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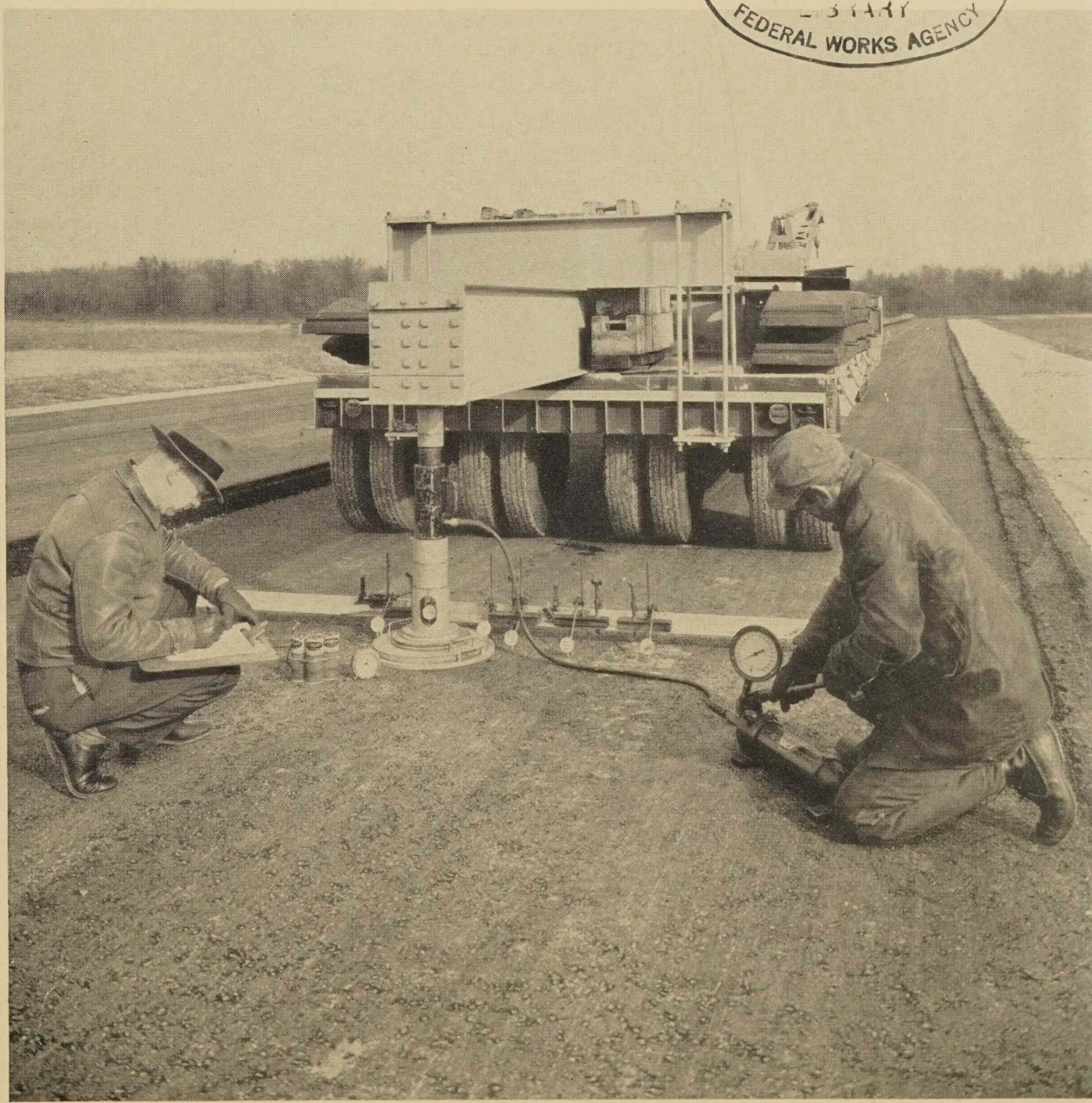

Testing the load-bearing capacity of bituminous pavement at Hybla Valley

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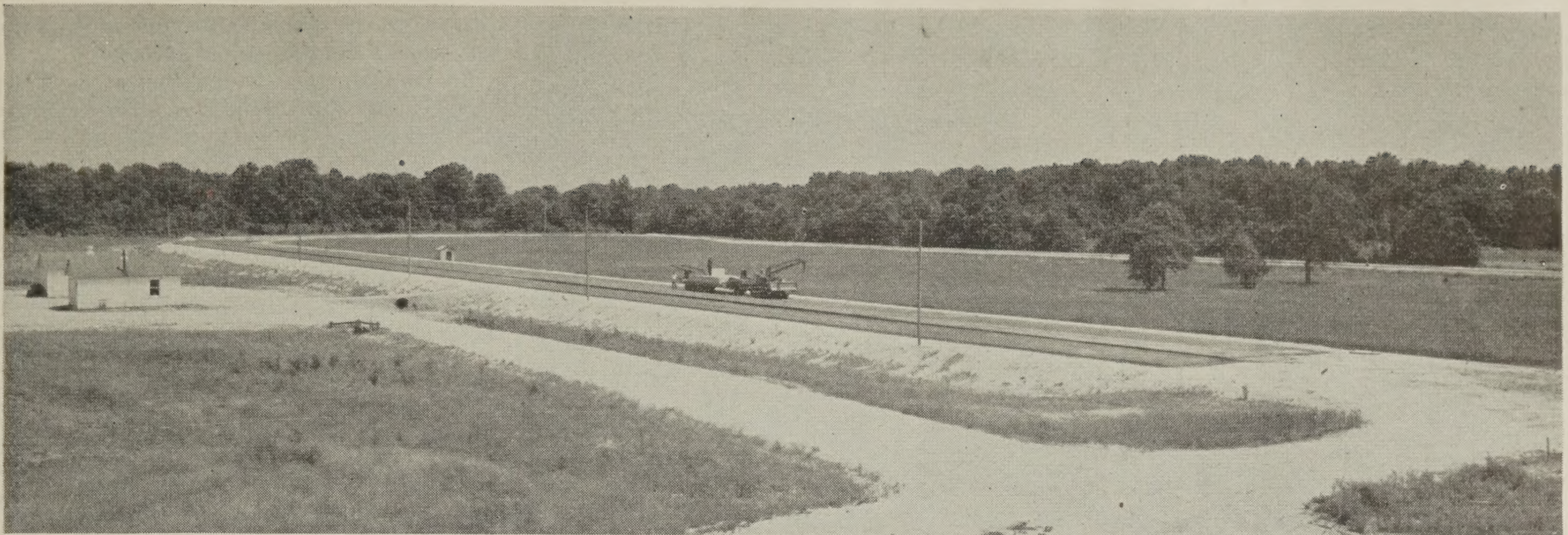

The Hybla Valley bituminous pavement test track

A Cooperative Study of Structural Design of Nonrigid Pavements

This article serves as an introduction to the cooperative investigation of the structural design of nonrigid pavements. The participating agencies have embarked on an extensive series of experiments that will result in the collection of a large body of important data and, it is hoped, in the development of significant conclusions of great value.

The principal objectives of the investigation include development of the load-supporting values of nonrigid pavements by full-scale field tests, correlation of these data with laboratory tests to determine whether the latter can be used alone in the design of pavement thickness, and correlation of the data with in-place determinations of various values of the base-course and subgrade components.

As the investigation advances it is anticipated that a series of articles will be published to report the progress o the work, the data collected, and the conclusions drawn therefrom. This introductory article provides a description of the investigation, its purposes, and methods of procedure, and will serve as background for the articles to come.

BY THE DIVISION OF PHYSICAL RESEARCH
PUBLIC ROADS ADMINISTRATION

Reported by A. C. BENKELMAN, Research Specialist
and F. R. OLMSTEAD, Senior Soils Specialist

THIS paper is an initial progress report of the investigation of nonrigid pavement design being undertaken as a cooperative project by the Highway Research Board, the Asphalt Institute, and the Public Roads Administration. The phase of the investigation covered in this report, dealing with the construction and testing of experimental sections of pavement, was planned and begun during the war period at a time when it was difficult to achieve and maintain a normal rate of progress. While some preliminary data of a highly significant character have already been secured, the present discussion is concerned primarily with a detailed description of the project. Included is a statement of objectives, a discussion of the methods of approach, a descriptive account of the construction of the test pavement, and finally a statement dealing with the development of test apparatus and testing techniques.

Interest in the problems of structural design of the bituminous or nonrigid pavement grew enormously during the war period. To determine how thick such pavements should be to accommodate heavy airplanes, research work of a scope that would never have been considered feasible in peacetime was undertaken. Most of the work was carried on under the supervision of engineers of the War and Navy Departments. In spite of the fact that much of the work had to be planned and executed in as short a period of time as possible, a great deal of useful and pertinent information was developed. It has served to bring about a much clearer perspective of the problem and has resulted in the development of several methods of thickness design that have considerable merit.

The scope of the present cooperative investigation is sufficiently comprehensive to permit examination, study, and intercorrelation of all known theories and methods of design.

The principal objectives of the investigation include:

1. The development, by means of bearing and moving-wheel load tests on full-size pavement sections in the field, of fundamental data on the load-supporting value of nonrigid pavement surfacings of various thicknesses in combination with various base-course thicknesses and degrees of subgrade support.

2. The correlation of the field data with appropriate laboratory tests for the purpose of determining whether or not tests of the latter nature may subsequently be used by themselves as a basis for a sound method for the design of the thickness of the pavement.

3. The correlation of the field data with in-place determinations of the density, moisture content, California bearing ratio, and North Dakota cone values of the base-course and subgrade components.

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Figure 1.—Aerial view of the oval test track with one tangent paved. The auxiliary test pavement is at the left of the approach road.

DESCRIPTION OF TEST PAVEMENT SECTIONS

The sections of test pavement were built outdoors with standard highway construction equipment. They are arranged in the form of an oval test track having two parallel tangents 800 feet in length connected at each end by a circular curve of 200-foot radius. The test sections themselves are confined to the tangents of the track, the connecting curves serving as a means of travel for the vehicles that will be used to apply moving loads to a portion of the width of the pavement.

The site of the experiment is near Hybla Valley, Va., on a tract of Government-owned land adjacent to U S Route 1 about 10 miles south of Washington, D. C. Views of the test area are shown in figure 1 and in the illustration at the top of page 21. The exact location for the track was selected after making a very thorough soil survey of the entire tract. While this survey revealed the presence of a fairly uniform soil formation throughout the location selected, it was decided to take every reasonable precaution to insure having a homogeneous subgrade on which to lay the test pavement. This was accomplished by building embankments of a selected soil to a height of 5 feet on the line of the tangents of the track. The equipment and methods used in the placement of the embankments are described later in this report. The moisture content of the soil was controlled within a selected range, and the material was compacted to a specified density.

The plan and profile of the completed test pavement on the north tangent of the oval track are shown in figure 2. The pavement consists of four sections, each 200 feet in length and 44 feet in width. The thickness of the base course, as indicated, ranges from 6 to 24 inches. The pavement is divided transversely into three lanes, the outer two being 16 feet in width and the center 12 feet. The thickness of the bituminous surface in the outer lanes is 3 inches and that of the center lane varies, half (100 feet) of each section being 6 inches and half 9 inches thick. Later on in this report factual data are presented relative to the design and composition of the base course and bituminous surface.

In planning the investigation as a whole, it was recognized that a considerable amount of exploratory work would have to be done in the development of test equipment and in the standardization of methods of load testing as well as of sampling and testing the different pavement components. This work necessitated the construction of a preliminary or auxiliary test pavement on which, to date, a great variety of work of an exploratory nature has been conducted.

The plan and profile of the auxiliary pavement are shown in figure 3. The total length of this pavement is 300 feet. It consists of three sections, the base and surface course thicknesses of which are indicated on the figure.

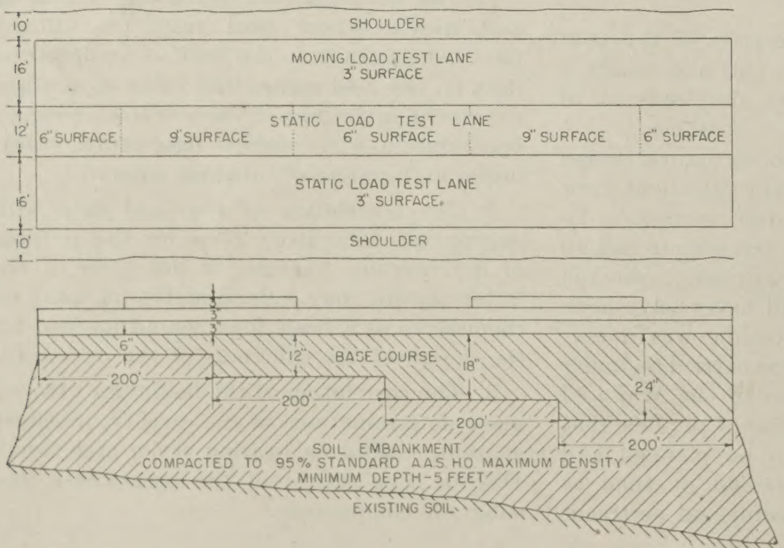
CONSTRUCTION OF TEST PAVEMENTS

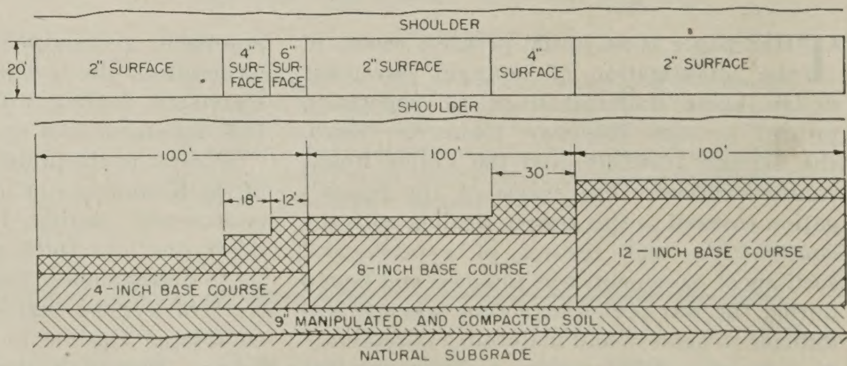
The auxiliary and oval test track pavements were built in an area containing soils of about the same physical characteristics. Tables 1 and 2 indicate typical compaction and bearing test data of the natural earth subgrade used for the auxiliary test pavement and of the selected earth borrow used for the construction of the 800-foot tangents of the oval test track. Typical test constants of the soils were as follows:

Mechanical analysis:	
passing No. 10 sieve..percent..	100
passing No. 40 sieve...do....	95
passing No. 200 sieve..do....	71
Liquid limit	51
Plasticity index	27
Shrinkage ratio	1. 9
Shrinkage limit	15
Centrifuge moisture equivalent	35
Field moisture equivalent	30
Specific gravity	2. 76

Although the soils used for the subgrades of these test pavements were quite similar, the method of design and construction was different. The subgrade of the tangents of the oval test track (fig. 2) was built as an embankment; the subgrade of the auxiliary test pavement (fig. 3) was constructed as an all cut section in the undisturbed soil.

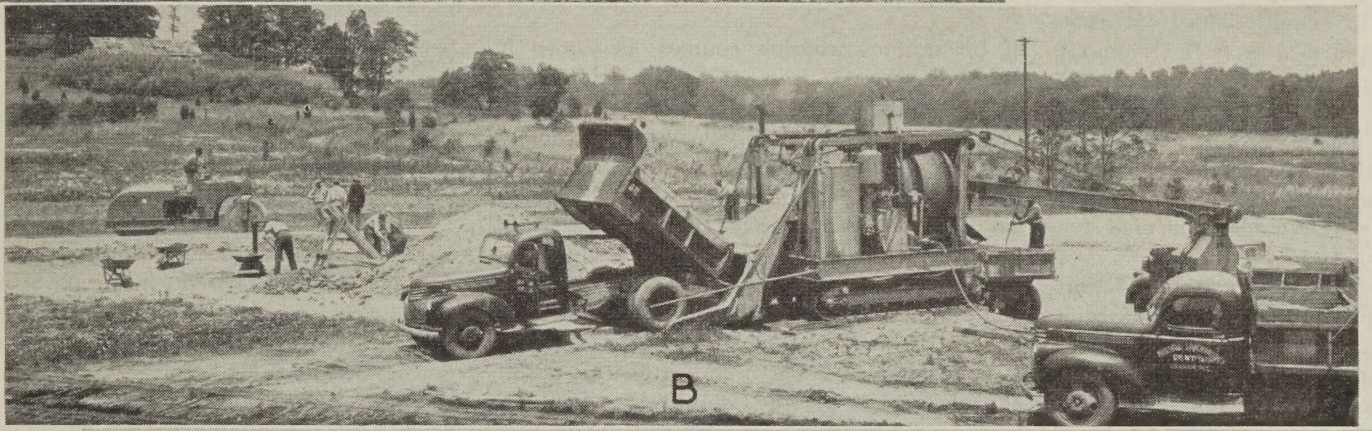
The embankments on the oval track are 5 feet high and were built with selected earth

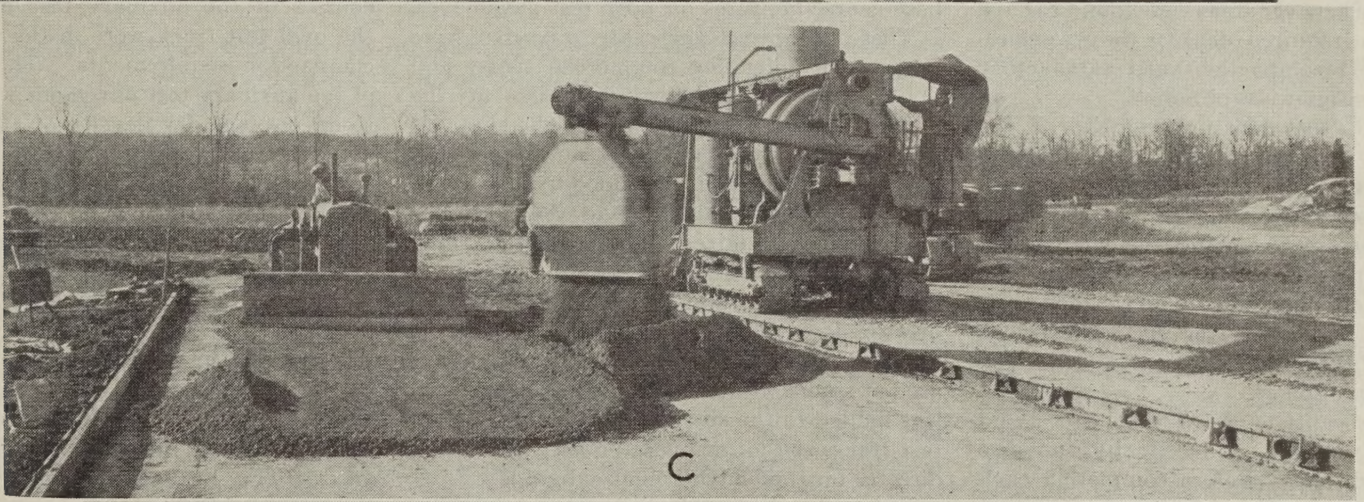


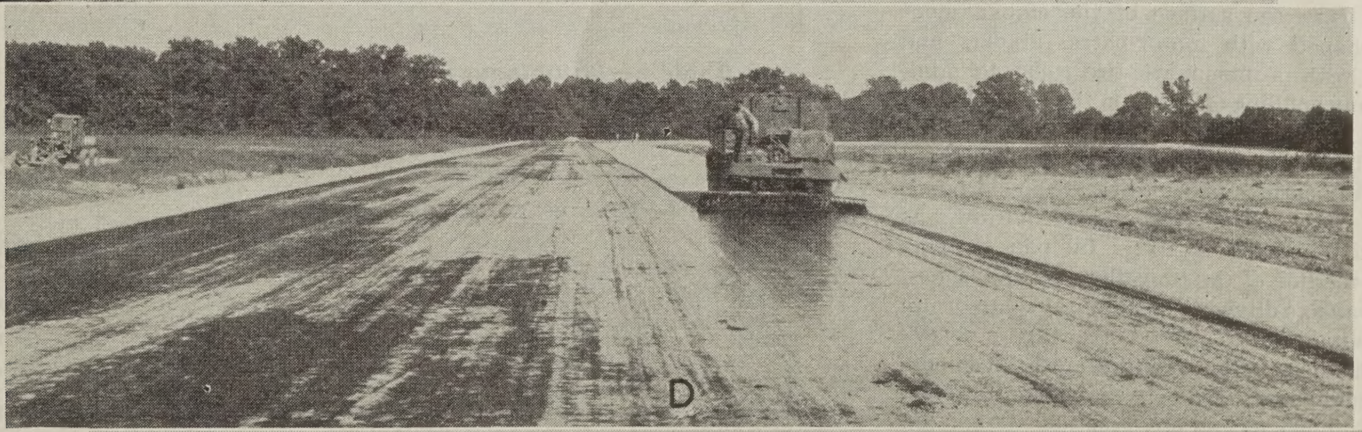


(Left) ***Figure 2.—Plan and profile, tangent section of oval test track pavement.***

(Above) ***Figure 3.—Plan and profile, auxiliary test pavement.***







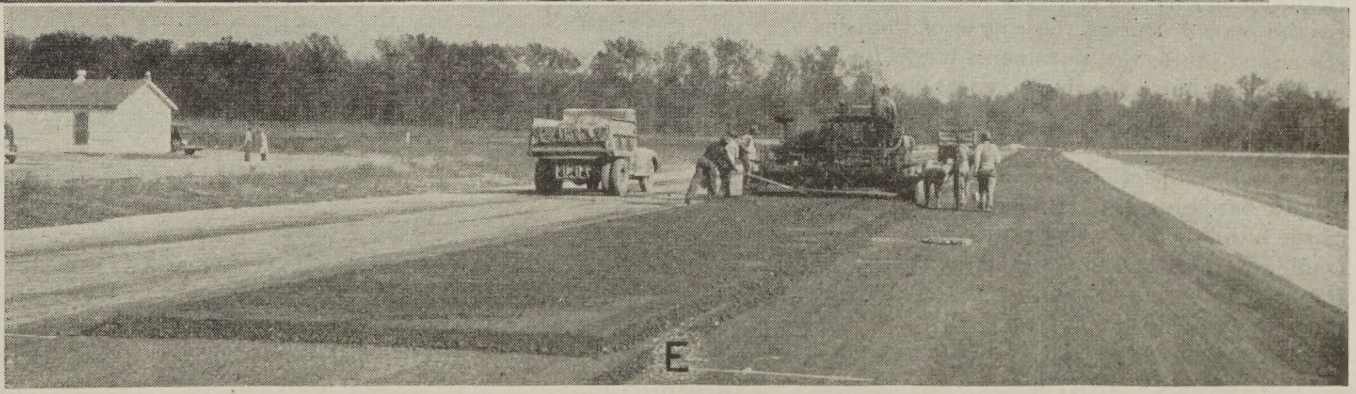


Figure 4.—Construction of the test tracks: (A) Building the oval track embankment; (B) mixing base-course materials for the auxiliary track; (C) spreading base-course materials on the oval track; (D) priming the base course; and (E) laying the bituminous surface.

Table 1.—*Typical compaction data of subgrade soil for test pavements*

Test method	Optimum moisture	Maximum dry density
	Percent	*Lb. per cu. ft.*
Standard A. A. S. H. O.	19.6	105.1
Army: [1]		
4-inch mold [2]	14.4	120.0
6-inch mold [3]	13.3	119.8

[1] Method described in U. S. Engineer Manual, March 1943.
[2] 25 blows per layer. [3] 56 blows per layer.

Table 2.—*Typical bearing test data of subgrade soil for test pavements*

Method	Bearing ratio after immersion [1]	Dry density	Moisture content [2]	Moisture content of top inch	Volume change after 96 hours
	Percent	*Lb. per cu. ft.*	*Percent*	*Percent*	*Percent*
Army	2.0	106.3	21.8	[3] 30.0	2.0
California	2.0	112.1	18.4	[3] 28.7	13.5

[1] Bearing ratio at 0.1-inch penetration.
[2] Based on dry weight. [3] 10-pound surcharge.

Table 3.—*Design characteristics of stabilized aggregate base courses*

	Specification limits [1]	Auxiliary pavement base course [2]	Oval test track base course [2]
Percent passing sieve size:			
1½ inch	100	100	100
1 inch	100	99	95
¾ inch	70–100	90	85
⅜ inch	50–80	67	67
No. 4	35–85	56	55
No. 10	25–50	47	46
No. 40	15–30	24	20
No. 200	5–15	9.5	6
Liquid limit	25 max	17	17
Plasticity index	6 max	4	3
In-place density [3]		130	136
Number tests averaged		13	70

[1] A. A. S. H. O. stabilized aggregate base-course specification B–1.
[2] Average of all tests.
[3] Dry density in pounds per cubic foot.

borrow compacted in 4-inch layers to at least 95 percent of standard A. A. S. H. O. maximum density at moisture contents less than 2 percent above the optimum. The subgrade of the auxiliary test pavement consisted of the natural subgrade with the upper 9-inch layer manipulated and compacted to the same minimum compaction requirements specified for the embankments of the oval track.

The field control of the subgrade compaction consisted of making moisture tests of soils before rolling, and in-place density and moisture tests after compaction. Tests were made for each 400 cubic yards of material compacted; and whenever areas were found to have less than the required density they were reworked and recompacted until satisfactory in-place densities were obtained.

In general, the moisture content of the natural soils was slightly above that required for satisfactory compaction. The excess soil moisture was removed during the manipulation operation before starting the compaction. The placement, manipulation, and compaction operations were planned so that a 4-inch layer of soil could be placed over the entire length and width of the 800-foot tangent during each day of construction.

Several precautions were taken to avoid construction delays induced by adverse weather conditions. Before completing each day's work the surface of the embankment was shaped with motor-patrol graders and rolled with a pneumatic-tire roller to minimize any ponding of water on the embankment in the event of a rain. Likewise, the selected earth borrow area was graded and maintained free from surface irregularities to avoid ponding of water. The selected earth borrow was removed in shallow layers by pan scrapers along the line of the prevailing ground slopes, and drainage ditches were maintained to insure adequate drainage of the borrow area.

After the subgrades for both test pavements were completed they were shaped and rerolled to the alinement and cross sections designated by the plans. During the course of the construction of these test pavements it was found necessary to finish the auxiliary test pavement prior to the completion of the oval test track. Certain experimental design data on pavement thickness, compaction of the stabilized aggregate base course, and probable performance of the bituminous wearing course under static loading were needed to plan the final design of the base and wearing courses for the 800-foot tangents of the oval test track. The auxiliary test pavement designed for this preliminary study was constructed with 4-, 8-, and 12-inch base courses with several different thicknesses of bituminous wearing courses, as shown in figure 3.

The stabilized aggregate mixture used for the base course of the auxiliary test pavement was designed to conform with the A. A. S. H. O. B–1 grading specifications for stabilized aggregates. In the case of the oval test track, however, this specification was modified to permit the use of commercial coarse aggregate containing a slight amount of oversize material (passing 1½-inch sieve, retained on 1-inch sieve). Table 3 indicates the design characteristics of the stabilized aggregate base-course mixtures for both test pavements.

These stabilized aggregate mixtures were prepared by blending commercial coarse and fine aggregates on a weight basis at the aggregate plant. After the preweighed batches of aggregates were hauled to the project site, pulverized binder soil was added to each batch from a proportioning bin and the materials were uniformly mixed in a concrete paving mixer. The moisture necessary for proper compaction was added during the mixing operations. In the construction of the auxiliary test pavement, the mixer was operated at a fixed location and the mixed material was hauled to the site in trucks. On the oval test track, the mixer operated and discharged directly on the embankment subgrade.

The stabilized aggregate base courses were built using standard construction procedures. The material was spread in 3-inch layers and compacted by means of smooth-face and pneumatic-tire rollers until the required density was obtained.

The essential differences between the base courses of the auxiliary test pavement and the oval test track were in the thickness and compaction requirements. The base course of the auxiliary test pavement was compacted to an in-place dry density of 130 pounds per cubic foot and was built in three thicknesses—4, 8, and 12 inches. The base course for the north 800-foot tangent of the oval test track was compacted to an in-place dry density of 136 pounds per cubic foot and was constructed in four thicknesses—6, 12, 18, and 24 inches.

The asphaltic prime coat which was applied to the surface of the completed base course was the same for both test pavements. MC–1 was applied at a rate of 0.25 gallon per square yard. The bituminous concrete pavement

Table 4.—*Bituminous mixture specifications for the test pavements*

Constituent	Auxiliary test pavement A–2 [1]	Oval test track pavement: Binder A–2–a II [1]	Oval test track pavement: Surface A–2–a IV [1]
MINERAL MATTER:			
Coarse aggregate: [2]	*Percent*	*Percent*	*Percent*
Passing 1½-in. sieve, retained on 1-in.	0– 5		
Passing 1½-in. sieve, retained on ¾-in.		14–48	
Passing 1-in. sieve, retained on ¾-in.	15–25		
Passing ¾-in. sieve, retained on ⅜-in.		3–45	18–50
Passing ¾-in. sieve, retained on No. 4	20–35		
Passing ⅜-in. sieve, retained on No. 4			3–36
Passing No. 4 sieve, retained on No. 10	5–15	5–15	9–22
Subtotal	60–70	55–75	50–65
Fine aggregate: [3]			
Passing No. 10 sieve, retained on No. 40		3–21	5–22
Passing No. 40 sieve, retained on No. 80		6–25	9–27
Passing No. 80 sieve, retained on No. 200		3–16	5–18
Passing No. 10 sieve, retained on No. 200	25–35		
Mineral filler passing No. 200 sieve	4– 6	4– 6	5– 8
Subtotal	30–40	25–45	35–50
Total mineral matter	100	100	100
TOTAL MIX:			
Mineral aggregate	92–95	93–95	92–94
Asphaltic cement [4]	5– 8	5– 7	6– 8
Total	100	100	100

[1] Asphalt Institute dense graded aggregate specification, hot-mix type.
[2] Crushed limestone.
[3] Natural river sand.
[4] Asphaltic cement, 85–100 penetration, Federal specification SS–A–706–b.

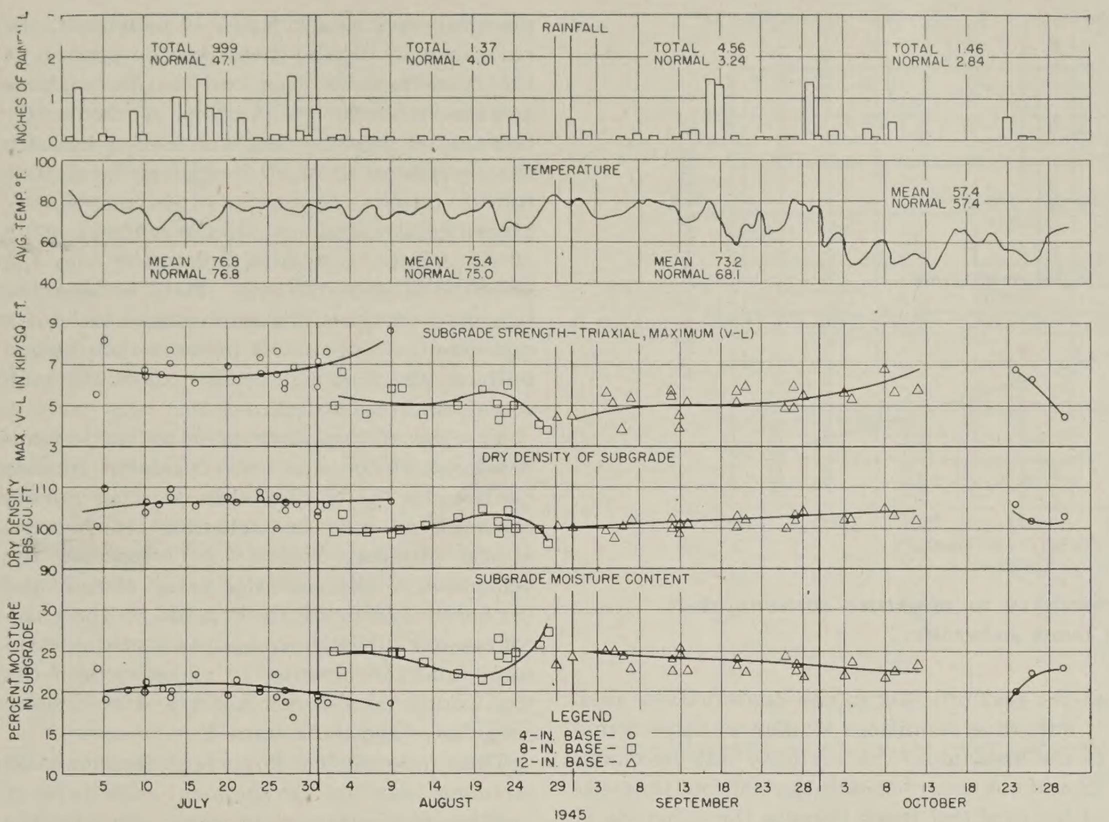


Figure 5.—Relation between weather conditions and strength, density, and moisture content of subgrade under base courses.

was placed after the prime coat had been thoroughly cured.

The bituminous concrete paving mixtures used on the auxiliary pavement and on the oval track were of the hot plant-mixed type conforming to the design requirements usually specified for highway construction. Table 4 gives the specification requirements of the two paving mixtures. The placement and compaction operations were similar to those recommended for highway construction of this pavement type. Typical views of the construction of the auxiliary and oval track pavements are shown in figure 4.

SUBGRADE STUDIES OF AUXILIARY TEST PAVEMENT

A limited study of the behavior of the subgrade beneath the base course of the auxiliary test pavement was made to investigate the effect of the climatic factors—rainfall and temperature—upon subgrade strength (max. $V-L$),[1] density, and moisture content. The initial work was confined for the most part to study of the behavior of the 9-inch compacted layer of soil directly beneath the base course. However, at periodic intervals the moisture gradient was determined for the natural soil to a depth of approximately 36 inches, in order to obtain relative data for comparison of moisture movements at various elevations within the zone that might be influenced by the proposed plate loading of the test pavement.

The variations in strength, density, and moisture content of the 9-inch compacted subgrade layer are shown in figure 5. A comparison of these data show the following trends:

1. The strength (max. $V-L$) varies directly as the density and inversely as the moisture content of the soil.
2. The moisture content gradually increases with time and with the seasonal drop in average daily temperature.
3. Minor variations in strength, density, and moisture content appear to be related to the monthly fluctuations in the climatic factors—temperature and rainfall.

The general relations between these variables were obtained from the averaging of a number of individual test results. Figures 6 and 7 indicate the relations of the average strength (max. $V-L$) to the dry density and to the moisture content of the compacted 9-inch subgrade layer.

[1] The subgrade strength (max. $V-L$) as used in this study refers to the maximum difference between the vertical and lateral pressures as measured by the triaxial compression test.

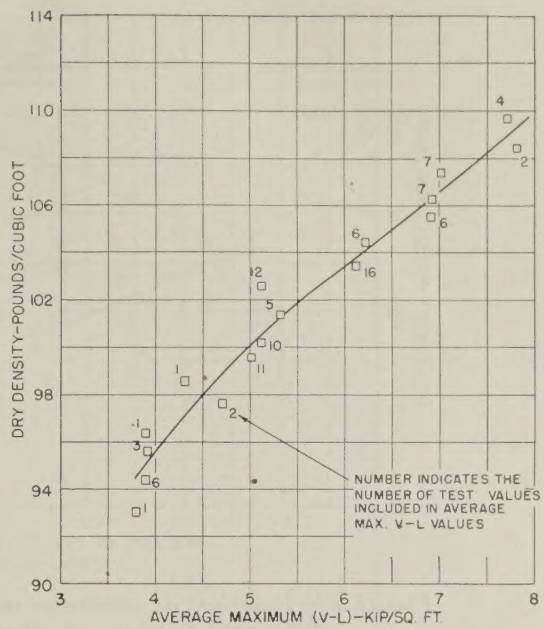


Figure 6.—Relation of maximum V—L (triaxial compression) to density.

The relations of average modulus of elasticity of the subgrade soil, calculated from the triaxial compression test data, to moisture content and to density of the subgrade are shown in figure 8. The modulus varies directly as the density and inversely as the moisture content of the subgrade.

The average seasonal strength (max. $V-L$), density, and moisture content of the soil in the 9-inch compacted layer for several seasons of the year are tabulated in table 5. A study of these data show that the strength of the soil varies with the seasons, being lowest during the winter and highest in the summer. The strength varies inversely and the density directly with the moisture content of the soil. These seasonal variations emphasize the need of planning comparative pavement-evaluation

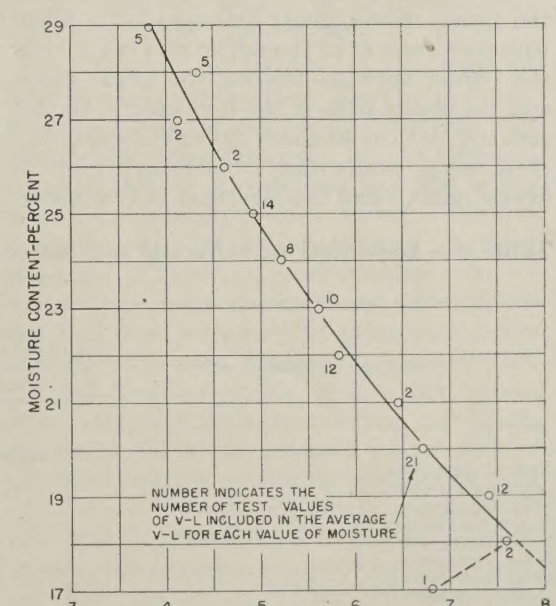


Figure 7.—Relation of maximum V—L (triaxial compression) to moisture content.

Table 5.—*Seasonal variations in subgrade moisture, density, and strength under the 4-inch base of the auxiliary test pavement* [1]

Type of test	Date of tests, 1945–46—				
	July 10–15	Oct. 24–25	Nov. 23–27	Jan. 10–11	Mar. 4
Number of tests averaged	5	3	2	2	4
Average moisture ... percent	20.0	21.7	28.3	29.0	28.5
Average dry density ... lb. per cu. ft.	105.6	103.3	95.0	93.5	95.5
Average maximum $V-L$ ... kips per sq. ft.	7.0	5.8	3.5	3.4	4.7
Average saturation ... percent	89	91	98	96	98

[1] All tests were sampled from an area approximately 55 feet in length.

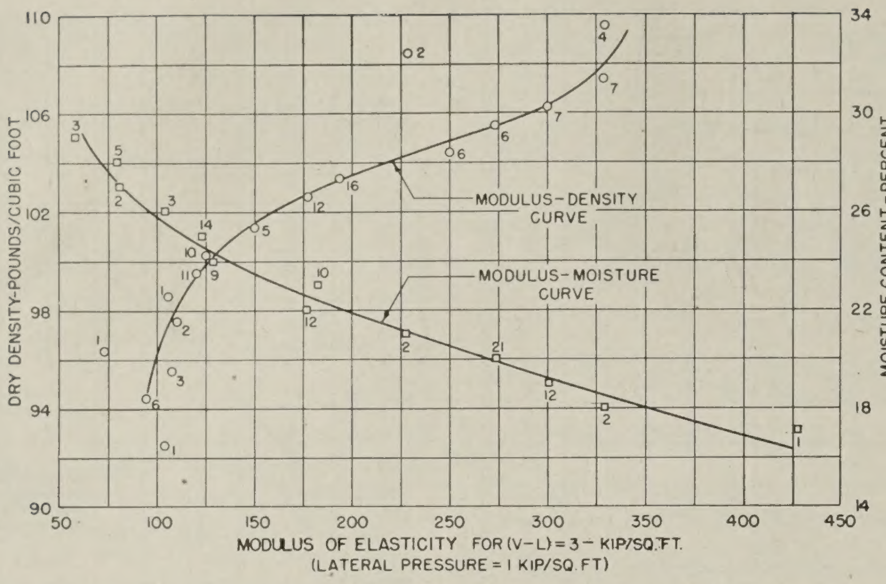


Figure 8.—Relation of modulus of elasticity to moisture content and density of the auxiliary test track subgrade.

tests so that they can be made during periods when the climatic factors are uniform. Adverse changes in climate affect the strength of both the subgrade and the pavement. Both of these strength values must be considered in the evaluation of pavement test data.

The periodic variations in subgrade moisture content at several elevations below the three thicknesses of the base course of the auxiliary test pavement are shown in table 6. A study of these data shows that there appear to be both horizontal and vertical fluctuations in the subgrade moisture content found beneath the base course. Reversals in the moisture contents at various elevations seem to occur for no apparent reason. Perhaps minor variations in texture or structure of the undisturbed soil, in the form of thin lenses of silt and fine sand or seepage planes, may be one of the causes for the erratic fluctuations in the subgrade moisture content beneath the test sections.

It was noticed during field examinations of the general area that in some instances, during the spring season, auger holes gradually filled with free water even though no rain fell during the period between making the auger holes and the observation of the free water. In the area in which the auxiliary pavement was built, there was a definite relation between periods of heavy rainfall and the elevation of free water in the test pits dug in the construction area.

Subgrade conditions similar to those found in the area under the auxiliary test pavement should not occur beneath the 800-foot tangents of the oval test track because the subgrade of the latter is a fill section, composed of soils that were uniformly mixed and compacted to a depth of 5 feet to minimize moisture variations resulting from the change in texture or structure.

A 16-point automatic temperature recorder will be installed in the subgrade, base course, and bituminous pavement of the oval test track to measure the variations in temperature at various elevations below the surface of the pavement. These continuous records of temperature will be used to study the insulation effect of the various thicknesses of pavement as well as to furnish temperature data for use in the correlation of the pavement evaluation work.

LOAD-BEARING SURFACES

In connection with the load testing of nonrigid pavements it was recognized that the use of rigid plates would cause stresses in the pavement structure radically different in character from those produced by pneumatic tires. The reason for this is that the material beneath a rigid plate must displace equally at all points even though the resistance offered to displacement is not uniform. In a cohesive material, such as a bituminous pavement, the resistance to displacement will be greater at the boundaries of the load-bearing surfaces and less in the central portion. Consequently the contact pressure that will develop beneath the peripheral area of a rigid plate as it is forced into a bituminous pavement may exceed greatly that beneath the central portion of the plate. Under a pneumatic tire the situation is quite different. Here, because the tire can undergo a change in shape without a significant change in its pressure-distribution pattern, the material being loaded will tend to deform in a natural manner.

The use of tires themselves as static load-bearing surfaces was not considered feasible (1) because the contact area does not remain constant but increases with load, (2) because of the physical difficulty of measuring the deflection of the material being tested, and (3) because tests are to be made on the base-course and subgrade components of the pavement, and the opening to accommodate the tire would have to be larger and of a more irregular shape than desired.

Thus, one of the important instrumental problems was the development of a type of load-bearing surface that would simulate the action of a tire and yet have none of its undesirable physical characteristics.

In the development work on this problem it was necessary to study the pressure distribution characteristics of a variety of materials. A special pressure-indicating apparatus was constructed with which the contact pressure at any point beneath a particular loaded material could be determined.

The fact that sponge-rubber mats had been used to reduce high peripheral pressures beneath rigid plates in concrete-pavement loading tests prompted a series of initial studies with this material. The effect of the type and thickness of the rubber cushion, both when confined and when nonconfined laterally, was studied. It was found that in either case the pattern of pressure distribution was extremely irregular. When unconfined, the rubber cushion displaced laterally under load, thus reducing the normal intensity of the pressure beneath the peripheral area and increasing the pressure beneath the central area of contact. When confined, an edge or perimeter effect was created resulting in a high concentration of pressure at the outer limits of the area of contact.

Table 6.—*Variations in subgrade soil moisture content (in percent) at several depths below the surface of the subgrade of the auxiliary test pavement, at various times of the year*

Location of sampling area	Depth below bottom of base course	Moisture content for date sampled (1945–46)										Moisture range			Average moisture content
		Aug. 30	Sept. 27	Oct. 16	Oct. 31	Jan. 11	Feb. 20	Mar. 5	Mar. 20	Apr. 2	Apr. 16	Minimum	Maximum	Difference	
	Inches														
Natural subgrade [1]	3	17.7	17.9	17.7	18.0	22.5	20.5	26.9	22.0	22.2	20.7	17.7	26.9	9.2	20.6
4-inch base section	1	20.9	19.7	20.6	21.3	--------	22.5	--------	22.3	21.7	23.0	19.7	23.0	3.3	21.5
8-inch base section	1	27.1	26.6	27.8	26.8	--------	26.6	--------	28.8	27.0	24.8	24.8	28.8	4.0	26.9
12-inch base section	1	23.2	24.2	24.5	22.7	--------	24.6	--------	24.3	24.7	20.1	20.1	24.7	4.6	23.5
Natural subgrade [1]	14	25.8	35.1	35.3	38.5	35.3	--------	22.1	21.9	24.3	24.8	21.9	38.5	16.6	30.4
4-inch base section	10	23.1	23.5	25.0	22.9	--------	23.3	28.2	29.2	24.3	--------	22.9	29.2	6.3	24.9
8-inch base section	10	19.4	25.0	22.6	21.7	--------	25.6	25.2	24.6	25.9	22.7	19.4	25.9	6.5	23.6
12-inch base section	10	23.9	24.4	24.7	25.9	--------	22.9	20.6	26.3	28.5	18.2	18.2	26.3	8.1	23.9
Natural subgrade [1]	26	31.9	31.7	30.1	31.0	35.7	36.6	33.5	36.0	30.7	32.3	30.1	35.7	5.6	33.0
4-inch base section	22	26.8	26.3	25.0	24.7	25.6	24.0	22.2	27.3	--------	26.4	22.2	27.3	5.1	25.4
8-inch base section	22	22.8	24.0	24.4	21.8	25.6	25.4	23.8	23.9	27.2	24.1	21.8	27.2	5.4	24.4
12-inch base section	22	23.5	26.2	24.6	24.5	--------	29.2	25.7	27.0	25.0	23.8	23.5	29.2	5.7	26.6

[1] Natural subgrade refers to an undisturbed subgrade location adjacent to the auxiliary test pavement.

Considerable time was spent in a study of the pressure distribution characteristics of a specially built rubber bag. This was composed of gum rubber molded in a cylindrical shape, and had a vertical hole through its center through which deflection measurements of the material under test could be made. This rubber bag was tested in a number of ways, with and without lateral confinement, both when inflated with air and when filled with water. The distribution of pressure beneath the bag was erratic. When confined laterally and overinflated with either air or water, the pressure was concentrated largely over an annular section between the perimeter and center opening of the bag. When confined laterally and underinflated, a high concentration of pressure developed beneath the side wall and around the center opening. When unconfined, the bag compressed vertically under load to the extent of creating abnormal pressure beneath the side walls and around the center opening. There appeared to be no way in which the bag could be used so that the carcass or vertical walls of the unit did not adversely influence the pattern of pressure distribution.

Many other types of material were studied from the standpoint of their potential ability to transmit load and give the desired pattern of pressure distribution. Among other things, the pressure-transmitting characteristics of thin rubber membranes were investigated in considerable detail. The fact that the pressure transmitted by such a material was invariably found to be equal to the pressure imposed upon it led to the decision that the ultimate design of the bearing surface should embody the use of such an elastic membrane as the primary pressure-transmitting medium. The problem resolved itself into one of determining how to attach the membrane to a chamber or cell so that when air under pressure is introduced (1) the membrane would not blow out, (2) the contact area would not vary to any appreciable extent with load, and (3) the intensity of the transmitted pressure over the entire facial area of the bearing surface would be reasonably uniform.

Three types of bearing cells employing an elastic membrane were developed. They are shown schematically in figure 9. The cell on the left is simply an air chamber with intact side walls. The cell in the center has two cylindrical sections, the lower telescoping into the upper. The cell on the right is similar to that on the left except that a section of flexible metal bellows is installed in the cylindrical section. The performance characteristics of all three cells were studied both with the elastic diaphragm attached to the inside and to the outside of their respective base sections.

From a standpoint of the forces of action and reaction all three cells are basically the same when the diaphragm is on the inside. A given internal air pressure will produce an upward reaction, R, equal to the area of the diaphragm times the prevailing pressure: $R=A\times P$. To develop the same unit intensity of pressure beneath the rim or end projection of the cylinder as that acting through the diaphragm, a supplemental load, L, can be applied by external means to the top of the cell, of a magnitude such that L equals rim area times inflation pressure.

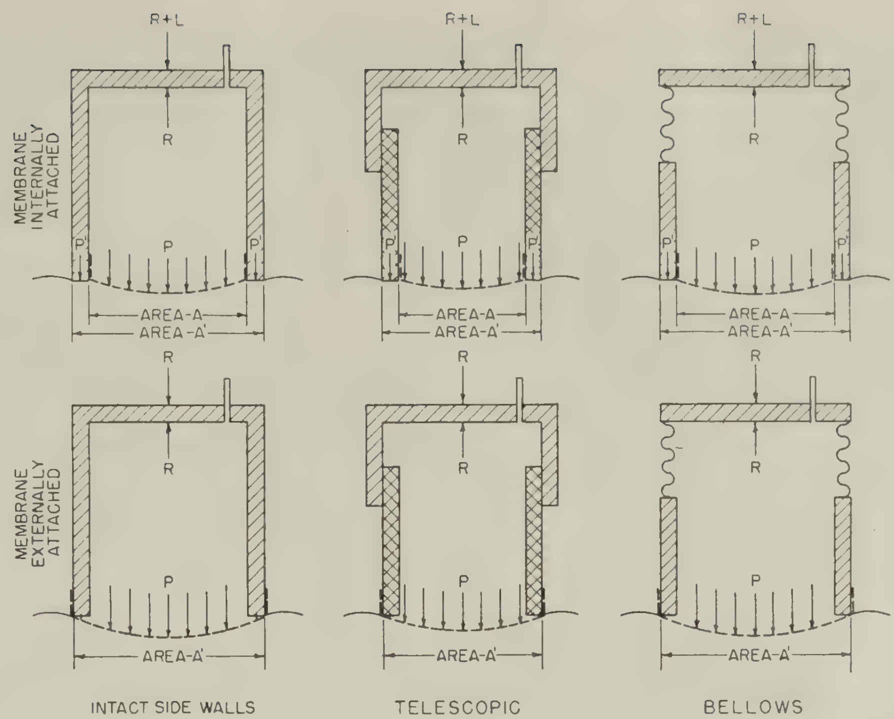


Figure 9.—Schematic cross sections of the flexible-surface load-bearing cells.

Thus it appears that any one of the three cells could be used in such a way that the contact pressure would be uniform over its entire facial area—that is, beneath both the diaphragm and the end projection of the rim section. In this case the rim section would be a part of the bearing surface and would constitute its peripheral area. However, in tests with the cell having intact side walls, extreme difficulty was encountered in manually controlling its operation so that load L at all times was in balance with load R. There appeared to be no way, mechanically or automatically, of maintaining the desired balance between the two components of the load as the internal air pressure is increased or decreased during the progress of a test.

The telescopic cell was developed in an attempt to overcome the operational difficulties of the cell with intact side walls. It was reasoned that in a cell of telescopic design the internal air pressure would bear against the upper projection of the lower section and that this pressure, less the effects of friction that may be present or may develop as the sections move with respect to one another, would be transmitted downward to the material under test. In other words, the pressure transmitted automatically by the end projection of the rim section would be approximately equal to that of the internally existing air pressure. In order to prevent undue leakage of air from the space between the sections, however, it was necessary to use a leather gasket seal that had a relatively high coefficient of friction. This seal served to reduce the pressure transmitted by the rim about 30 percent when the sections were in the process of opening, and to increase this pressure about the same amount when the sections were closing.

It is possible that a bearing surface of the telescopic type, having the diaphragm attached to the inside of the base section, could be used in a manner that would tend to promote continuous and progressive opening of the movable sections during the course of a test. This would result in a reasonably constant deficiency of pressure beneath the rim section, and such a situation might not be considered objectionable if the ratio of the rim area to the diaphragm area is low. However, if it is considered essential to have the same intensity of pressure transmitted through the rim section as that acting through the diaphragm, the total reaction load must be controlled or made to equal the total facial area of the cell times the internally existing air pressure. In contrast to a cell having intact side walls, this can be accomplished without serious danger of overloading or underloading the rim section.

The bearing cell shown at the right in figure 9 was conceived in an effort to overcome the undesirable features of the intact side wall cell and the telescopic cell. It was reasoned that the bellows convolutions installed in the cylindrical section would allow a certain measure of vertical movement without altering the distribution of pressure over its contact face. A considerable amount of work has been done with a cell of this type. While the bellows section, designed to withstand an internal pressure of 100 pounds per square inch, exhibited a higher spring rate than desired, it did serve to impart a much greater degree of vertical flexibility to the cell than when the side walls were intact. Because of this vertical flexibility, the load may be applied and released more uniformly and with less likelihood of rupturing the membrane than was experienced with the rigid-walled cell.

The foregoing discussion described the three flexible bearing surface cells with the dia-

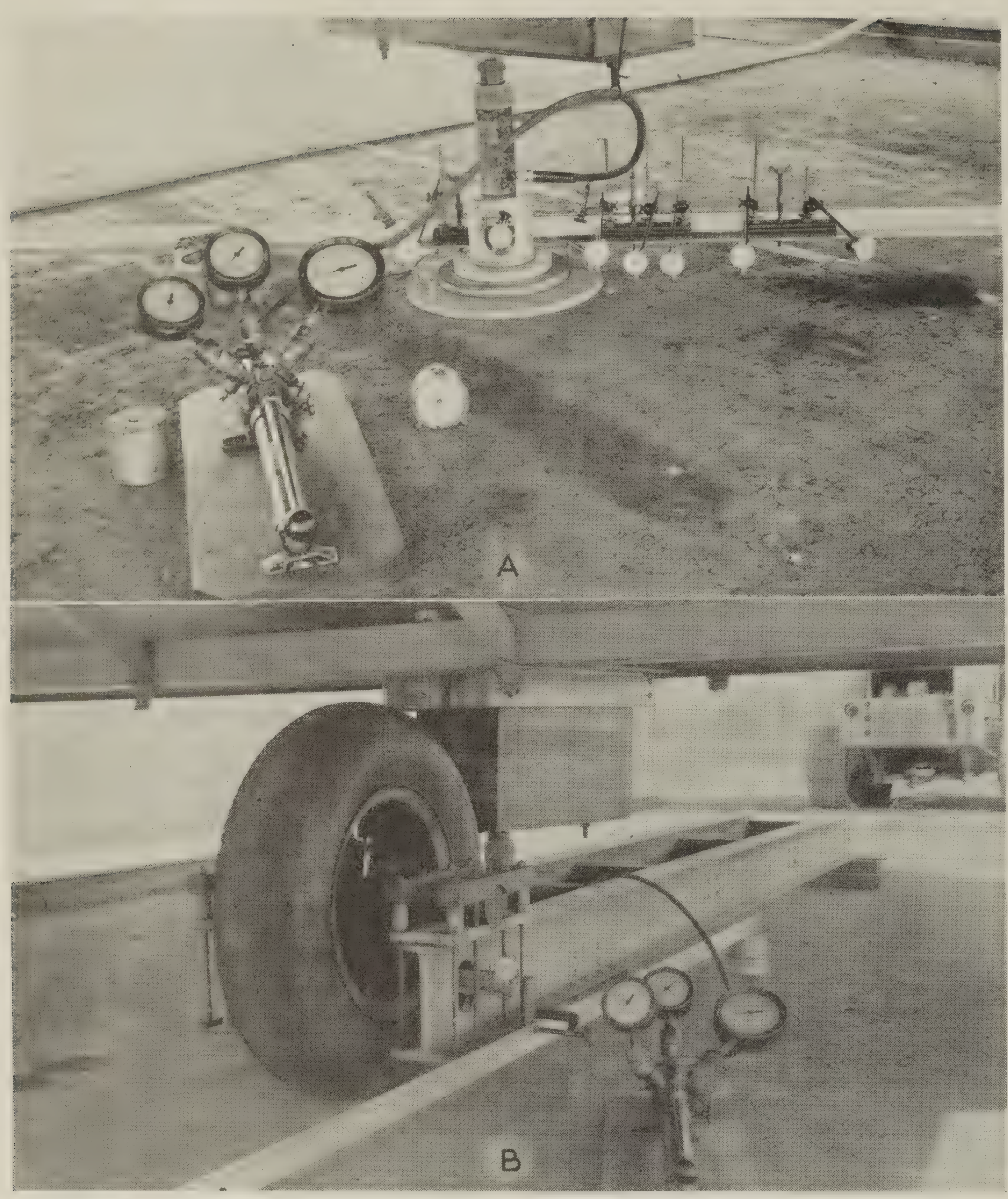


Figure 10.—Load-test assemblies using (A) *rigid plates and* (B) *a pneumatic tire.*

phragm attached to the inside of the lower cylindrical section. In some of the preliminary tests with these cells it was found that the peripheral rim section left what appeared to be an abnormal cut or sharp impression in the material tested. This was in marked contrast to the impression left by a loaded tire on the same material. Here the impression around the edge of the imprint area was relatively smooth.

As a result of these observations the possibility of attaching the diaphragm to the outside rather than to the inside of the unit was considered. It was realized that in this event the contact face of the unit would vary depending upon the magnitude of the applied load, the carcass stiffness of the diaphragm, and the amount of clearance between the end projection of the rim and the diaphragm or material on which it rests. As a result it is not possible to formulate the forces of action and reaction to the same degree of accuracy as when the diaphragm is attached to the inside. It has been found, however, by controlling the amount of clearance between the rim and diaphragm, that the variations in size of contact area are of negligible proportions. In tests with this type of diaphragm attachment the tendency toward the development of a shearing or cutting action at the perimeter of the contact area is greatly reduced.

LOAD-TESTING PROCEDURES

One of the purposes of the auxiliary test pavement was to serve in the development of a procedure for conducting the load-bearing tests on the oval track pavement—a procedure which could be relied upon to give the best possible results. Methods for making such tests have never been standardized, particularly those having to do with the load-supporting capacity of pavements.

Three procedures of load testing using rigid plates were studied. The first involved the application of five or more load increments, each increment being applied and released five times. Each increment produced a net deflection of the medium under test of 0.1 inch—the five increments thus producing a gross deflection of 0.5 inch. In tests using this procedure the applied load was not released nor was the released load reapplied until the rate of vertical movement had slowed down to 0.002 inch per minute. Tests were made according to this procedure on the pavement surface, base course, and subgrade of the three sections of the auxiliary pavement (see fig. 3) using plates of 9- and 18-inch diameter. Duplicate tests were made in all cases, and where the results of the two tests were not in reasonably good agreement a third test was run.

The second procedure differed from the first in that the load was applied rapidly and without interruption until visible rupture of the material occurred or until its resistance was overcome. The application of the load was such as to produce a reasonably constant rate of deflection of 1.2 inch per minute. Tests were conducted according to this procedure on the pavement surface, base course, and subgrade of the auxiliary pavement sections with four rigid plates of 6-, 9-, 12-, and 18-inch diameter. In addition a special series of tests was made, using this procedure, in an effort to develop some preliminary information relative to the effect of the temperature of the bituminous surface upon the load-supporting capacity of the pavement structure. Tests were run with the same group of rigid plates upon the 2-, 4-, and 6-inch thicknesses of bituminous surface over the 4-inch base section of pavement when the temperature of the surface was about 68°, 85°, and 105° F.

The third procedure was that of repetitional loading in which from 15 to 100 load applications were applied and released. Tests were made on the top of the base of all three auxiliary pavement sections with the 9- and 18-inch diameter plates. The series of tests was planned specifically for the purpose of studying the elastic action of the base course-subgrade component of the pavement.

In addition to the various tests described above a great number of tests were conducted upon the sections of the auxiliary pavement in connection with the problem of the development of a flexible type of bearing surface. The several load-test assemblies are shown in figures 10 and 11. A view of a rigid plate assembly appears in figure 10*A*. The plates are arranged in typical pyramid fashion to insure rigidity. Vertical movement readings are made with dial gages bearing against the center and edges of the base plate and against the surface of the pavement at distances of 3, 6, 12, and 24 inches from the plate. Load is applied by means of a retractable hydraulic jack. In this as well as in all the other load assemblies, an electric buzzer is clamped to the dial-supporting beam for the purpose of freeing the dial stems when readings are made. The effect is similar to that of tapping a gage to make sure that the needle is not stuck. It has been found that the use of such a buzzer not only serves to facilitate reading the gages but tends to give more accurate results.

Figure 11*A* shows a load assembly employing the telescopic type of bearing surface described previously. The deflection of the pavement surface in this case is measured at the center of the unit on the top of a rod extending downward through the cell to the diaphragm. The deflection of the rim of the cell or the material beneath the end projection of the rim is measured by means of three dial gages operating against brackets attached to the rim. The air pressure in the cell is controlled by means of the pressure regulating valve assembly shown. The hydraulic jack superimposed on the cell is used to determine or control the reaction load. In figure 11*B* another load-test assembly is shown, employing the bellows type of bearing surface. This cell is operated and the deflections measured in much the same manner as described for the telescopic cell.

In figure 10*B* still another load assembly is shown, utilizing a pneumatic tire as the load-bearing surface. Here the vertical movement of the pavement surface beneath the center of the tire contact area is measured by means of a specially designed deflection rod unit that is installed in the tire. Lack of clearance beneath the water tank that was used on the auxiliary test track to provide the necessary load reactions made it necessary to mount the tire on the end of the beam assembly shown and apply the load in the manner indicated.

In load testing the pavements and base courses of the oval test track a 25-ton trailer, loaded with concrete or steel blocks, is being used. The weight is concentrated on the bearing surface by means of a hydraulic jack, as shown in the cover illustration.

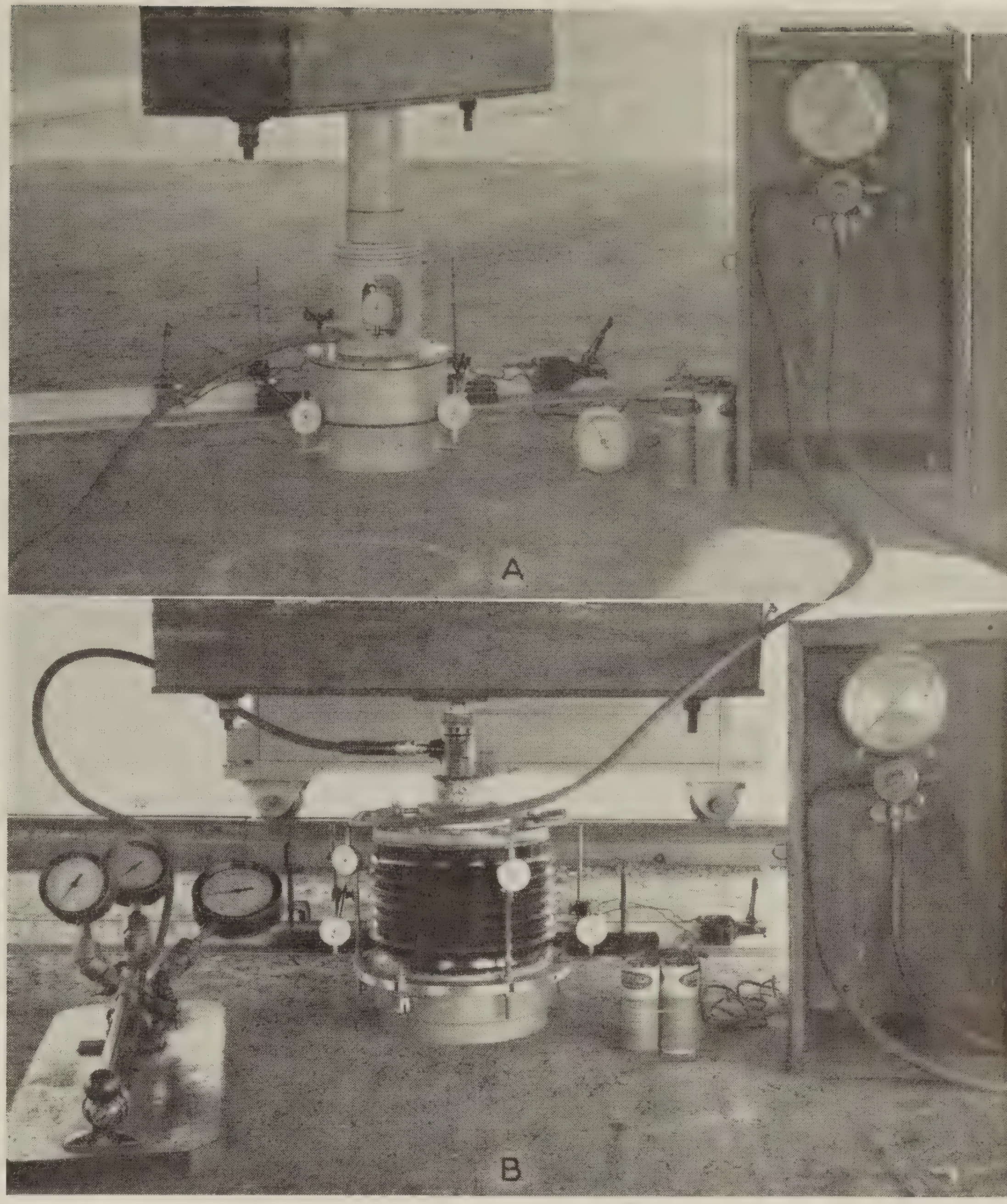


Figure 11.—Load-test assemblies using (A) *the telescopic cell and* (B) *the bellows cell.*

SUMMARY

A great deal of pertinent and useful information has been obtained from the tests made to date on the sections of the auxiliary pavement. This information has been particularly helpful in the formulation of the testing program for the sections of pavement on the oval test track.

The investigation has been planned in such a way that it should be possible to compare and correlate all existing proposed methods of thickness design. Thus in the course of the investigation such tests as the North Dakota cone test, the California bearing ratio test, and the triaxial compression test must be made on certain of the component materials of the pavement at each location where a bearing test is made. In addition, samples of these components will be obtained for moisture content and density determinations.

The completion of the planned program of tests should throw considerable additional light on many questions which are as yet, to a great extent, unanswered.

764708—47——2

Measuring changes in volume and sonic modulus of elasticity of frozen and thawed concrete beams

The Effect on Properties of Concrete of Natural and Portland Cement Blends[1]

BY THE DIVISION OF PHYSICAL RESEARCH PUBLIC ROADS ADMINISTRATION

Reported by A. G. TIMMS, Senior Materials Engineer, W. E. GRIEB, Associate Highway Engineer, and GEORGE WERNER, Associate Materials Engineer

Confirming the results of earlier studies, this investigation of concretes made with blends of natural and portland cements shows that such concretes, because they entrain air, have increased resistance to freezing and thawing. Offset against this gain is a somewhat lessened strength. The investigation found no properties in the natural cements, other than their air-entraining ability, that influenced the increased resistance to freezing and thawing.

In plain concrete slabs as constructed in the field, the top of the slab usually has lower resistance to freezing and thawing than the bottom. This investigation indicates that air entrainment improves the concrete throughout the slab with regard to resistance to freezing and thawing and tends to make it more uniform.

AN INVESTIGATION of the use of blends of natural and portland cement for improving the quality of concrete was reported in 1938 by the Public Roads Administration.[2] That investigation, prompted by the increasing use of blends in the field, was limited to the use of one natural cement in the case of the laboratory-fabricated specimens and to two natural cements in the case of cores drilled from pavements and tested in the laboratory.

The present report summarizes the results of a considerably expanded laboratory investigation begun in January 1942 that involved the use of seven natural cements and one slag cement, each of which was blended with each of three plain or nonair-entraining portland cements. In addition, comparable tests were made with the three plain portland cements as well as with the air-entraining counterparts of two of them. Some of the natural cements contained sufficient air-entraining material to produce air-entraining concrete, while others produced little or no additional air.

For convenience in conducting this investigation the tests were divided into three series, each designed to study certain properties of concrete. Series 1 was planned to study the flexural and compressive age-strength relations; series 2[3] was designed to investigate drying shrinkage and resistance to freezing and thawing in water, and to study the effect of initial curing and an interruption in the freezing and thawing cycle; and series 3 covered an investigation of the effect of segregation of the constituent materials due to manipulation of the surface in the finishing operations in laboratory-mixed concrete placed to simulate pavement construction practice.

CONCLUSIONS

In this investigation the plain portland cements, the air-entraining portland cements, and the natural cement blends entrained air in volumes of from 1.3 to 5.3 percent—all well

[1] Presented at the fiftieth annual meeting of the American Society for Testing Materials, June 17, 1947, at Atlantic City, N. J.

[2] *The Effect of Using a Blend of Portland and Natural Cement on the Physical Properties of Mortar and Concrete*, by W. F. Kellermann and D. G. Runner; PUBLIC ROADS, vol. 19, No. 8, October 1938, p. 153; *also* Proceedings of the American Society for Testing Materials, vol. 38, Part II, 1938, p. 329.

[3] Certain of the freezing and thawing test data from series 2 of this investigation were presented in the paper *Effect of Blended Cements and Vinsol Resin-Treated Cements on Durability of Concrete*, by W. F. Kellermann; Journal of the American Concrete Institute, vol. 17, No. 6, June 1946, p. 681.

within the upper 6 percent limit usually specified in current construction practice where air-entraining concrete is used. The results of tests performed on the concrete specimens confirm the conclusions of previous investigations that increased resistance to freezing and thawing is obtained by the use of air-entraining concrete, though at some sacrifice in strength.

It is concluded from the results of the tests that the increased resistance to freezing and thawing of concrete made with blends of portland and natural cements of the types and in the proportions used in this investigation is due to the fact that the concrete contained entrained air.

Many investigations have shown that the upper portion of a plain concrete slab as cast is less resistant to freezing and thawing than the bottom. These tests indicate that air entrainment will tend to improve the uniformity of the concrete throughout the depth of the slab.

GENERAL TEST PROCEDURE

In construction, the general practice is to use a blend consisting of one bag of natural cement and either five or six bags of portland cement. In this investigation all blends were made in the proportion of 14 percent natural cement to 86 percent portland cement, by weight. This is about the proportion of one bag of natural cement weighing 80 pounds to five bags of portland cement weighing 94 pounds each.

The three portland cements used in the investigation were designated by capital letters *A*, *B*, and *C*. All of them were nonair-entraining and as such were designated as *A1*, *B1*, and *C1*. Flake Vinsol resin interground with cements *A* and *B* produced air-entraining portland cements designated as *A2* and *B2*. There was no air-entraining counterpart of cement *C*.

The natural cements used in the investigation were designated by Roman numerals *I* to *V*, inclusive, and the slag cement by Roman numeral *VI*. Two lots of each of the natural cements *II* and *IV*, differing considerably in the amount of interground air-entraining (chloroform-soluble) material they contained, were tested. The samples with the smaller amounts were designated as *IIa* and *IVa*, while the samples with the larger amounts were designated as *IIb* and *IVb*.

In summary, the notation identifying the several portland cements, natural cements, and the slag cement used in the investigation is as follows:

Portland cements.. capital letters *A*, *B*, *C*
- nonair-entraining.......... suffix *1*
- air-entraining.............. suffix *2*

Natural cements.. Roman numerals *I* to *V*
- with smaller amount of air-entraining material........ suffix *a*
- with larger amount of air-entraining material........ suffix *b*

Slag cement....... Roman numeral *VI*

Results of chemical and physical tests on the cements are given in tables 1 and 2. The results of grading and other physical tests of the aggregates used are shown in table 3.

Table 1.—*Chemical and physical properties of portland cements*

CHEMICAL ANALYSES

	Portland cements				
	A1	*A2*	*B1*	*B2*	*C1*
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Silica (SiO_2)	21.60	21.60	23.90	23.90	21.55
Alumina (Al_2O_3)	6.14	6.29	4.27	4.14	6.02
Ferric oxide (Fe_2O_3)	2.56	2.56	3.28	3.36	3.68
Lime (CaO)	63.90	63.85	64.60	64.65	63.90
Magnesia (MgO)	2.93	2.93	1.09	1.10	1.10
Sulfuric anhydride (SO_3)	1.58	1.61	1.41	1.42	1.58
Sodium and potassium oxide (Na_2O+K_2O)	.68	.54	.64	.61	.68
Loss on ignition	.62	.64	.84	.78	1.42
Chloroform—soluble material [1]	.007	.035	.006	.040	.010

COMPUTED COMPOUND COMPOSITIONS

	A1	*A2*	*B1*	*B2*	*C1*
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Tricalcium silicate (C_3S)	46	45	44	45	46
Dicalcium silicate (C_2S)	27	28	36	35	27
Tricalcium aluminate (C_3A)	12	12	6	5	10
Tetracalcium alumino—ferrite (C_4AF)	8	8	10	10	11
Calcium sulfate ($CaSO_4$)	2.7	2.7	2.4	2.4	2.7

PHYSICAL PROPERTIES

	A1	*A2*	*B1*	*B2*	*C1*
Apparent specific gravity	3.17	3.21	3.17	3.22	3.19
Specific surface (Wagner) ... cm.[2] per gm	1,730	1,670	1,690	1,735	1,910
Autoclave expansion ... percent	.15	.17	.01	.02	.05
Normal consistency ... percent	23.5	23.0	24.5	24.0	25.0
Time of setting (Gillmore test):					
Initial set ... hr.-min	3-10	2-40	3-45	3-30	3-30
Final set ... hr.-min	4-45	4-40	5-00	5-25	5-00
Tensile strength (1:3 mortar):					
3 days ... lb. per sq. in	305	280	285	225	295
7 days ... lb. per sq. in	415	370	325	305	360
28 days ... lb. per sq. in	480	490	465	385	445
Compressive strength (1:3 mortar):					
3 days ... lb. per sq. in	2,015	1,805	1,695	1,420	2,050
7 days ... lb. per sq. in	3,645	3,495	2,870	2,395	3,515
28 days ... lb. per sq. in	5,450	5,020	4,715	4,045	5,615

[1] A. S. T. M. Method C-114-42 used.

All mixes were designed on the basis of use of nonair-entraining portland cements and no adjustments were made in the proportions when using the air-entraining portland cements or the blends containing natural and slag cements. As a result, the proportions of cement to aggregate by weight were the same for all cement combinations. The mixes were also designed to obtain a constant consistency as measured by the slump test. Observations were made of all of the concrete mixes as to workability. This varied considerably. All

Table 2.—*Chemical and physical properties of natural and slag cements*

CHEMICAL ANALYSES

	Natural cements							Slag cement
	I	*IIa*	*IIb*	*III*	IV*a*	*IVb*	*V*	*VI*
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Silica (SiO_2)	24.15	23.60	23.55	18.15	25.25	25.25	23.75	30.50
Alumina (Al_2O_3)	5.28	7.19	7.40	4.57	5.50	5.50	5.30	9.41
Ferric oxide (Fe_2O_3)	4.32	2.56	2.40	2.08	2.40	2.40	2.80	1.44
Lime (CaO)	33.45	33.05	33.00	46.80	34.45	34.30	34.45	51.15
Magnesia (MgO)	21.65	20.77	20.68	10.45	21.62	21.96	21.65	1.83
Sulfuric anhydride (SO_3)	1.30	1.61	1.96	1.78	2.35	2.25	2.30	.45
Sodium and potassium oxide (Na_2O+K_2O)	1.78	1.78	1.83	1.06	3.00	2.93	3.41	--------
Loss on ignition	7.90	9.54	9.29	15.07	5.50	5.35	6.40	3.85
Insoluble residue	14.25	13.85	13.85	6.30	11.15	11.05	11.65	--------
Chloroform—soluble material [1]	.05	.04	.32	[2] .33+	.08	.13	.18	--------

PHYSICAL PROPERTIES

	I	*IIa*	*IIb*	*III*	IV*a*	*IVb*	*V*	*VI*
Apparent specific gravity	3.06	2.90	2.88	2.86	3.03	3.05	3.00	2.80
Specific surface, (Wagner) ... cm.[2] per gm	2,310	2,350	2,445	3,030	2,285	2,035	2,565	2,770
Autoclave expansion ... percent	[3] 3.8	1.8	2.5	4.2	15.0	15.0	[3] 5.5	—.1
Normal consistency ... percent	29.0	31.0	32.0	34.0	36.0	38.0	32.0	32.0
Time of setting (Gillmore test):								
Initial set ... hr.-min	0-45	3-15	1-10	2-15	1-00	0-55	1-10	2-35
Final set ... hr.-min	1-15	5-30	2-35	5-15	1-45	1-45	1-50	5-00
Tensile strength (1:2 mortar):								
3 days ... lb. per sq. in	40	35	65	80	[4] 40	[4] 65	75	[4] 60
7 days ... lb. per sq. in	50	65	115	125	[4] 90	[4] 90	125	[4] 215
28 days ... lb. per sq. in	70	170	235	270	[4] 210	[4] 200	195	[4] 380
Compressive strength (1:2 mortar):								
3 days ... lb. per sq. in	135	175	360	510	[4] 185	[4] 215	540	[4] 275
7 days ... lb. per sq. in	150	365	655	765	[4] 305	[4] 285	700	[4] 1,060
28 days ... lb. per sq. in	430	1,160	1,605	1,790	[4] 855	[4] 710	1,150	[4] 2,490

[1] A. S. T. M. Method C-114-42 used.
[2] All chloroform—soluble material not extracted.
[3] Specimen warped.
[4] 1:3 mortar used.

Table 3.—*Grading and other physical properties of aggregates*

GRADING

	Potomac River sand used in series 1 and 2	Maryland bank sand used in series 3	Potomac River gravel used in series 1 and 3	Potomac River gravel used in series 2
Percent passing sieve size:				
1½-inch	100	100	100	100
1-inch	100	100	68	100
¾-inch	100	100	43	80
½-inch	100	100	29	50
⅜-inch	100	100	15	30
No. 4	98	94	3	0
No. 8	87	79		
No. 16	76	66		
No. 30	57	44		
No. 50	23	14		
No. 100	5	2		

OTHER PHYSICAL PROPERTIES

Fineness modulus	2.54	3.01	7.39	6.90
Bulk specific gravity (dry)	2.59	2.57	2.59	2.59
Absorption (24 hour) percent	1.2	1.2	1.1	1.1

Table 5.—*Compressive strength of cylinders,*[1] *series 1 tests*

Cements	Compressive strength [2] at— 3 days		7 days		28 days		180 days		360 days		4 years	
	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio
	P. s. i.	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*
A1	2,130	100	4,000	100	5,890	100	6,070	100	7,090	100	7,220	100
A2	1,850	87	3,950	99	5,600	95	6,430	106	6,990	99	7,390	102
A1+I	2,170	102	3,590	90	4,970	84	5,700	94	6,430	91	6,690	93
A1+IIa	2,280	107	3,770	94	5,740	98	6,090	100	7,150	101	7,740	107
A1+IIb	1,690	79	2,840	71	4,290	73	4,670	77	5,490	77	5,510	76
A1+III	2,200	103	3,560	89	5,280	90	6,170	102	6,750	95	7,100	98
A1+IVa	1,830	86	3,190	80	4,750	81	5,490	90	5,980	84	6,550	91
A1+IVb	1,530	72	2,680	67	3,950	67	4,620	76	5,220	74	5,540	77
A1+V	2,070	97	3,330	83	4,970	84	5,580	92	6,250	88	6,630	92
A1+VI	2,000	94	3,720	93	6,200	105			7,570	107		
B1	1,960	100	3,430	100	5,370	100	7,320	100	8,380	100	8,190	100
B2	1,690	86	3,120	91	4,560	85	6,950	95	7,030	84	7,250	89
B1+I	1,900	97	3,270	95	4,850	90	6,500	89	6,980	83	7,910	97
B1+IIa	1,910	97	3,410	99	5,160	96	6,720	92	7,370	88	8,150	100
B1+IIb	1,550	79	2,640	77	3,520	66	5,080	69	5,890	70	6,480	79
B1+III	1,860	95	3,100	90	4,530	84	6,160	84	7,180	86	7,590	93
B1+IVa	1,720	88	2,860	83	4,350	81	5,990	82	6,490	77	7,390	90
B1+IVb	1,330	68	2,360	69	3,610	67	5,110	70	5,630	67	6,310	77
B1+V	1,810	92	2,860	83	4,460	83	6,170	84	6,770	81	7,600	93
B1+VI	1,740	89	3,160	92	5,230	97			7,690	92		
C1	2,410	100	3,860	100	5,880	100	6,870	100	7,510	100	7,570	100
C1+I	2,180	90	3,740	97	5,270	90	6,120	89	7,470	99	7,330	97
C1+IIa	2,230	93	3,540	92	5,260	90	6,570	96	6,760	90	8,150	108
C1+IIb	1,920	80	3,020	78	4,520	77	5,170	75	5,870	78	6,360	84
C1+III	2,120	88	3,600	93	5,240	89	6,300	92	7,130	95	7,790	103
C1+IVa	2,050	85	3,170	82	4,940	84	5,800	84	6,100	81	6,850	90
C1+IVb	1,700	71	2,850	74	4,310	73	5,140	75	5,670	76	6,220	82
C1+V	2,180	90	3,380	88	4,930	84	6,150	90	6,680	89	7,570	100
C1+VI	2,280	95	3,680	95	5,760	98			8,050	107		

[1] Specimens used were 6- by 12-inch cylinders.
[2] The compressive strength values are based on the average of 3 tests in each case. The ratio of strength is based on the strength of plain portland cements *A1*, *B1*, and *C1*, respectively.

of the air-entraining mixtures produced more workable concrete than those that contained only small quantities of air.

The amount of water per unit volume of concrete was calculated from actual yield tests. Differences in water contents reflect differences in the volume of air in the mixes as well as slightly different water requirements of the various cements.

STRENGTH TESTS

The tests comprising series 1 of the investigation were planned to study the flexural and compressive age-strength relation. Mix data for the concretes used in this series are given in table 4. All of the portland cements and blends of the nonair-entraining portland cements with each of the natural and slag cements were used. The mix contained six bags of cement per cubic yard. Potomac River gravel (1½-inch to No. 4) and sand were used as aggregates. Their grading and other physical properties are shown in table 3.

Specimens used in the compression tests were 6- by 12-inch cylinders, and those used in the flexure tests were 6- by 6- by 20-inch beams. The specimens were moist cured and were tested at ages ranging from 3 days to 4 years.

Table 4 shows the calculated air contents for the different concretes. These varied from 1.2 percent for concrete containing nonair-entraining cement *B1* to 5.3 percent for concrete containing blend *A1+IIb*. All of the blends of natural cements and the slag cement produced air contents in excess of 2 percent except natural cement *IIa* and slag cement *VI*. The air-entraining portland cements developed about 3 percent air.

Results of compressive and flexural strength tests for specimens ranging in age from 3 days to 4 years are given in tables 5 and 6, respectively, and are shown graphically in figures 1 to 4, inclusive.

Figures 1 and 2 show the relation between air content and compressive and flexural strength at 28 days. The data at this age were selected for plotting as being typical of the data obtained at all ages. The test data indicate that, for air contents in excess of 2 percent, the compressive strength was reduced approximately in proportion to the increase in the amount of entrained air.

From table 5 it will be noted that for the concrete with the two highest air contents, *A1+IIb* and *A1+IVb*, the compressive strength at 28 days was reduced approximately 30 percent as compared to that obtained with plain portland cement concrete. The corresponding decrease at 4 years was about 24 percent.

In figures 3 and 4 are plotted the age-strength relations for all cement combinations. In each case the data have been plotted in three groups. The left-hand diagrams show the age-strength relations for

Table 4.—*Mix*[1] *data, series 1 tests*

Cements [2]	Water content [3]	W_c[4]	Slump	Flow	Weight of fresh concrete	Actual cement content [3]	Calculated air content
	Gal. per bag		*Inches*	*Percent*	*Lb. per cu. ft.*	*Bags per cu. yd.*	*Percent*
A1	5.2	0.152	2.6	55	149.6	6.0	1.3
A2	5.0	.146	2.8	50	148.1	5.9	2.8
A1+I	5.2	.150	2.9	55	148.0	5.9	2.3
A1+IIa	5.2	.152	2.6	52	149.2	5.9	1.5
A1+IIb	5.1	.144	3.1	58	143.6	5.7	5.3
A1+III	5.2	.149	3.0	54	146.8	5.8	3.0
A1+IVa	5.2	.151	2.8	60	146.8	5.8	3.0
A1+IVb	5.2	.146	2.7	59	143.8	5.7	5.1
A1+V	5.2	.149	2.9	59	146.5	5.8	3.3
A1+VI	5.2	.152	3.0	59	149.3	6.0	1.3
B1	5.1	.150	2.8	52	150.1	6.0	1.2
B2	4.9	.141	3.0	50	148.0	5.9	3.1
B1+I	5.1	.149	2.8	51	148.6	5.9	2.1
B1+IIa	5.1	.150	2.4	46	149.7	6.0	1.3
B1+IIb	5.0	.143	2.8	55	144.8	5.8	4.6
B1+III	5.1	.147	2.7	54	147.4	5.9	2.8
B1+IVa	5.2	.150	2.8	52	147.3	5.9	2.8
B1+IVb	5.2	.147	2.8	57	144.5	5.8	4.6
B1+V	5.1	.147	2.7	53	147.0	5.9	3.1
B1+VI	5.1	.150	2.9	53	149.5	6.0	1.4
C1	5.2	.152	3.0	49	149.4	6.0	1.5
C1+I	5.2	.150	2.9	51	148.1	5.9	2.3
C1+IIa	5.2	.151	2.9	46	149.0	5.9	1.6
C1+IIb	5.1	.144	2.8	48	144.4	5.8	4.8
C1+III	5.2	.149	3.1	51	147.2	5.9	2.8
C1+IVa	5.2	.151	3.1	53	146.8	5.8	3.1
C1+IVb	5.2	.149	2.8	54	144.2	5.7	4.8
C1+V	5.2	.149	2.9	51	146.7	5.8	3.2
C1+VI	5.2	.151	2.7	47	148.9	5.9	1.7

[1] Mix by dry weight, in pounds=94:166:369, using 1½-inch to No. 4 gravel.
[2] Where natural or slag cement was used, proportions were 86 percent portland cement to 14 percent natural or slag cement, by weight.
[3] Where natural or slag cement was used, 94 pounds was considered as one bag.
[4] Water per unit volume of concrete, based on actual yield tests.

Table 6.—*Flexural strength of beams,[1] series 1 tests*

Cements	Modulus of rupture [2] at—											
	3 days		7 days		28 days		180 days		360 days		4 years	
	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio	Strength	Ratio
	P. s. i.	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*	*P. s. i.*	*Percent*
A1	320	100	460	100	605	100	635	100	660	100	660	100
A2	310	97	470	102	615	102	555	87	600	91	640	97
A1+I	305	95	455	99	535	88	555	87	555	84	605	92
A1+IIa	310	97	430	93	520	86	585	92	620	94	690	105
A1+IIb	275	86	400	87	480	79	490	77	515	78	550	83
A1+III	320	100	430	93	515	85	585	92	625	95	650	98
A1+IVa	320	100	430	93	520	86	505	80	505	77	595	90
A1+IVb	270	84	385	84	460	76	495	78	495	75	570	86
A1+V	305	95	420	91	520	86	540	85	600	91	590	89
A1+VI	330	103	460	100	595	98	----------	------	670	102	----------	------
B1	320	100	430	100	560	100	665	100	745	100	735	100
B2	280	88	430	100	550	98	650	98	690	93	625	85
B1+I	305	95	415	97	540	96	625	94	640	86	695	95
B1+IIa	300	94	425	99	545	97	650	98	710	95	710	97
B1+IIb	255	80	390	91	485	87	525	79	640	86	650	88
B1+III	285	89	400	93	550	98	635	95	670	90	725	99
B1+IVa	275	86	370	86	490	87	550	83	660	89	660	90
B1+IVb	225	70	340	79	475	85	510	77	575	77	625	85
B1+V	280	88	385	90	545	97	565	85	635	85	695	95
B1+VI	260	81	425	99	565	101	----------	------	705	95	----------	------
C1	380	100	480	100	575	100	640	100	675	100	695	100
C1+I	350	92	480	100	555	97	590	92	640	95	680	98
C1+IIa	340	89	480	100	565	98	580	91	690	102	730	105
C1+IIb	320	84	430	90	510	89	520	81	570	84	625	90
C1+III	350	92	445	93	580	101	605	95	670	99	705	101
C1+IVa	345	91	435	91	575	100	525	82	680	101	650	94
C1+IVb	295	79	380	79	510	89	495	77	550	81	615	88
C1+V	340	89	410	85	520	90	570	89	560	83	680	98
C1+VI	320	84	445	93	585	102	----------	------	650	96	----------	------

[1] Specimens used were 6- by 6- by 20-inch beams, tested with third-point loading on an 18-inch span; side as molded in tension (A. S. T. M. designation C-78-44).
[2] The modulus of rupture strength values are based on the average of 3 tests in each case. The ratio of strength is based on the strength of plain portland cements *A1*, *B1*, and *C1*, respectively.

Table 7.—*Mix [1] data, series 2 tests*

Cements [2]	Water content [3]	Slump
	Gal. per bag	*Inches*
A1	6.1	3.3
A2	5.7	3.1
A1+I	6.1	3.8
A1+IIa	6.1	3.4
A1+IIb	5.8	3.9
A1+III	6.0	3.8
A1+V	6.0	4.0
A1+VI	6.0	3.2
B1	5.9	3.5
B2	5.6	3.8
B1+I	6.0	3.3
B1+IIa	6.0	3.1
B1+IIb	5.7	3.1
B1+III	5.8	3.3
B1+V	5.9	3.6
B1+VI	5.9	3.0

[1] Mix by dry weight, in pounds=94 : 194 : 339, using 1-inch to No. 4 gravel. Nominal cement factor=5.8 bags per cubic yard.
[2] Where natural or slag cement was used, proportions were 86 percent portland cement to 14 percent natural or slag cement, by weight.
[3] Where natural or slag cement was used, 94 pounds was considered as 1 bag.

the plain cement concretes, for the air-entraining portland cements (*A2* and *B2* only), and for the slag cement blends. The center diagrams show the corresponding values for concretes containing natural cements *II* and *IV;* that is, the two natural cements that had different amounts of interground air-entraining material. The right-hand diagrams give corresponding data for the other three natural cements.

An inspection of figure 3 reveals that blends of all three plain portland cements with natural cements *IIb* and *IVb* showed considerable reduction in strength compared to the plain portland cement. With the exception of the slag cement *VI*, the other blends gave strengths intermediate between the plain portland and the two high air-entraining natural cements *IIb* and *IVb*. For several ages the blend *A1+IIa* gave higher compressive strengths than the concrete made from plain portland cement.

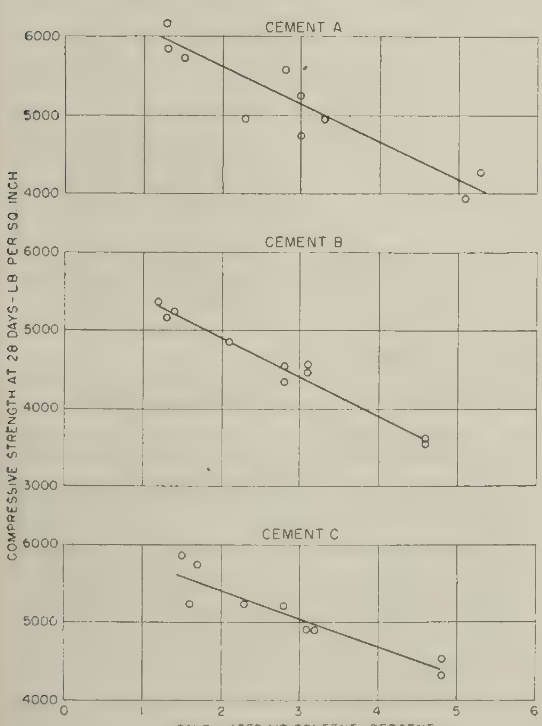


Figure 1.—*Relation of compressive strength at 28 days to calculated air content.*

As will be seen in table 5, the percentage reduction in strength due to air entrainment was less, in all cases, at the 4-year period than at the 28-day period.

The combinations of slag cement *VI* with each of the three portland cements showed approximately as high or higher strengths at 1 year as the portland cements with which they were blended. In general, the air-entraining portland cements showed slightly lower strengths than the companion plain portland cements. The reduction in strength is about what would be expected [4] based on the amount of calculated entrained air.

Blends of low air-entraining natural cements and portland cements gave markedly higher strengths than blends of high air-entraining natural cements and portland cements at all ages of test. This is plainly seen in the center diagram of figure 3.

These general relations hold for all three of the portland cements used. The same trends brought out for compressive strength are characteristic of flexure except that they are not so pronounced: See table 6 and figure 4.

[4] *Tests of Concrete Containing Air-Entraining Portland Cements or Air-Entraining Materials Added to Batch at Mixer*, by H. F. Gonnerman; Journal of the American Concrete Institute, vol. 15, No. 6, June 1944, p. 477.

DRYING SHRINKAGE AND FREEZING AND THAWING TESTS

The tests comprising series 2 of this investigation were designed to study drying shrinkage and resistance to alternate freezing and thawing in water, and to study the effect of initial curing and an interruption in the freezing and thawing cycle. Two of the nonair-entraining portland cements, the two air-entraining portland cements, and blends of the two nonair-entraining portland cements with five of the natural cements and the slag cement were used. The nominal cement factor was 5.8 bags per cubic yard. Mix data for the concretes are given in table 7. Potomac River sand and gravel, of grading and other physical properties shown in table 3, were used as aggregates. The maximum size of the aggregates was limited to 1 inch because of the small size of the test specimens, which were 3- by 4- by 16-inch beams.

For each variable, seven 3- by 4- by 16-inch beams were made for test—three for

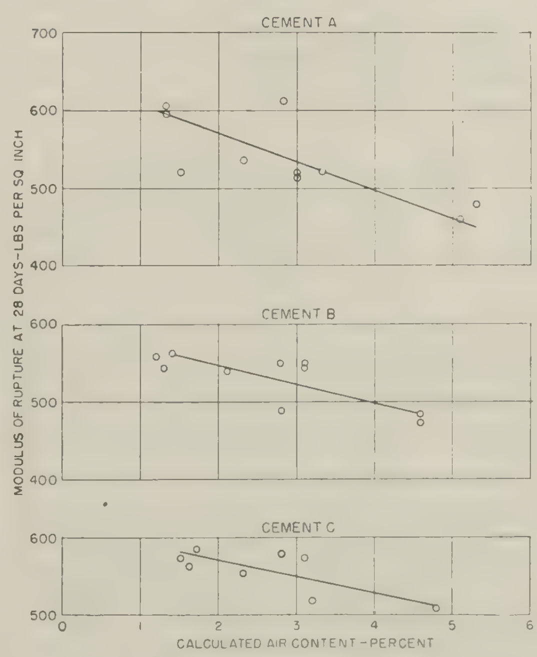


Figure 2.—*Relation of flexural strength at 28 days to calculated air content.*

Table 8.—*Drying shrinkage, series 2 tests*

Cements	Drying shrinkage after storage [1] for—			
	50 days	100 days	150 days.	200 days
	Percent	*Percent*	*Percent*	*Percent*
A1	0.045	0.054	0.058	0.060
A2	.048	.058	.061	.062
A1+I	.049	.059	.064	.067
A1+IIa	.051	.062	.066	.068
A1+IIb	.058	.069	.074	.076
A1+III	.049	.059	.064	.066
A1+V	.049	.058	.062	.063
A1+VI	.045	.054	.058	.060
B1	.040	.049	.052	.053
B2	.042	.050	.054	.055
B1+I	.045	.054	.059	.061
B1+IIa	.045	.054	.058	.059
B1+IIb	.046	.055	.059	.061
B1+III	.043	.051	.052	.053
B1+V	.048	.056	.060	.061
B1+VI	.040	.049	.052	.053

[1] Specimens were 3- by 4- by 16-inch beams. They were stored in laboratory air at 70° F., relative humidity 50 percent. Each result is the average of 3 tests.

drying shrinkage, two for freezing and thawing, and two for flexural strength tests as a standard of comparison for the specimens frozen and thawed.

The specimens for drying shrinkage were stored in laboratory air maintained at a constant temperature of 70° F.±2° and constant relative humidity of 50 percent±2 percent. Measurements were made at periodic intervals.

Two groups of freezing and thawing tests were conducted. In the first group the tests were made on specimens cured continuously in moist air for 230 days before freezing was started. Sonic tests were made at periodic intervals on these beams during freezing and thawing,[5] and flexural strength tests were made after 25 cycles of freezing and thawing. Companion beams, moist cured continuously, were also tested for flexural strength at the same age as those subjected to freezing and thawing.

In the second group of tests, two of three specimens previously used for drying shrinkage measurements were, at the conclusion of the drying period, soaked in water for 7 days and then frozen and thawed in water, using the same cycle as was used in the first group. Freezing and thawing was discontinued for 128 days at the end of 60 cycles, after which the cycles of freezing and thawing were again started.

Values for percentage of contraction in air are given in table 8. The initial readings were made after the specimens had been in the mold, under wet burlap, for 24 hours. The specimens were then placed in air storage at 70° F.

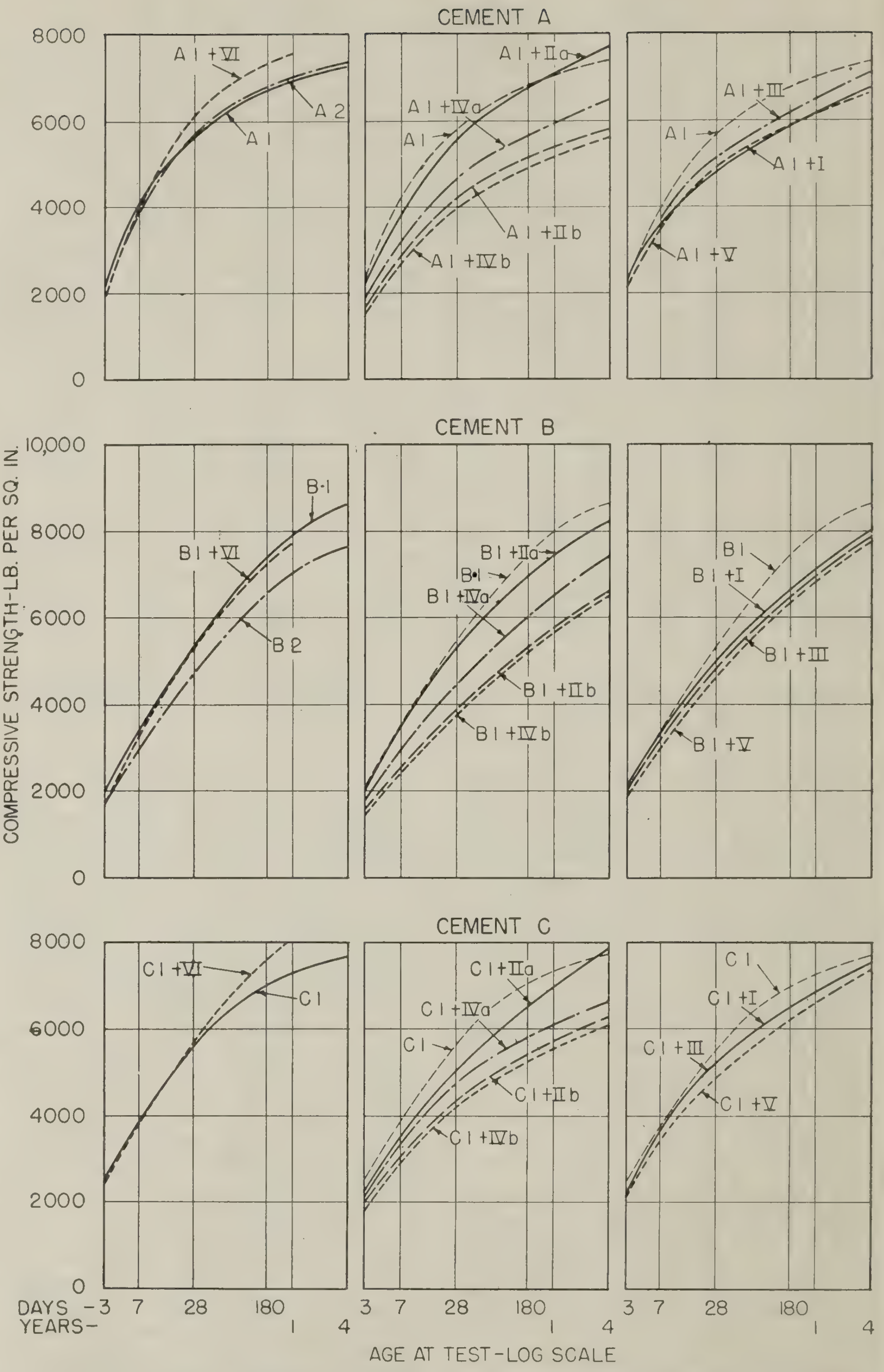


Figure 3.—Effect of blending cements on the compressive strength of concrete.

and 50 percent relative humidity. The values shown in the table were taken from smooth curves.

As may be seen in figure 5, there is no outstanding difference in the shrinkage of concrete made with any of the cements or blends tested. In the case of cement *A1* blended with the air-entrained natural cement *IIb*, the curve indicates slightly greater shrinkage than the other cements tested. It is very doubtful that this small difference is significant. There is also a slight difference in the shrinkage characteristics of concrete containing cement *A* as compared to cement *B*, but this may be a function of their composition. The principal difference in compound composition between the two cements is the tricalcium aluminate content, in which cement *A* is high (12 percent) while cement *B* is low (6 percent).

The first group of freezing and thawing tests was made on 3- by 4- by 16-inch concrete beams cured continuously in moist air for 230 days before freezing started. The specimens were frozen and thawed immersed in water.

[5] Changes in the modulus of elasticity of a concrete beam subjected to alternate freezing and thawing are indicative of the progressive disintegration of the specimen. The modulus of elasticity can be determined by sonic measurement of the specimen's natural frequency of transverse vibration. The specimen is electromechanically vibrated at varying frequencies and an electric pick-up registers the amplitude of vibration, which reaches a well-defined peak at the natural frequency of the specimen. The modulus of elasticity is derived by formula from the weight and dimensions of the specimen and the square of its natural frequency, N^2. Comparison of the effects of freezing and thawing on beams can be made directly on the basis of N^2 when expressed as a percentage change. Details of the sonic test are given in *Application of Sonic Method to Freezing and Thawing Studies of Concrete*, by F. B. Hornibrook; American Society for Testing Materials Bulletin No. 101, December 1939, p. 5.

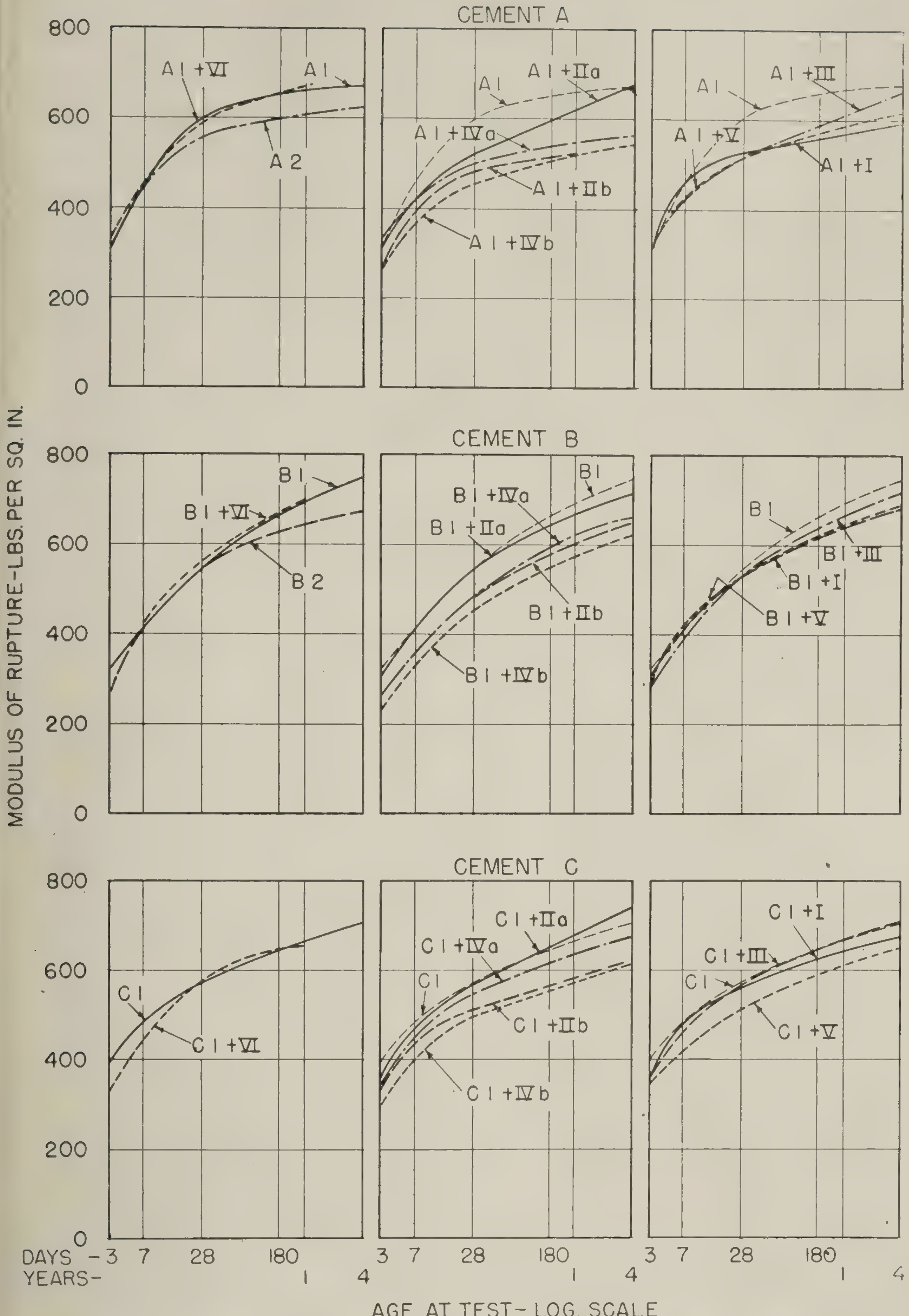


Figure 4.—Effect of blending cements on the flexural strength of concrete.

Table 9.—*Reduction in frequency squared of beams [1] frozen and thawed in water, series 2 tests*

Cements	Reduction in frequency squared [2] (expressed in percent) after freezing and thawing for—				
	5 cycles	10 cycles	15 cycles	20 cycles	25 cycles
A1	34	43	58	62	72
A2	15	21	24	31	35
A1+I	23	35	37	51	66
A1+IIa	48	69	77	85	89
A1+IIb	19	25	28	41	48
A1+III	13	18	22	29	33
A1+V	15	22	27	31	40
A1+VI	40	61	72	79	82
B1	51	73	84	88	92
B2	12	14	22	25	31
B1+I	32	49	71	74	80
B1+IIa	63	80	88	91	95
B1+IIb	15	23	32	35	44
B1+III	11	17	25	26	33
B1+V	12	19	28	32	38
B1+VI	53	75	85	91	93

[1] The 3- by 4- by 16-inch beams were stored continuously in moist air for 230 days before freezing.
[2] Each value is the average of readings taken on 3- and 4-inch axes of 2 beams.

the temperature range being from 70° to −20° F. The natural frequency of vibration of each specimen was determined sonically at 70° F. just before freezing and thawing started and at the completion of each five cycles of freezing and thawing. The results of these tests are shown in table 9 and are plotted in figure 6. The test was discontinued at the end of 25 cycles because of the marked reduction in natural frequency exhibited by most of the specimens. The specimens were then tested as beams and the flexural strength determined. At the same time continuously moist-cured companion specimens that had not been frozen and thawed were tested, and the reduction in strength caused by freezing and thawing computed.

In table 10 are given the flexural strength data and the percentage drop in sonic modulus of elasticity expressed in terms of the frequency squared, N^2. The values for modulus of rupture of the unfrozen specimens refer to the strength of the specimens that were stored in moist air continuously and tested at the same time the specimens subjected to alternate freezing and thawing were tested. These values are used in computing the values tabulated as reduction in modulus of rupture.

No unit weight determinations were made for the concrete mixes used in the series 2 tests, and therefore it was not possible to compute the air contents of these mixes. In order to obtain data on their relative air contents a supplementary series of tests was run later. These data are shown in table 11. Comparing the calculated values for air content as shown in this table with the corresponding values for the mixes used in series 1 as shown in table 4, it is found that in the cases of those combinations which can be classified as nonair-entraining the values for series 2 are slightly lower than those for series 1. However, the differences average less than 0.5 percent. In the case of the concretes that can be classed as air entraining—those made with cement *A2* and the combinations of *A1* and *B1* with natural cements *IIb*, *III*, and

Table 10.—*Reduction in modulus of rupture and frequency squared of beams [1] after 25 cycles of freezing and thawing in water, series 2 tests*

Cements	Modulus of rupture [2] of—		Reduction in—	
	Unfrozen specimens [3]	Frozen specimens	Modulus of rupture	Frequency squared (4)
	Lb. per sq. in.	*Lb. per sq. in.*	*Percent*	*Percent*
A1	790	245	69	72
A2	770	525	32	35
A1+I	730	285	61	66
A1+IIa	705	130	82	88
A1+IIb	695	295	58	48
A1+III	720	430	40	33
A1+V	720	395	45	40
A1+VI	800	180	78	82
B1	780	110	86	92
B2	815	515	37	32
B1+I	870	200	77	81
B1+IIa	780	90	88	94
B1+IIb	745	325	56	44
B1+III	850	445	48	35
B1+V	[5] 830	440	47	39
B1+VI	895	95	89	93

[1] The 3- by 4- by 16-inch beams were stored continuously in moist air for 230 days before freezing.
[2] Beams were tested with center-point loading on a 14-inch span. The beams were tested so that the 3-inch dimension was the depth of the beams as loaded; thus the side of the beam as molded was in tension. Each value is the average of 2 tests unless otherwise noted.
[3] Specimens were stored continuously in moist air at 70° F. until tested, and were tested at the same time freezing and thawing specimens were tested.
[4] Values are based on the average of readings on 3- and 4-inch axes of 2 beams.
[5] One test only.

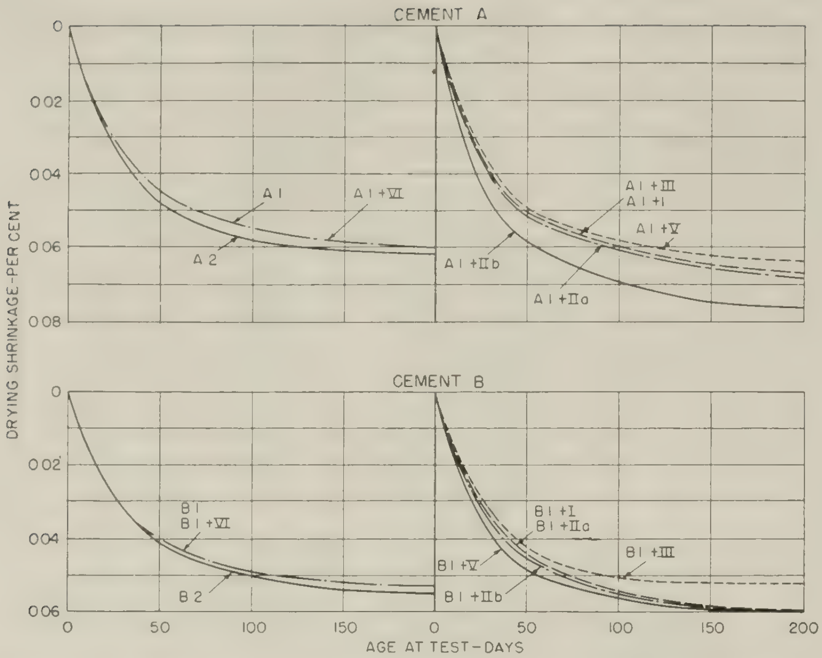


Figure 5.—Effect of blending cements on drying shrinkage of concrete.

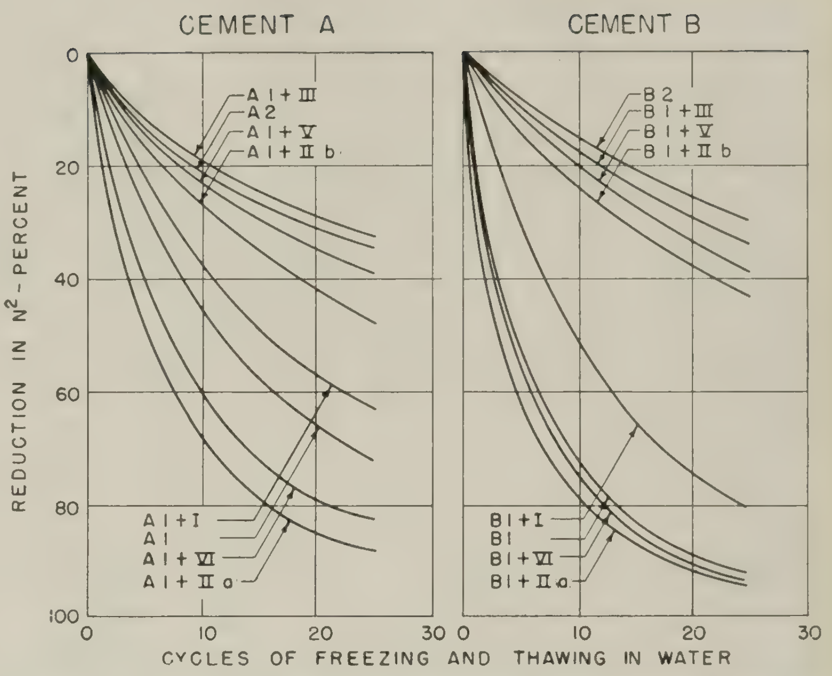


Figure 6.—Effect of blending cements on resistance of concrete to freezing and thawing.

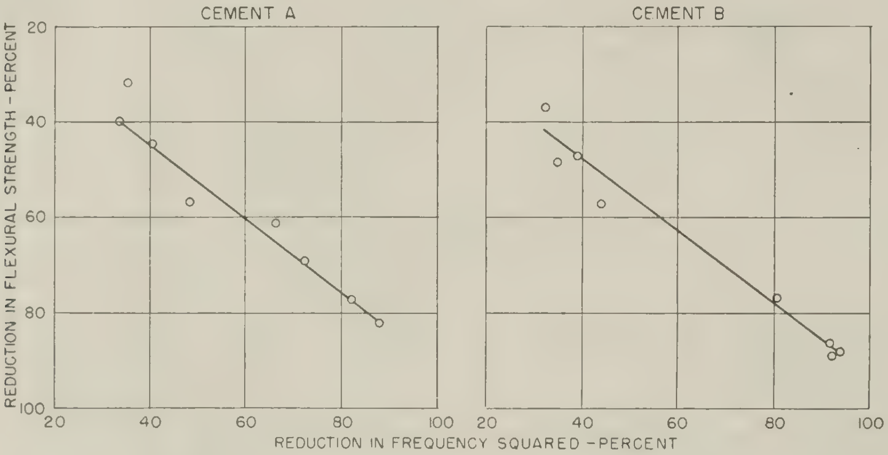


Figure 7.—Relation of reduction in flexural strength to reduction in frequency squared after 25 cycles of freezing and thawing.

V—the values in series 2 are somewhat higher, the average difference being about 1.2 percent. The one exception is air-entraining cement *A2* which shows lower air content in series 2. Cement *B2* was not available for the supplementary test in series 2.

Due to the fact that no determinations of air content were made on the concrete actually used in the freezing and thawing tests, only general reference to the relative air contents as "high" or "low" is made in the discussion of these data. It is felt that the values reported in tables 4 and 11 are sufficiently consistent to warrant this classification.

Table 11.—*Mix* [1] *data, series 2 supplemental tests* [2]

Cements [3]	Water content [4]	W_c [5]	Slump	Weight of fresh concrete	Actual cement content [4]	Air content: Calculated	Air content: Measured [6]
	Gal. per bag		*Inches*	*Lb. per cu. ft.*	*Bags per cu. yd.*	*Percent*	*Percent*
A1	5.8	0.169	3.5	147.8	5.8	0.9	1.7
A2	5.7	.168	3.8	146.4	5.8	2.0	2.6
A1+I	5.8	.169	3.6	146.4	5.8	1.7	2.5
A1+IIa	5.8	.169	2.6	147.2	5.8	1.1	1.6
A1+IIb	5.8	.169	4.2	138.9	5.5	6.7	7.3
A1+III	5.8	.169	3.7	142.9	5.7	4.0	5.0
A1+V	5.8	.169	3.5	141.9	5.6	4.7	5.3
A1+VI	5.8	.169	3.2	147.0	5.8	1.2	2.1
B1	5.9	.172	3.6	147.6	5.8	.9	1.4
B2 [7]							
B1+I	5.9	.172	3.1	146.2	5.8	1.7	2.0
B1+IIa	6.0	.174	3.4	147.0	5.8	.9	1.7
B1+IIb	5.9	.172	4.1	139.7	5.5	6.1	6.8
B1+III	5.9	.172	3.5	143.1	5.7	3.7	4.8
B1+V	5.9	.172	3.4	142.3	5.6	4.3	5.5
B1+VI	6.0	.174	3.4	147.0	5.8	.9	1.8

[1] Mix by dry weight, in pounds=94:194:339, using 1-inch to No. 4 gravel.
[2] These mixes were made about 2½ years after the original mixes in series 2 to check air content. The cements were the same as used in the original series.
[3] Where natural or slag cement was used, proportions were 86 percent portland cement to 14 percent natural or slag cement by weight.
[4] Where natural or slag cement was used, 94 pounds was considered as one bag.
[5] Water per unit volume of concrete, based on actual yield test.
[6] Using Pearson pycnometer.
[7] Cement *B2* was not available.

Referring to the left-hand diagram in figure 6 (which gives the results for cement *A*), it is noted that there is a marked difference in resistance to freezing and thawing resulting from the use of different blends. The greatest reductions in N^2 were found with blends containing natural cement *IIa* and the slag cement *VI*. The plain portland cement *A1* gave results but slightly better. These concretes all had low air contents.

Greatly increased resistance was obtained with the air-entraining portland cement *A2* and with the blends of *A1* with natural cements *IIb*, *III*, and *V*, all of which gave relatively high air contents in concrete. The blends containing natural cement *I* gave intermediate results. The air content of these concretes could be classed as intermediate.

In the right-hand diagram of figure 6 are shown similar curves for cement *B*. The essential difference between these results and those for cement *A* is the poorer resistance shown for the plain portland cement and the blends containing cements *IIa* and *VI*. In this case there is a greater spread between the concretes of poor resistance and those showing good resistance than with cement *A*. The results with blends of cement *B1* confirm the conclusion from the results with cement *A1* that the amount of entrained air is the governing factor between good and poor resistance to the action of freezing and thawing in water. The order of resistance using cement *B1* was, with one exception, the same as that with cement *A1*.

The reduction in natural frequency squared (N^2) was accompanied by a corresponding reduction in flexural strength. This relation is shown in figure 7. The data indicate that the reduction in flexural strength can be predicted with reasonable accuracy from a determination of the reduction in N^2.

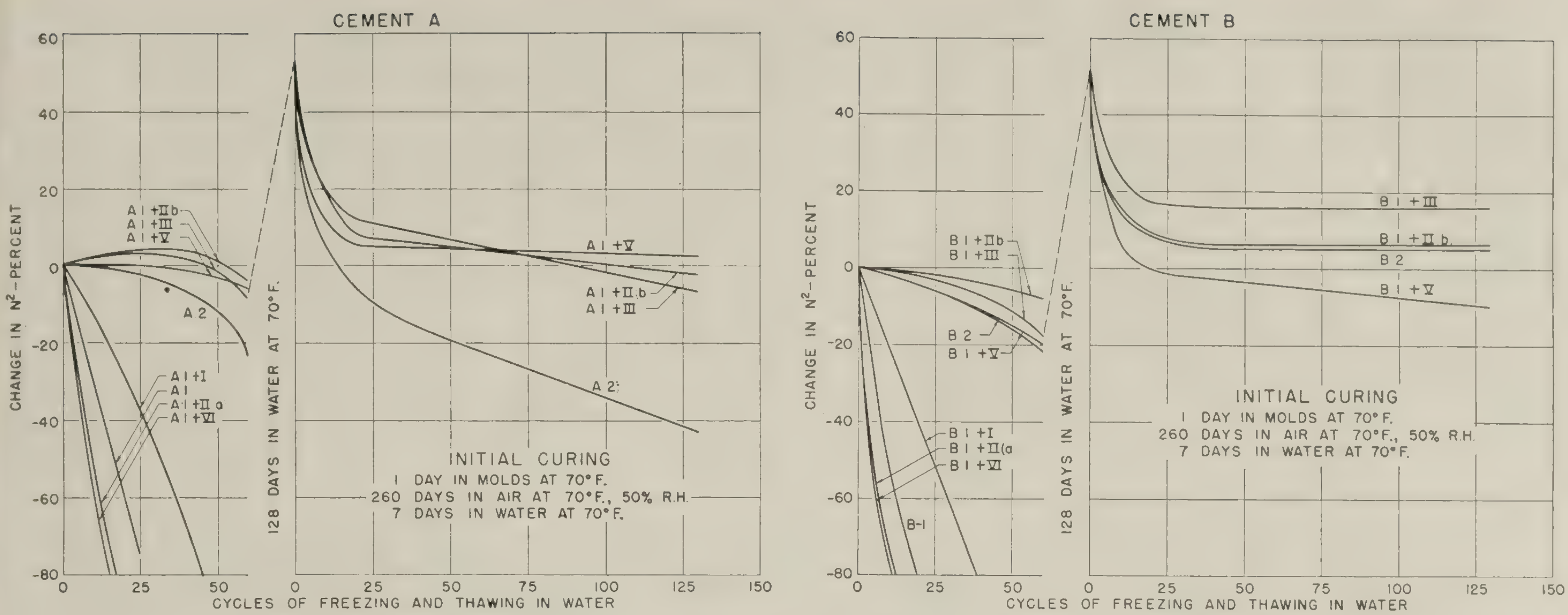


Figure 8.—Effect of blending cements on resistance of concrete to freezing and thawing.

A second set of beam specimens in series 2 (the same specimens used for shrinkage measurements) were cured under wet burlap for 24 hours, followed by storage in laboratory air at 70° F. and 50 percent relative humidity for 260 days. They were then placed in water for 7 days, after which they were frozen and thawed in water, using the procedure previously described. At the end of 60 cycles of freezing and thawing, at which time many of the specimens had disintegrated, the test had to be discontinued temporarily because of other needs for the freezing equipment. Those specimens still remaining were stored in water at 70° F. for 128 days, after which they were again subjected to the freezing and thawing cycle. To simplify discussion, the first group of 60 cycles of freezing and thawing is referred to as the first phase and the second group of freezing and thawing cycles following the 128-day rest period as the second phase.

Results of these interrupted freezing and thawing tests are given in table 12 and are shown graphically in figure 8. The data for cement *A* in table 12 and figure 8 indicate very rapid deterioration during the first phase for the plain portland cement *A1* and the blends which had low air contents. Air-entraining cement *A2* and the blends which had high air contents, however, showed greatly superior resistance in comparison. These combinations showing good resistance were the same as those showing good resistance in the tests made after continuous water curing (see fig. 6). However, the shapes of the curves are reversed. The general trend at 60 cycles was a rapid drop in N^2. As previously explained, those combinations which had not failed at 60 cycles were stored in water for 128 days. At the end of this period they recovered the loss in N^2 indicated at 60 cycles and gained about 45 percent over the initial amount indicated before the first phase of freezing and thawing. As may be seen from the curves for the second phase, the rate in reduction of N^2 was very rapid for the first 20 cycles of freezing and thawing but thereafter, with the exception of the curve for air-entraining cement *A2*, the rate was relatively slow. At 145 cycles of freezing and thawing the value of N^2 for the three blends was about the same as that obtained at the start of the first phase of the test.

The shapes of the curves for the second phase are not the same as those for the first phase but are more like those shown in figure 6. Apparently the resistance to deterioration is influenced by the curing and the moisture condition of the specimens at the time freezing and thawing is started, which accounts for the difference in the shape of the curve. The curves for the second phase of cement *A* in figure 8 and also those shown in figure 6 represent concretes that had been subjected to prolonged moist curing. Conversely, the results for the first phase in figure 8 are for concretes exposed to the drying action of air at 70° F. and 50 percent relative humidity over a long period of time.

It has been shown by other investigations that specimens which have been subjected to cycles of freezing and thawing will, when given a rest period in water, show a recovery in sonic modulus of elasticity.[6]

Weight determinations made during the first phase indicated that all the specimens absorbed water during the first 10 cycles of freezing and thawing and some continued to gain in weight up to 40 cycles. Those specimens that gained the least in weight were those which showed low air contents. They also broke down rapidly in the freezing and thawing tests. Those specimens that gained the most in weight over a longer period were those of relatively high air contents. These gave good resistance in the freezing test. The results indicate that those concretes of low air content were practically saturated after a 7-day immersion period. The concretes having relatively high air contents were not completely saturated at the end of the 7-day im-

[6] *Progress Report, Committee on Durability of Concrete;* Proceedings of the Highway Research Board, vol. 24, 1944, p. 196, fig. 17.

Table 12.—*Effect of interrupted freezing and thawing on reduction of frequency squared of beams [1] frozen and thawed in water, series 2 tests*

Cements	Reduction [2] in frequency squared (expressed in percent) after freezing and thawing [3]													
	Number of cycles in first freezing and thawing phase						Number of cycles in second freezing and thawing phase							
	10	20	30	40	50	60	0	20	40	60	80	100	120	145
A1	28	50	(4)	------	------	------	------	------	------	------	------	------	------	------
A2	2	+6	2	7	15	24	+50	6	17	20	25	36	40	53
A1+I	10	14	44	69	87	(5)	------	------	------	------	------	------	------	------
A1+IIa	46	74	(4)	------	------	------	------	------	------	------	------	------	------	------
A1+IIb	0	+5	+4	+3	+2	4	+44	+8	+8	+1	+4	+4	2	2
A1+III	1	+6	+4	0	1	9	+53	+11	+12	+4	+1	3	6	9
A1+V	2	+1	0	2	3	6	+41	+6	+5	+5	+2	+3	+2	+2
A1+VI	62	97	(6)	------	------	------	------	------	------	------	------	------	------	------
B1	50	78	(4)	------	------	------	------	------	------	------	------	------	------	------
B2	8	3	9	11	16	22	+49	+9	+9	+4	+6	+3	+7	+5
B1+I	24	36	70	84	95	(5)	------	------	------	------	------	------	------	------
B1+IIa	75	93	(4)	------	------	------	------	------	------	------	------	------	------	------
B1+IIb	3	+2	3	5	6	7	+40	+10	+12	+7	+10	+7	+8	+6
B1+III	2	+1	3	9	11	18	+51	+17	+15	+15	+22	+17	+17	+18
B1+V	5	2	8	12	17	19	+40	1	1	11	8	12	11	8
B1+VI	80	87	(4)	------	------	------	------	------	------	------	------	------	------	------

[1] Specimens were 3- by 4- by 16-inch beams stored in laboratory air at 70° F. and 50 percent relative humidity for 260 days, followed by 7 days in water prior to start of first freezing.
[2] Each value is the average of readings taken on 3- and 4-inch axes. Plus values indicate an increase in frequency squared.
[3] Specimens were stored in water at 70° F. for 128 days between the first and second freezing and thawing cycle phases.
[4] Failed at 25 cycles. [5] Failed at 50 cycles. [6] Failed at 20 cycles.

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Table 13.—*Mix*[1] *data, series 3 tests*

Cements [2]	Water content [3]	Slump
	Gal. per bag	*Inches*
A1	4.8	3.3
A2	4.6	3.7
A1+I	4.8	3.1
A1+IIa	4.8	2.8
A1+IIb	4.7	3.0
A1+III	4.8	2.6
A1+IVa	4.9	2.7
A1+IVb	4.8	2.8
A1+V	4.8	3.3
A1+VI	4.8	3.5
B1	4.8	2.8
B2	4.6	3.3
B1+I	4.8	3.1
B1+IIa	4.8	3.1
B1+IIb	4.7	2.5
B1+III	4.8	2.5
B1+IVa	4.9	3.1
B1+IVb	4.9	2.9
B1+V	4.8	2.8
B1+VI	4.8	3.3

1 Mix by dry weight, in pounds=94:177:358, using 1½-inch to No. 4 gravel.
2 Where natural or slag cement was used, proportions were 86 percent portland cement to 14 percent natural or slag cement, by weight.
3 Where natural or slag cement was used, 94 pounds was considered as one bag.

Table 14.—*Weight loss of the core tops due to freezing and thawing,*[1] *series 3 tests*

Cements	Weight loss (expressed in percent) after freezing and thawing for—							
	25 cycles	50 cycles	75 cycles	100 cycles	125 cycles	150 cycles	175 cycles	200 cycles
A1	2.4	5.7	8.6	15.5	(2)	----------	----------	----------
A2	0	.1	.2	.6	1.6	3.1	5.3	7.1
A1+I	.2	.8	1.3	2.8	5.4	7.9	10.7	13.0
A1+IIa	2.1	7.1	[3] 47.2	(4)	----------	----------	----------	----------
A1+IIb	0	.7	1.9	3.5	6.6	9.2	14.7	19.4
A1+III	0	.2	1.0	2.3	4.4	6.3	9.2	12.3
A1+IVa	.3	.7	1.5	3.1	5.6	7.1	10.1	12.3
A1+IVb	0	0	.5	1.8	4.9	6.9	11.2	15.7
A1+V	0	.2	.8	2.3	4.2	6.1	9.0	11.1
A1+VI	.2	1.3	4.1	[3] 39.9	[3] 49.6	(5)	----------	----------
B1	1.7	4.3	8.3	11.1	17.4	[3] 48.1	(6)	----------
B2	.2	.5	1.1	2.0	3.4	5.4	7.4	9.0
B1+I	.6	2.2	4.8	9.4	16.6	28.5	[2] 59.1	(7)
B1+IIa	2.4	5.9	9.8	[3] 69.1	(8)	----------	----------	----------
B1+IIb	.1	.4	.9	1.9	3.7	6.0	10.3	13.5
B1+III	.6	1.4	1.9	3.1	4.8	6.6	10.2	12.6
B1+IVa	.3	1.3	2.2	3.7	6.6	10.6	15.7	21.0
B1+IVb	0	.1	.6	1.4	2.9	4.5	7.2	9.1
B1+V	0	.2	.8	2.0	3.6	5.4	9.0	11.0
B1+VI	4.2	9.2	26.0	(4)	----------	----------	----------	----------

1 Tests were made on 2-inch disks cut from the tops of cores 6 inches in diameter and 8 inches in height. Freezing and thawing was done in a 10-percent calcium chloride solution. Each value is the average of three tests.
2 Failed at 125 cycles.
3 One specimen disintegrated, and weight loss was assumed to be 100 percent for that specimen.
4 Failed at 80 cycles.
5 Failed at 130 cycles.
6 Failed at 160 cycles.
7 Failed at 180 cycles.
8 Failed at 110 cycles.

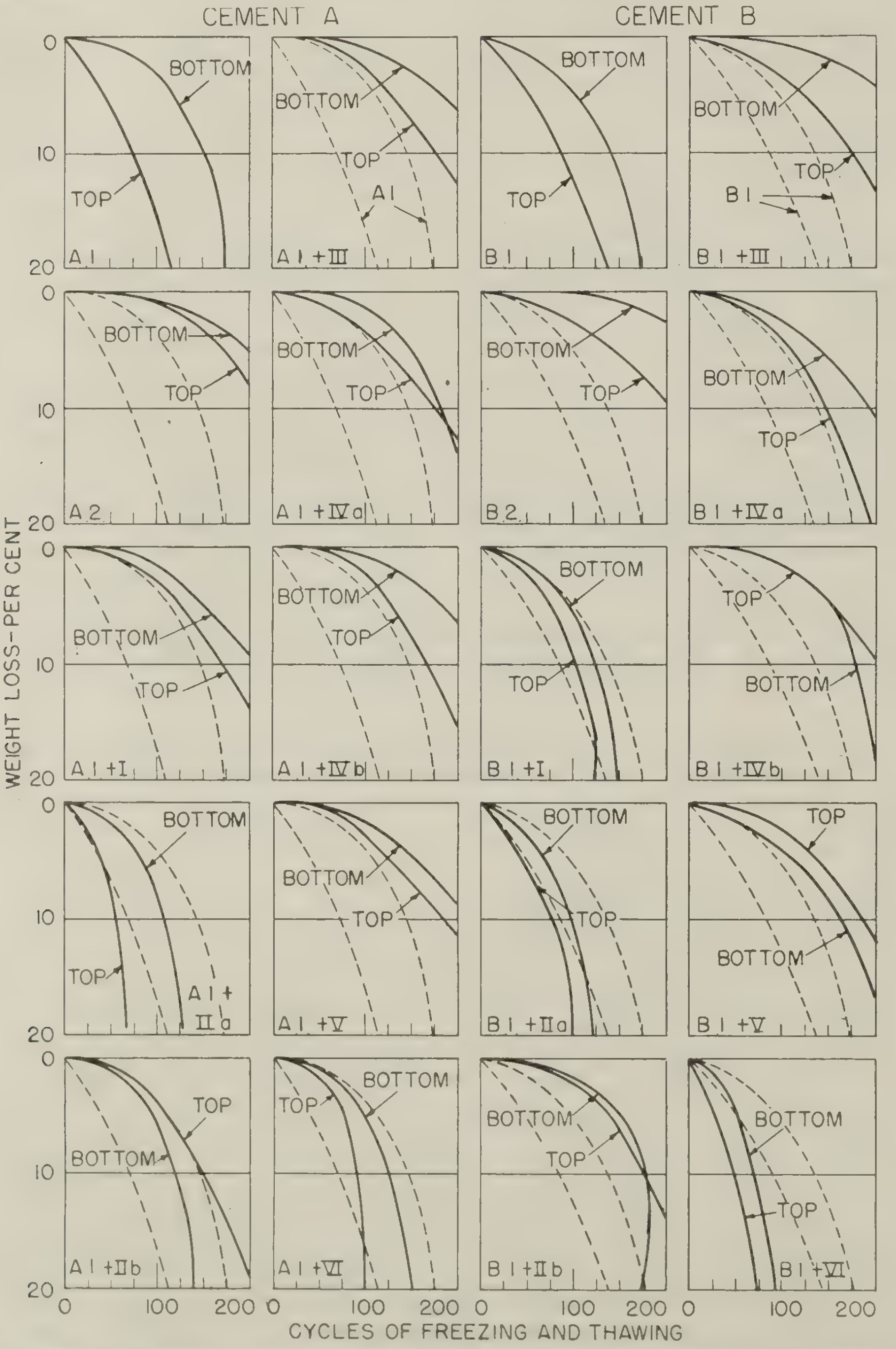

Figure 9.—Weight losses of disks cut from tops and bottoms of frozen and thawed concrete cores.

mersion period and continued to absorb water during the freezing and thawing cycle.

The foregoing observations are offered as an explanation of the shape of the curves for the concretes showing good resistance in the freezing and thawing test during the first phase rather than as an explanation of the difference in behavior between air-entrained and non-air-entrained concrete.

Results shown in figure 8 for blends with cement *B* parallel very closely those shown for cement *A*. The plain cement *B1* and those blends of low air content gave very poor resistance, the order of resistance being exactly the same as with cement *A*. Concretes showing good resistance at the end of 60 cycles of the first phase also exhibited about the same increase in N^2 during the 128-day water storage period as was shown for the corresponding blends of cement *A*.

The data show that interrupting the daily cycle of freezing and thawing and allowing the test specimens to remain unfrozen in water will result in a recovery in sonic modulus.

THE EFFECT OF SEGREGATION

The tests comprising series 3 of the investigation were designed to study the effect of segregation of the constituent materials due to manipulation of the surface in the finishing operations in laboratory-mixed concrete placed to simulate pavement construction practice. Two nonair-entraining and two air-entraining portland cements, and blends of the two former with each of the seven natural cements and the slag cement were used. Mix data are given in table 13. Grading and other physical properties of the aggregates used are shown in table 3. Experience has shown that the typical Potomac River sand grading used in the series 1 and 2 tests does not induce appreciable bleeding or water gain in concrete; and on this account a somewhat coarser sand was used in the series 3 tests. The coarse aggregate was the same as that used in series 1.

The specimens for the tests were slabs 2 feet square and 8 inches thick, cast in forms 2 feet

Table 15.—*Weight loss of the core bottoms due to freezing and thawing,*[1] *series 3 tests*

Cements	Weight loss (expressed in percent) after freezing and thawing for—							
	25 cycles	50 cycles	75 cycles	100 cycles	125 cycles	150 cycles	175 cycles	200 cycles
A1	0.5	0.9	1.6	2.7	5.7	9.6	[2] 53.2	(3)
A2	.4	.5	.6	.7	1.2	1.9	3.5	4.5
A1+I	0	.2	.6	1.5	3.3	5.1	7.2	8.8
A1+IIa	.4	1.1	3.3	7.7	18.5	(4)	----	----
A1+IIb [5]	0	1.5	2.5	4.9	11.1	[2] 38.5	[2] 41.6	[2] 43.7
A1+III	0	0	.2	.8	1.7	2.5	4.8	5.8
A1+IVa	0	.5	.7	1.4	2.8	4.0	9.8	13.6
A1+IVb	.4	.4	.4	.9	1.5	2.3	4.2	5.9
A1+V	.5	.6	.7	1.5	2.3	4.4	7.0	8.4
A1+VI	.5	.8	2.1	6.0	9.7	[2] 41.9	(6)	----
B1	.4	2.0	3.5	4.5	7.0	11.6	20.1	[2] 47.1
B2	0	.1	.1	.2	.3	.8	1.7	2.3
B1+I	.2	1.1	2.4	5.8	10.0	15.0	[2] 55.1	[2] 55.7
B1+IIa	.7	2.5	5.3	(7)	----	----	----	----
B1+IIb	.4	.4	.5	.9	2.3	3.9	9.2	[2] 43.7
B1+III	0	.2	.4	.6	1.0	1.7	3.1	4.0
B1+IVa	.5	.7	1.4	2.2	3.8	5.4	8.0	10.9
B1+IVb	.1	.5	1.0	1.7	2.9	4.6	11.6	17.7
B1+V	.4	1.8	2.5	3.9	5.7	7.7	12.1	16.7
B1+VI	.9	4.4	12.8	(8)	----	----	----	----

[1] Tests were made on 2-inch disks cut from the bottoms of cores 6 inches in diameter and 8 inches in height. Freezing and thawing was done in a 10-percent calcium chloride solution. Each value is the average of three tests except as noted.
[2] One specimen disintegrated, and weight loss was assumed to be 100 percent for that specimen.
[3] Failed at 190 cycles.
[4] Failed at 140 cycles.
[5] Average of 2 tests only.
[6] Failed at 170 cycles.
[7] Failed at 100 cycles.
[8] Failed at 90 cycles.

wide by 4 feet long and screeded with an 8-inch screed weighing 100 pounds per foot.

A tar-paper separator installed full depth midway in the form made it possible to obtain two 2- by 2-foot test slabs without restricting the lateral movement of the concrete caused by the reciprocating motion of the screed. The screed was operated with its long axis parallel to the 4-foot axis of the slabs. Following two passes of the screed, the slabs were belted in a manner similar to that employed in actual road practice. The procedure described was carried out to simulate construction practice and to induce bleeding in the slabs similar to that which frequently occurs in the field. The slabs were cured the first 24 hours under wet burlap in the laboratory, followed by 1 year's curing in the moist room. Five cores 6 inches in diameter were then drilled from each slab. These cores were returned to the moist room for another year of curing, after which 2-inch disks were cut from the tops and bottoms of three of the five cores from each slab. These disks were returned to the moist room for another 15 days' curing, after which they were air-dried at 70° F. and 50 percent relative humidity for 7 days. They were then immersed in a 10-percent calcium chloride solution for 24 hours before being subjected to alternate freezing and thawing in a 10-percent calcium chloride solution. The disks were approximately 750 days old when freezing started.

The weight losses after freezing and thawing are given in table 14 for the disks cut from the tops of the cores, and in table 15 for those cut from the bottoms. The data are shown graphically in figure 9. The dotted-line curves in each chart represent the weight loss curves for the portland cements *A1* and *B1*, and are provided for comparison with weight-loss curves for the blends.

In general, the bottoms of the cores showed more resistance to freezing and thawing in calcium chloride solution than the tops. There were only three exceptions out of 20 cement combinations. The plain portland cement concretes *A1* and *B1* and the blends *A1+IIa*, *A1+IIb*, *A1+VI*, *B1+I*, *B1+IIa*, and *B1+VI* all showed relatively poor resistance to freezing and thawing in calcium chloride solution. Four of the blends, *A1+IIa*, *A1+VI*, *B1+IIa*, and *B1+VI*, had poorer resistance than the comparable plain portland cements, whereas the air-entraining portland cements and the blends of higher air content were of benefit in producing better resistance than was furnished by the plain cement. This confirms the conclusion based on freezing and thawing tests of beam specimens in water. The improvement in resistance over the plain portland cement concrete was generally more pronounced for the tops than for the bottoms of the cores.

The remaining two cores of the original five drilled from each slab were given approximately 15 months' additional moist curing, after which 2-inch disks were cut from the tops and bottoms of each core and tested for specific gravity and absorption. These disks were approximately 830 days old when tested. Upon removal from moist storage they were dried to constant weight at 350° F. They were then boiled for 5 hours and allowed to stand in water for 19 additional hours, after which they were again weighed. From these data, absorptions and specific gravities were calculated by two different methods: First, on the basis of weights resulting from curing for 830 days in moist air and second, on the basis of weights after boiling for 5 hours followed by immersion for 19 hours.

From the results given in table 16 it is readily apparent that the disks from the top showed higher absorption than those from the bottom of the slabs. This was true irrespective of whether plain portland cement, air-entraining portland cement, or blended cement was used. The disks representing air-entraining concrete showed nearly as great a difference between top and bottom as those representing concrete in which the amount of entrained air was low.

The use of air-entraining concrete did affect the ratio of absorption after boiling for 5 hours to absorption due to moist curing for 830 days. Considering the plain portland cements and the blends containing natural cements *IIa*, *IVa*, and *VI*, it will be noted that approximately the same values were obtained for absorption in both tests. Those combinations all had low air contents. The blends containing cements *IIb* and *IVb* gave much higher values for absorption in the 5-hour boiling test than in the 830-day moist curing test. These two blends showed the highest air contents of any concretes tested.

SUMMARY

The following summarizes the test results:

1. Blends containing the nonair-entraining natural cements had higher strengths than blends of air-entraining natural cements at all ages of test.

2. For all combinations the strengths of the concretes were inversely proportional to the air contents.

Table 16.—*Specific gravity and absorption of disks cut from tops and bottoms of cores,*[1] *series 3 tests*

Cements	Specific gravity,[2] 830 days' moist curing		Absorption,[3] 830 days' moist curing		Specific gravity (dry) [4]		Absorption,[5] 5 hours' boiling	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
			Percent	*Percent*			*Percent*	*Percent*
A1	2.43	2.47	5.06	4.27	2.33	2.37	4.94	4.22
A2	2.36	2.42	5.90	4.56	2.23	2.32	6.39	4.93
A1+I	2.41	2.45	5.38	4.50	2.29	2.34	5.60	4.76
A1+IIa	2.42	2.46	5.54	4.62	2.30	2.35	5.48	4.62
A1+IIb	2.33	2.38	5.85	4.58	2.20	2.27	7.01	5.66
A1+III	2.35	2.41	6.18	4.56	2.22	2.31	6.88	5.26
A1+IVa	2.38	2.43	6.27	4.84	2.24	2.32	6.52	5.15
A1+IVb	2.32	2.37	6.10	4.76	2.18	2.27	7.50	5.64
A1+V	2.36	2.41	5.96	4.83	2.22	2.30	6.80	5.38
A1+VI	2.42	2.46	5.71	4.45	2.29	2.36	5.65	4.39
B1	2.44	2.50	5.56	4.08	2.31	2.40	5.38	4.03
B2	2.34	2.42	5.64	4.34	2.21	2.32	6.74	4.96
B1+I	2.39	2.45	5.96	4.35	2.26	2.35	6.32	4.62
B1+IIa	2.41	2.46	5.98	4.66	2.27	2.35	5.88	4.52
B1+IIb	2.34	2.39	5.76	4.73	2.21	2.29	6.80	5.57
B1+III	2.36	2.44	6.22	4.70	2.22	2.33	6.66	4.90
B1+IVa	2.38	2.44	6.34	4.88	2.24	2.32	6.45	4.92
B1+IVb	2.33	2.41	6.38	4.38	2.19	2.31	7.27	5.10
B1+V	2.35	2.44	6.39	4.39	2.21	2.34	6.98	4.96
B1+VI	2.42	2.48	5.88	4.54	2.29	2.37	5.82	4.38

[1] Specimens were 2-inch disks cut from the tops and bottoms of cores 6 inches in diameter and 8 inches in height. Each value average of two tests.
[2] Bulk specific gravity based on the wet weight after 830 days in moist air.
[3] Based on the wet weight after 830 days in moist air.
[4] Bulk specific gravity based on weight after drying to constant weight at 350° F.
[5] Based on weight after 5-hour boiling and 19 hours in water.

3. The blends of the slag cement with each of the three portland cements had equal or higher strengths at 1 year than the same portland cements without blending.

4. There were no outstanding differences in the drying shrinkage characteristics of concretes made with any of the cement combinations tested.

5. Greatly increased resistance to freezing and thawing in water was obtained with the air-entraining portland cements and with the blends which produced relatively high air contents in concrete.

6. In the interrupted cycles of freezing and thawing in water, only those concretes having relatively high air contents had sufficient resistance to disintegration to survive, as indicated by a drop in N^2 of less than 25 percent in the first phase of 60 cycles. After a further storage period of 128 days in water these same high-air concretes developed further resistance as indicated by a recovery of N^2 that surpassed the initial reading at the start of the first phase. In the subsequent freezing and thawing (second phase) they showed better resistance than they did at 60 cycles in the first phase.

7. Under continued moist curing, concretes with the lower percentages of entrained air absorbed their maximum capacity for water; those containing high percentages of air did not, but the voids filled with water after the drying and boiling treatment.

8. In practically all cases, the bottoms of the cores gave better resistance to freezing and thawing in calcium chloride solution than the tops. The air-entraining portland cements and in general the natural cement blends gave better resistance than the plain portland cements.

New Publications

The Public Roads Administration has recently issued a new bulletin, Highway Statistics, Summary to 1945, in which is presented historical information of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, highway finance, and mileage. Most of the statistics are carried back over periods of from 20 to 50 years, as available data permitted. A brief explanatory test is included.

The new bulletin provides background for the annual series begun with the issuance of Highway Statistics, 1945. The second annual publication, Highway Statistics, 1946, will be available about December 21. All of these bulletins are for sale by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at the following prices:

Highway Statistics, Summary to 1945, 40 cents.

Highway Statistics, 1945, 35 cents.

Highway Statistics, 1946, about 35 cents (available after December 21).

Work of the Public Roads Administration, 1947, the annual report of the Public Roads Administration for the fiscal year ended June 30, 1947, is now available from the Superintendent of Documents at 20 cents a copy.

U. S. GOVERNMENT PRINTING OFFICE: 1947

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Federal Works Building, Washington 25, D. C.

PUBLICATIONS of the Public Roads Administration

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Please do not send orders to the Public Roads Administration.

ANNUAL REPORTS

(See also adjacent column)

Reports of the Chief of the Bureau of Public Roads:

1931, 10 cents.	1934, 10 cents.	1937, 10 cents
1932, 5 cents.	1935, 5 cents.	1938, 10 cents.
1933, 5 cents.	1936, 10 cents.	1939, 10 cents.

Work of the Public Roads Administration:

1940, 10 cents.	1941, 15 cents.	1942, 10 cents.
1946, 20 cents.	1947, 20 cents.	

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 265T . . . . . Electrical Equipment on Movable Bridges. 40 cents.

No. 191MP . . . Roadside Improvement. 10 cents.

No. 272MP . . . Construction of Private Driveways. 10 cents.

No. 1279D . . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . . Highway Bridge Location. 15 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 1 dollar.

Highways of History. 25 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks. 1 dollar.

Public Land Acquisition for Highway Purposes. 10 cents.

Public Control of Highway Access and Roadside Development (revision). 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces. 10 cents.

Legal Aspects of Controlling Highway Access. 15 cents.

House Document No. 379. Interregional Highways. 75 cents.

Highway Statistics, Summary to 1945. 40 cents.

Highway Statistics, 1945. 35 cents.

Highway Statistics, 1946. About 35 cents (available December 21).

Model Traffic Ordinance. 10 cents.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft. $1.50.

Single copies of the following publications may be obtained free upon request addressed to the Public Roads Administration. They are not sold by the Superintendent of Documents.

ANNUAL REPORTS

(See also adjacent column)

Public Roads Administration Annual Reports:

1943.	1944.	1945.

MISCELLANEOUS PUBLICATIONS

No. 279MP . . . Bibliography on Highway Lighting.

No. 296MP . . . Bibliography on Highway Safety.

No. 1036Y . . . Road Work on Farm Outlets Needs Skill and Right Equipment.

Indexes to PUBLIC ROADS, volumes 17–23, inclusive.

Bibliography on Automobile Parking in the United States.

Express Highways in the United States: a Bibliography.

Bibliography on Land Acquisition for Public Roads.

REPORTS IN COOPERATION WITH UNIVERSITY OF ILLINOIS

No. 313 . . . Tests of Plaster-Model Slabs Subjected to Concentrated Loads.

No. 314 . . . Tests of Reinforced Concrete Slabs Subjected to Concentrated Loads.

No. 315 . . . Moments in Simple Span Bridge Slabs With Stiffened Edges.

No. 336 . . . Moments in I-Beam Bridges.

No. 345 . . . Ultimate Strength of Reinforced Concrete Beams as Related to the Plasticity Ratio of Concrete.

No. 346 . . . Highway Slab-Bridges with Curbs: Laboratory Tests and Proposed Design Method.

No. 363 . . . Study of Slab and Beam Highway Bridges.

No. 369 . . . Studies of Highway Skew Slab-Bridges with Curbs. Part I: Results of Analyses.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor-Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor-Vehicle Civil Liability Act.

Act IV.—Uniform Motor-Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.