.39	2,620,258.74	403.1	31.5	434.6	182,379.80	105,440.57	20.5	4.2	24.7	3,443,744.79	Montana
Nebraska	4,234.6	3,380,123.81	1,678,560.14	159.9	40.4	200.3	327,120.69	163,560.33	11.5	4.5	16.0	2,538,206.45	Nebraska
Nevada	1,291.4	1,633,286.93	1,317,427.96	38.7	135.3	174.0	851,343.37	686,563.38	3.3	54.4	57.7	266,079.89	Nevada
New Hampshire	418.4	625,542.30	246,912.44	12.1	2.5	14.6						553,268.42	New Hampshire
New Jersey	605.7	3,760,915.32	1,327,092.02	36.3	.2	36.5						1,592,693.57	New Jersey
New Mexico	2,222.8	1,942,277.87	1,275,777.00	89.3	17.7	107.0	1,016,475.46	647,378.40	41.1	6.5	47.6	883,167.09	New Mexico
New York	3,265.8	13,180,149.00	5,945,419.50	306.7		306.7	6,004,600.00	2,322,550.00	117.1		117.1	2,849,008.54	New York
North Carolina	2,222.8	1,386,104.89	670,155.41	76.1		76.1	338,402.82	169,201.39	7.1	9.0	16.1	3,710,893.71	North Carolina
North Dakota	5,017.6	3,258,451.21	1,631,733.01	353.7	249.1	602.8	1,920,049.65	978,007.79	189.8	448.2	638.0	1,187,254.68	North Dakota
Ohio	2,854.7	6,477,066.69	2,284,512.94	97.2	18.4	115.6	6,376,445.01	2,324,838.24	150.4	51.8	202.2	1,894,280.38	Ohio
Oklahoma	2,266.5	4,127,708.92	1,976,092.57	164.5	50.1	214.6	1,097,682.01	553,645.37	84.3	6.8	91.1	1,629,144.80	Oklahoma
Oregon	1,519.5	4,384,818.51	2,451,443.55	131.4	47.1	178.5	1,279,991.81	698,815.63	29.1	56.2	85.3	1,031,724.38	Oregon
Pennsylvania	3,012.6	2,011,139.61	824,066.79	48.8		48.8	7,548,654.14	3,467,301.85	259.2		259.2	1,657,527.60	Pennsylvania
Rhode Island	255.4	895,613.98	443,694.82	16.6	2.2	18.8	122,356.08	61,178.04	.2		.2	301,519.98	Rhode Island
South Carolina	2,022.6	2,620,162.19	1,154,390.02	55.1	50.7	105.8	337,792.67	168,242.45	18.1		18.1	881,417.68	South Carolina
South Dakota	4,028.4	3,495,987.17	1,976,474.77	239.9	181.9	421.8	276,819.41	191,436.52	49.0	6.7	55.7	1,164,012.22	South Dakota
Tennessee	1,678.6	1,363,759.03	665,020.93	52.5	6.4	58.9	413,992.12	206,996.03	4.5	5.5	10.0	3,376,953.33	Tennessee
Texas	7,721.9	14,491,718.55	6,595,600.93	686.5	199.4	885.9	4,602,766.32	2,085,024.90	252.0	217.9	469.9	3,853,134.14	Texas
Utah	1,213.7	1,114,052.85	799,096.25	91.4	6.6	98.0	669,624.92	495,522.31	19.4	107.6	127.0	750,154.84	Utah
Vermont	339.1	1,244,646.56	568,891.61	39.2		39.2	171,773.36	65,419.72	6.5		6.5		Vermont
Virginia	1,925.9	1,880,674.52	940,337.19	108.4	.1	108.5	317,554.02	151,912.13	14.5	8.7	23.2	2,003,328.75	Virginia
Washington	1,183.1	3,031,135.35	1,495,394.35	106.4	17.7	124.1	326,289.53	176,000.00	12.0		12.0	1,170,143.83	Washington
West Virginia	889.8	1,679,821.81	720,327.72	37.8	10.5	48.3	548,482.63	261,406.32	16.4		16.4	978,250.89	West Virginia
Wisconsin	2,671.2	4,503,175.61	1,890,331.85	154.3	9.6	163.9	2,046,550.97	822,086.27	27.8	45.1	72.9	1,164,379.63	Wisconsin
Wyoming	1,937.2	3,395,825.03	2,065,905.41	225.1	155.5	380.6	448,120.95	288,692.67	16.5	60.9	77.4	244,716.57	Wyoming
Hawaii	76.3	809,346.39	347,057.35	22.0		22.0	422,233.57	417,721.92	6.7	3.6	10.3	1,564,725.89	Hawaii
TOTALS	100,898.3	219,647,002.68	103,025,880.83	7,616.7	2,117.1	9,733.8	69,731,729.49	31,845,100.66	2,335.6	1,864.3	4,199.9	71,382,727.69	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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