JOURNAL

of the

AMERICAN CONCRETE INSTITUTE

(ACI PROCEEDINGS Vol. 42)

Vol. 17 June 1946 No. 6

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JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Published by the American Concrete Institute. The Institute was founded 1905; incorporated in the District of Columbia in 1906 as The National Association of Cement Users; the name changed 1913 by charter amendment; reincorporated, with new statement of objects, August 8, 1945. The Journal is issued six times yearly in the months of January, February, April, June, September and November under the authority of the

Papers and other contributions prepared with a view to Journal publication should be submitted in triplicate, addressed: Secretary, Publications Committee, American Concrete Institute, 7400 Second Boulevard, Detroit 2, Michigan. Publication of a contribution does not imply the acquiescence of critics (whose advice is sought by the Committee, prior to acceptance) or of the Institute in the opinions which it expresses nor the approval of data or practice which the contribution records. Institute authority attaches only to Standards formally adopted as provided in the By-Laws. A committee report implies merely the joint contribution of an appointed group.

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Bound volumes 1 to 40 of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE (1905 to 1944) are for sale as far as available, at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

Publication address: 7400 Second Boulevard, Detroit 2, Michigan. Printed in U. S. A. Entered at the Post Office at Detroit, Michigan, as mail of the second class under provisions of the Act of March 3, 1879

the ACI JOURNAL

is edited by the Secretary of the Publications Committee under the direction of the Committee

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AMERICAN CONCRETE INSTITUTE

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Thanks to $3\frac{1}{2}$ per cent of the Institute Members who took the lead in a recent effort, the month of May broke all records since the late twenties in membership gain. 89 ACI Members were sponsors and co-sponsors of 112 Member applicants in May. June, so far, is good, but not so good. A more general participation might well be a measure of the greater benefits to ALL Members.

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Title 42-22 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Asphaltic Oil-Latex Joint-Sealing Compound*

By BRYANT W. POCOCK†

SYNOPSIS

The development of asphaltic oil-latex compounds for use in sealing expansion joints in concrete pavements is discussed. Laboratory tests devised by the Michigan State Highway Department for evaluating these seals are described and results of field installations in Michigan are reported. Tentative Michigan specifications for this type of seal are given.

INTRODUCTION

The historical background of asphalt-rubber compounds for highway purposes was characterized by the development in England and the Netherlands of various mixtures of latex with bituminous substances. These mixtures retained certain properties of the parent materials, notably the elasticity of the rubber and the adhesion, cohesion and ductility of the bitumen.

While these experiments were being conducted in Europe, the design of expansion joints for concrete pavements was undergoing its initial development in this country. American highway officials, recognizing the need for an effective sealing compound for use in excluding moisture and foreign bodies from expansion joints, tried tars, pitches, asphalts and many other substances. For some time, asphalt was held to be the best material available for use as a joint-sealing compound.

Asphalt, however, had certain disadvantages which soon became apparent. For one thing, expansion joint seals made of asphalt tended to flow at higher summer temperatures. Conversely, they became brittle in winter and lost most of their ductility, cohesion and adhesion to concrete.

It was only natural that American highway engineers should become interested in the progress of European experimentation with rubber and

*Submitted to the Institute, June 18, 1945.
†Research Chemist, Michigan State Highway Department.

asphalt mixtures in the hope of devising a joint-sealing compound of superior quality. As early as 1935, the state of California used a material consisting essentially of about 70 per cent SC-4 asphaltic oil and 30 per cent rubber latex as a seal on about 680 linear feet of expansion joints on the heavily traveled highway between Lebec and Grapevine in Kern County (1)*. These joints, spaced at 100-foot intervals, were sealed immediately after the concrete surface was completed. When inspected in 1939, after 4 years of service, the rubber compound joints were intact and in good condition.

Stanton (2) reported early experiences (1936) of the California Division of Highways with the new material, indicating the necessity of choosing a good asphaltic oil and basing its selection on high ductility at low temperatures. Stanton pointed out the deleterious effects of moisture in securing a good bond and showed how to convert the sealing compound into a crack filler by cutting the oil with gasoline before adding the latex.

By March, 1934, Woolf and Runner (3) had set up a procedure for testing various types of "resilient expansion joint fillers" in the laboratory. Among the nine types of fillers investigated were compounds of asphalt and rubber with particles of vulcanized rubber.

Encouraged by the results in California, the neighboring state of Oregon experimented with a seal made up of 30 parts of 60 per cent rubber latex and 70 parts of 150-200 penetration asphalt. The mixture was poured hot and the seal allowed to come within ¼ inch of the pavement level. The hot material was covered with rubber grindings to complete the seal and filling of the joint.

Impressed by the work in California and Oregon, engineers in Massachusetts and Oklahoma began experimenting with similar compounds. A formula was developed in Oklahoma for a joint seal using the following mixture:

Material	Parts by weight
SC-4	5.475
38 per cent Latex	2.431
Lime	0.0553
Cresol Soap Solution	0.0444
Paraffin	0.0395

The soap solution used in the formula was a proprietary material of the brand name "Kremulso." Experience in Oklahoma prior to 1939 (4) indicated that good joint seals are readily obtained with the above mix if the following precautions are taken: 1) thorough cleaning of the concrete sides of the joint; 2) accurate proportioning of materials; 3) uni-

*The numbers in parentheses refer to the bibliography appended hereto.

formity of control of mixing, temperature and consistency of each batch; 4) pouring of a uniform flush joint (seal); 5) delay of shouldering operations for one week after pouring of joints and 6) the exercise of care that joints are not poured at temperatures below 40 F.

As progress in the development of bitumen-rubber materials in the United States came to the attention of highway engineers, various proprietary or commercial preparations appeared on the American market, some of which showed considerable promise, and a few of which gave good results in actual service. Nevertheless, research continued in highway departments and other laboratories (5-9) on problems associated with the development of latex-bituminous types of joint-sealing compounds having superior properties.

The Michigan State Highway Department concerned itself at an early date with these investigations and undertook the development of a bituminous-rubber sealing compound suitable for use on expansion joints in Michigan climates (10). In December, 1936, project 36 G-4 was authorized by the Administration for the investigation and development of new joint sealers, notably types using normal latex and road oil. Whereas considerable success has attended this project during its eight-year life, the investigation is by no means closed. At the present time consideration is being given to the use of synthetic latices as substitutes for normal latex.

DEVELOPMENT OF COMPOUNDS IN MICHIGAN

Michigan investigations of joint-sealing compounds were first patterned very closely after those of California insofar as materials and properties were concerned. However, the scope of the project was soon enlarged to take into consideration many significant factors relative to the behavior of these compounds, including properties, materials, admixtures, preparation and handling of the compounds in the field.

It was early determined that a mixture of latex, lime and road oil of the SC-4 type A.A.I. (1940 designation) gave the best results. In this mix prevulcanized, specially stabilized latex of 58.8 per cent concentration of solids produced a very tough, elastic sealer, which set up rapidly. The workability of this mix was improved by adding paraffin and varying the percentage of lime. Paraffin also increased the toughness of the set mix, but it tended to lower adhesion and increase "graininess." Its use, therefore, was discontinued. Little success was obtained with powdered rubbers, probably because of their relatively larger particle size and the consequent slowness with which these materials disperse in road oil. Adhesion and ductility were gravely impaired by the addition of such inert fillers as bentonite (colloidal aluminum silicate), fine sawdust, diatomaceous silica, Fuller's earth and cotton in percentages sufficiently high to appreciably promote toughness and, therefore, were not given further consideration.

The sealing compound has been found to undergo a steady, prolonged vulcanization over a period of years, resulting in the progressive hardening observed in mixes several months after pouring. Attempts to control vulcanization by developing a vulcanizing agent of optimum activity included investigations using sulfur flowers and sulfur monochloride. Results indicated organic sulfur to be more satisfactory as a vulcanizing agent than inorganic sulfur. The problem was complicated by the necessity of maintaining suitable workability for pouring.

According to the work of H. L. Fisher (11), all vulcanizing agents are either oxidizing agents or require the presence of oxidizing agents. If this view is accepted, the presence of sulfur in the oxidizing group is not necessary, for the fundamental reaction is one of oxidation-reduction. There are many substances besides sulfur capable of giving rise to such a reaction. Several were tried out, such as piperidine pentamethylene dithiocarbamate, 50 per cent aqueous sodium mercaptobenzothiazole, zinc oxide and casein.

In general, it was found that although the use of these vulcanizing agents produced laboratory mixes whose properties when set were highly desirable, the quantity of vulcanizing material required was so critical that a variation of about 1 per cent would determine the difference between excellent workability and no workability.

The relative failure of sponge rubber to extrude under pressure, its comparative toughness when well vulcanized, and the economic feasibility of using 50 per cent air in the mix prompted an investigation of the possibility of producing voids in the sealing compound, and of the influence of such voids upon its properties.

The method used employed the chemical reaction which occurs between sodium hydroxide and powdered aluminum, during which molecular hydrogen gas is liberated:

$$2\,Al + 6\,NaOH \rightarrow 2\,Na_3AlO_3 + 3\,H_2$$

It was found that a mix with delayed volume expansion and satisfactory workability could be produced, but that here again the quantities of materials used were too critical for operations in the field. The properties of the resulting compound were too susceptible to very slight alterations in the technique of manufacture.

A satisfactory joint-sealing compound was finally developed in which prevulcanized latex was substituted for normal latex. This material adhered satisfactorily to concrete, remained ductile and displayed low susceptibility over a wide range of temperatures. It possessed satisfactory workability for pouring. Specifications for this seal are given at the end of the text.

The mixing and application of the compounded joint material in the field presented a trying problem. Various mechanical devices were tried with the object of mixing and pouring the compound in one or two operations. Any such method was found unsatisfactory, because the rate of set could not be effectively controlled. Finally, it was found that the asphaltic oil and latex could be satisfactorily combined in small batches (3 gal.) by either mixing by hand or by a small mechanical agitator. After proper blending of the materials the compound could then be readily transferred to a standard pouring pot for subsequent filling of the joint opening.

FIELD PROJECTS SEALED WITH ASPHALT-LATEX COMPOUND

By 1940, sufficient experimental work had been done by the Department to warrant the use of the asphalt-latex sealing compound on regular concrete pavement projects. The task of mixing the compound and sealing the joints became the responsibility of the contractor and his personnel. Expansion joints containing one-inch bituminous premolded fiber boards were sealed on two complete projects with asphalt-latex joint-sealing compound before the supply of rubber latex was stopped by the War Production Board. These projects include the design section of the Michigan Test Road, projects 18-20 C3 and 67-37 C4 on M-115 between US 10 and M 66 northwest of Farwell, Michigan, and the Grand Rapids-East Belt construction project F41-34 C6 on US 131 by-pass south of US 16.

Michigan Test Road

The joints on the Michigan Test Road were sealed in the summer and fall of 1940, using sealing compounds made with both normal latex and prevulcanized latex for comparative study. The mixing equipment and method of pouring the joints are illustrated in Fig. 1a and 1b. Preparation of the joints prior to pouring was done in accordance with accepted practice for any type of joint-sealing material. In general, the operation was done in the following manner:

The asphaltic oil was heated to approximately 90 C. in tar kettle (A), Fig. 1a. To mix a batch of asphalt-latex material, a quantity of hot asphaltic oil was drained into mixing drum (B) and lime added and mixed into the asphaltic oil. The correct amount of latex was allowed to flow slowly from container (C) into container (B) while constantly stirring to thoroughly blend the ingredients. The finished mixture was then transferred from (B) into pouring can for subsequent sealing of the joints as illustrated in Fig. 1b.

Compounds made with both normal and prevulcanized latex have given excellent service during the first five years without any maintenance whatsoever. However, present conditions indicate that the

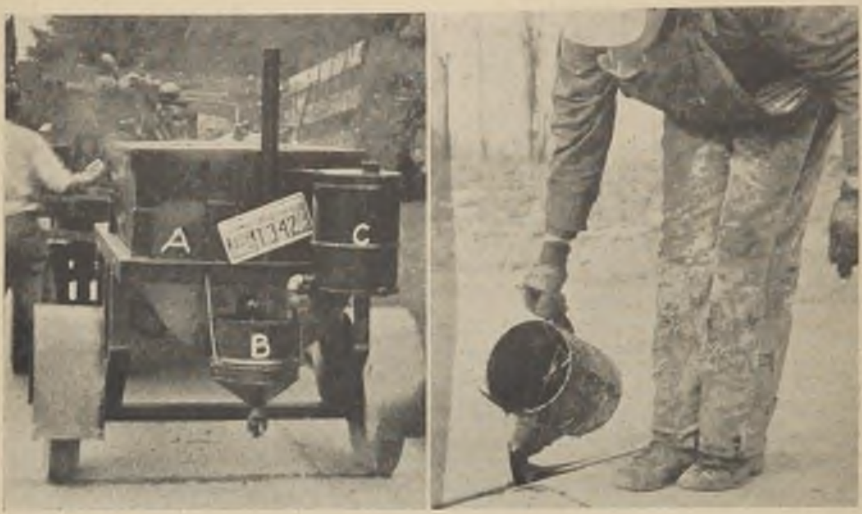

Fig. 1a. (left)—Equipment used in preparing asphalt-latex joint sealing compound.
A—Standard tar kettle; B—5 gal. mixing drum; C—5 gal. can to hold latex.

Fig. 1b. (right)—Pouring asphalt-latex joint sealing compound. (Note rubbery consistency of the material.)

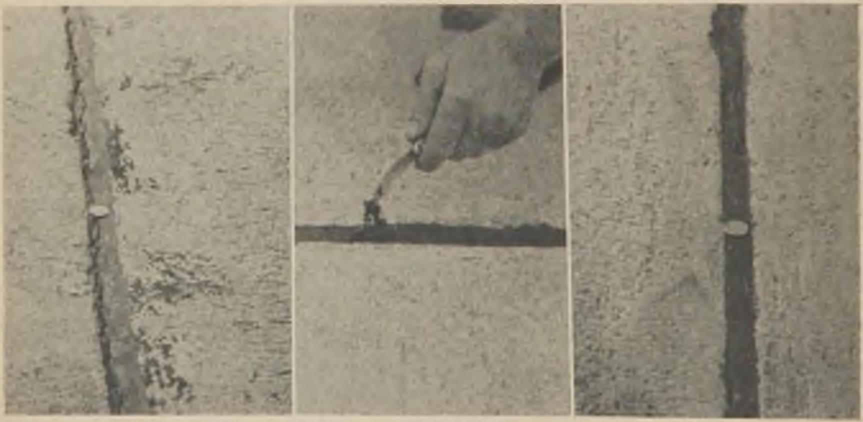

Fig. 2a. (left)—Joint seal containing normal latex after 4½ years in service. Note partial failure in cohesion which appeared after 3 years in service. This phenomenon takes place in cold weather and disappears during summer months. Photograph taken March 21, 1945.

Fig. 2b. (center)—Joint seal material containing normal latex after 4½ years in service. Note consistency of material during warm weather. Photograph taken August 15, 1944.

Fig. 2c. (right)—Joint seal containing pre-vulcanized latex. Note that this material does not fail in cohesion during cold weather and possesses better durability qualities than those of the joint seal with unvulcanized latex. Photograph taken March 21, 1945.

Fig. 3a. (left)—Joint seal containing pre-vulcanized latex. Joint sealed October, 1941. Photograph taken October 19, 1944.

Fig. 3b. (center)—Joint seal containing pre-vulcanized latex. Note unusual width of joint and perfect bond of joint seal material to concrete. Same joint as illustrated in "a". Photograph taken October 19, 1944.

Fig. 3c. (right)—Plasticity of the joint seal material illustrated in "a" after 3 years in service. Temperature 50 F. Photograph taken October 19, 1944.

prevulcanized latex produces a more durable compound. Fig. 2a, 2b and 2c illustrate the general condition of the material in the spring of 1944 and 1945. The compounds have retained their excellent bonding and plastic characteristics. It has been observed that at some points the joint seal has cracked or has lost some of its cohesive properties during the winter season although the material has remained plastic. In the summer these cracks disappear and the material appears to have considerable life. This phenomenon is more common in the compound made with normal latex and evidently this is the first stage in the ultimate failure of the joint-sealing compound. It may be possible to correct this weakness where it first appears by an application of some appropriate treatment. This is being studied.

Grand Rapids—East Belt

This project was constructed the following year, 1941, by a different contractor, but the joints were sealed in the manner described above, except that prevulcanized latex was used in preparing the seals. The general condition of the joints after 3 years is illustrated in Fig. 3a, 3b and 3c.

Quantity and cost per joint

In the case of a normal expansion joint of 1-inch opening, the concrete slab being 22 feet wide and the depth of sealer being $\frac{3}{4}$ inch, the total volume of sealer used in each joint would be 198 cu. in. This would be equivalent to 0.857 gal. To allow for waste, a quantity of 1 gal. of compound per joint seems to be a reasonable amount for estimate purposes.

Based on 1940 prices of materials in large quantities (50-gal. drums for road oil, 10-gal. drums for latex and 50-lb. lots for lime), a specific gravity of 0.981 for road oil, a specific gravity of 0.949 for prevulcanized latex, and a specific gravity of 0.97 for the final product, the following material costs can be stated for the oil-latex joint sealer:

(a) Per lb $0.0842
(b) Per gal $0.68
(c) Per joint $0.68

SEALING OF JOINTS WITH ASPHALT-LATEX COMPOUND

Experience in Michigan has shown that asphalt-latex joint-sealing compounds made and applied in accordance with the following practices are the most satisfactory.

Preparation of the joint

As stated in the 1942 specifications of the Michigan State Highway Department, the tops of expansion joints and all edged joints must be sealed as soon as the curing agent is removed and before any traffic is permitted on the pavement. Joint openings must be thoroughly cleaned, all contact faces wire brushed, and surfaces must be dry when the seal is poured. Any membrane curing material remaining after wire brushing would be almost certain to afford a good bond for the asphalt-latex compound.

Preparation of the asphalt-latex compound

A tank of at least 25 gal. capacity must be provided for heating the oil (Fig. 1a). The mixing tank should be of 5 to 10 gal. and a stirring device must be used which will produce a homogeneous mixture of uniform consistency.

The following steps are followed in compounding:

a. Heat oil to temperature of between 85 and 95 C.
b. Measure correct quantity of lime.
c. Measure correct quantity of oil.
d. Measure correct quantity of latex.
e. Pour correct quantity of hot oil into mixing tank.
f. Add correct quantity of lime to oil in mixing tank.
g. Stir lime into oil rapidly until thoroughly dispersed.
h. Continue to stir while adding approximately one-half of the correct quantity of latex to the mixing tank. Continue to stir until thoroughly blended.
i. Continue stirring while adding balance of latex until mix is homogeneous and of desirable thickness.

No more compound should be made than can be poured before it becomes too thick for handling. Stirring can be done by hand or mechanically. A 1-gal. container with a handle on one side and a spout on the other has been found suitable for pouring. In general, it may be said that 5 gal. of compound may be prepared at one time and still remain sufficiently workable for the operator to pour the entire amount. Compound or oil can be removed from equipment with kerosene or other organic solvent. Traces of latex can be flushed out with water when wet or removed by mechanical stripping when dry.

LABORATORY TESTS

Certain tentative laboratory tests have been developed by the Michigan State Highway Department for evaluating and comparing various properties of joint seals (10). The properties studied for which tests were devised included ductility (cohesion), adhesion and flow properties (viscosity).

Ductility and adhesion

A standard asphalt ductility machine was employed, using specially constructed steel forms. (Fig. 4a). These forms functioned to grip regular mortar briquettes which were molded with a brass strip imbedded through their centers in such a way that they left the molds in symmetrical halves, each half presenting a working surface of one square inch cross section.

Preparation of the Briquettes. A cement mortar consisting of two parts natural sand to one part portland cement with sufficient water for a troweling consistency is prepared. This mixture is placed in gang molds of the type used in making A.S.T.M. tensile strength briquettes. A metal brass divider is placed in the center of the mold so as to divide the briquette into two symmetrical halves of one square inch working

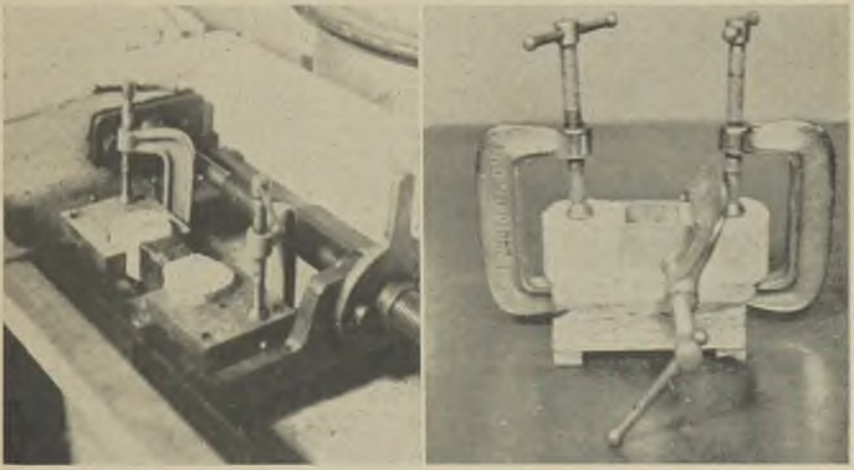

Fig. 4.—Ductility and Adhesion Test: a. (left)—Specimen in place ready for test; b. (right)—Mold assembled prior to pouring in joint seal.

Fig. 5a. (left)—Viscometer used to study flow properties of joint sealer.
Fig. 5b. (center)—Flow cells used with viscometer.
Fig. 5c. (right)—Flow cell assembled and ready for determining viscosity of joint seal.

surface area each (Fig. 4b). Specimens not having perpendicular faces are rejected.

The briquettes are cured in the moist room for one week, then allowed to air dry before use. The faces are ground on a glass plate prior to use with Grade F Carborundum powder to remove irregularities and laitance material after which they are washed and surface dried.

Preparation of the Sealer. A mercury amalgamated brass mold consisting of two side pieces, bottom plate, and three "C" clamps is attached to opposite briquette halves so that a 1 cu. in. space between the faces can be filled with the sealing compound to be tested. (Fig. 4b). The compound is heated to its application temperature, stirred and poured into the 1 cu. in. space a little higher than the upper surface of the briquette.

Method of Conducting the Test. After the sealing compound has been poured, the specimens are allowed to stand over night and excess compound over 1 cu. in. is trimmed off with a hot knife. The clamps are removed and the side pieces and bottom plate are disassembled. The specimens are placed in a refrigerator to prevent deformation.

A standard ductility machine is adapted for operation at the rate of 1 in. per hr. by use of necessary speed reductions. The glycerine bath used for low-temperature measurements is maintained at 0 F. by suitable additions of crushed dry ice and adequate stirring. Specimens are placed in the bath a half hour before testing. Fig. 4a shows the specimen in place and ready to be tested.

The distance the specimen can be drawn in inches before failure in bond (adhesion) or in ductility (cohesion) is recorded, and the type of failure is noted.

Viscosity

The constant temperature bath of a Kopper's viscometer was used in developing a test for the rheological properties of joint-sealing compounds, as shown in Fig. 5a. Special flow cells were designed (Fig. 5b) embodying the use of semi-ball ground-glass joints and clamps, which were furnished by the Scientific Glass Company. The test was based on a method proposed by Bingham and Stephens (14), making use of the formula of Pitman and Troxler (15):

$$\eta = \frac{PR^2t}{4\,h\,L}$$

where η = viscosity in poises
P = pressure applied in dynes per cm.^{2}
L = length of column of seal, cm.
t = time of flow, sec.
h = length of extrusion in cm. in time t
R = radius of tube in cm.

Viscosities were plotted on the standard A.S.T.M. chart, on which the double log of viscosity versus the log of the absolute temperature is a straight line function. The slope of this line is called the susceptibility of the material, and represents the degree to which its viscosity is affected by temperature. This value is also referred to in the literature as viscosity index.

Pouring of Sealing Compound into Flow Cell. The compound is heated to application temperature, melted, mixed to homogeneity and poured into the flow cell (Fig. 5b). The specimen is allowed to stand over night.

Assembly of Flow Cell. The cylinder is removed from the amalgamated plate, the unit is assembled in a horizontal position as in Fig. 5c, and special clamps are placed in position to hold the glass joints in place. Millimeter scales are placed at the rear and front of the tube.

Test Procedure. The flow cell unit and scales are placed in the constant temperature bath and left for 30 minutes. The vacuum hose is attached to one end of the cell, the other end being left open to the atmosphere unless more pressure is required to obtain flow. The time of flow from the ½ cm. mark to the 1 cm. mark is recorded, as well as the pressure indicated on the manometer.

Practical application of laboratory tests

The foregoing laboratory tests were conducted on joint-sealing materials of the following types:

Types 1 and 6. Proprietary brands of asphalt-rubber joint-sealing compound of the hot-pour type.

Type 2. Commercial joint seal of the asphalt type.

Type 3. Experimental asphalt-rubber compound not commercially available.

Type 4. Seal made of 30 per cent prevulcanized latex and 70 per cent SC-6 asphaltic oil.

Type 5. Seal made of 30 per cent normal latex and 70 per cent SC-6 asphaltic oil.

The above seals were so made as to be free of extraneous water, coal tar and other tar products, pressure still residua, pitches, or other products of decomposition. Manufacturing specifications for type 2 were such as to preclude heating the bituminous material above 700 F. or exposing it to pressures appreciably above atmospheric, and to assure that the finished product possess characteristics within the following limits:

1. Specific gravity at 77 F.	1.22 to 1.30
2. Flash point, Cleveland open cup	Not less than 350 F.
3. Melting point, ring and ball	122 F to 135 F.
4. Penetration at 77 F., 100 gm., 5 sec.	40 to 50
5. Ductility at 77 F.	Not less than 30 cm.
6. Loss at 325 F., 50 gm., 5 hrs.	Not more than 3.0 per cent
(a) Penetration at 77 F., 100 gm., 5 sec.	Not less than 20
7. (a) Bitumen sol. in CS_2	62 to 75 per cent
(b) Ash on ignition	20 to 32 per cent
(c) Difference between 100 per cent and sum of (a) + (b)	Not less than 4 per cent

Results of the adhesion and cohesion (ductility) studies on the above materials are shown graphically in Fig. 6. All specimens were drawn in the ductility bath at a rate of 1 in. per hr. At this low rate of drawing, all specimens gave good results at room temperature. As a matter of interest and for comparison a 24 penetration asphalt was included in this group of tests. This material failed at a distance of 0.125 in. In Fig. 6, distance drawn is plotted against temperature of the bath, the lowest temperature being 0 F. At 0 F., seals No. 4 and 5 went to the limit of the test (2 in.), whereas seal No. 1 went 0.625 in. before failure and both No. 2 and No. 3 failed in adhesion and cohesion at 0.125 in.

In carrying out adhesion and cohesion studies, it was found virtually impossible to get good adhesion to the mortar briquettes when the latter were cold and damp. This fact suggests the significance of cold, damp weather during joint sealing operations and the practical importance of developing a means of eliminating or minimizing its deleterious effect on adhesion of joint-sealing materials to the slab. No such means has yet

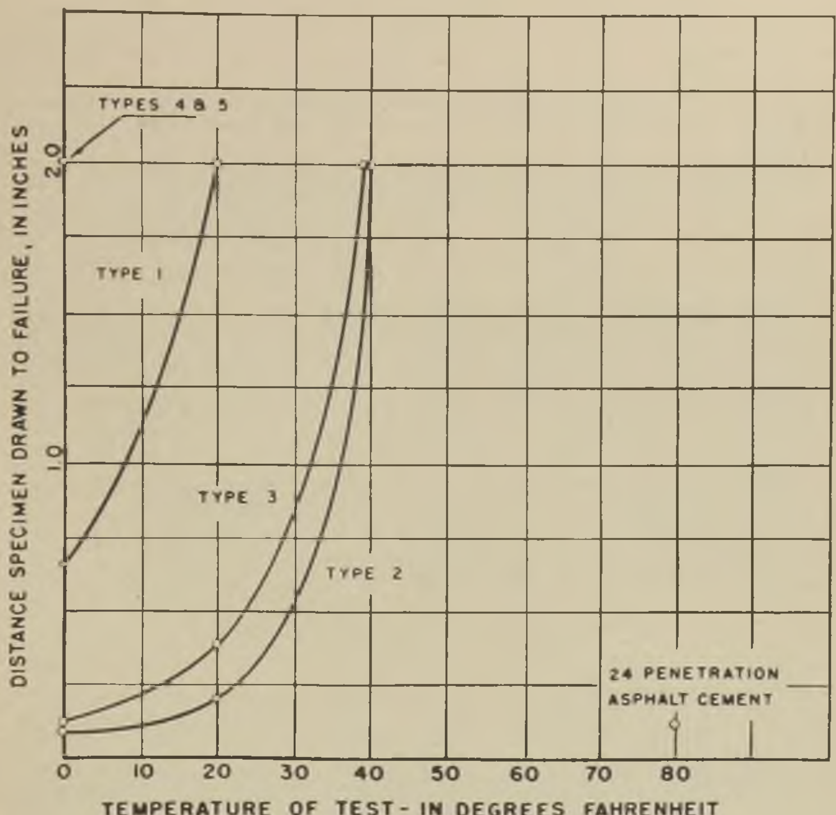

Fig. 6.—Effect of temperature on adhesion and cohesion.

been found satisfactory other than straightforward cleaning and drying of the joint surfaces.

Flow rate properties of the five types of joint seal as measured by the apparatus described above are plotted in Fig. 7.

CURRENT SPECIFICATIONS FOR ASPHALT-LATEX JOINT SEAL

The following additional specifications are given by the Michigan State Highway Department:

The vulcanized latex shall be an aqueous dispersion of rubber particles in an ammoniacal solution, having a rubber solids content not less than 58 per cent. It shall be kept in sealed containers to prevent evaporation of the emulsifying agent. The vulcanized latex shall be protected from temperatures less than 40 F. and greater than 100 F.

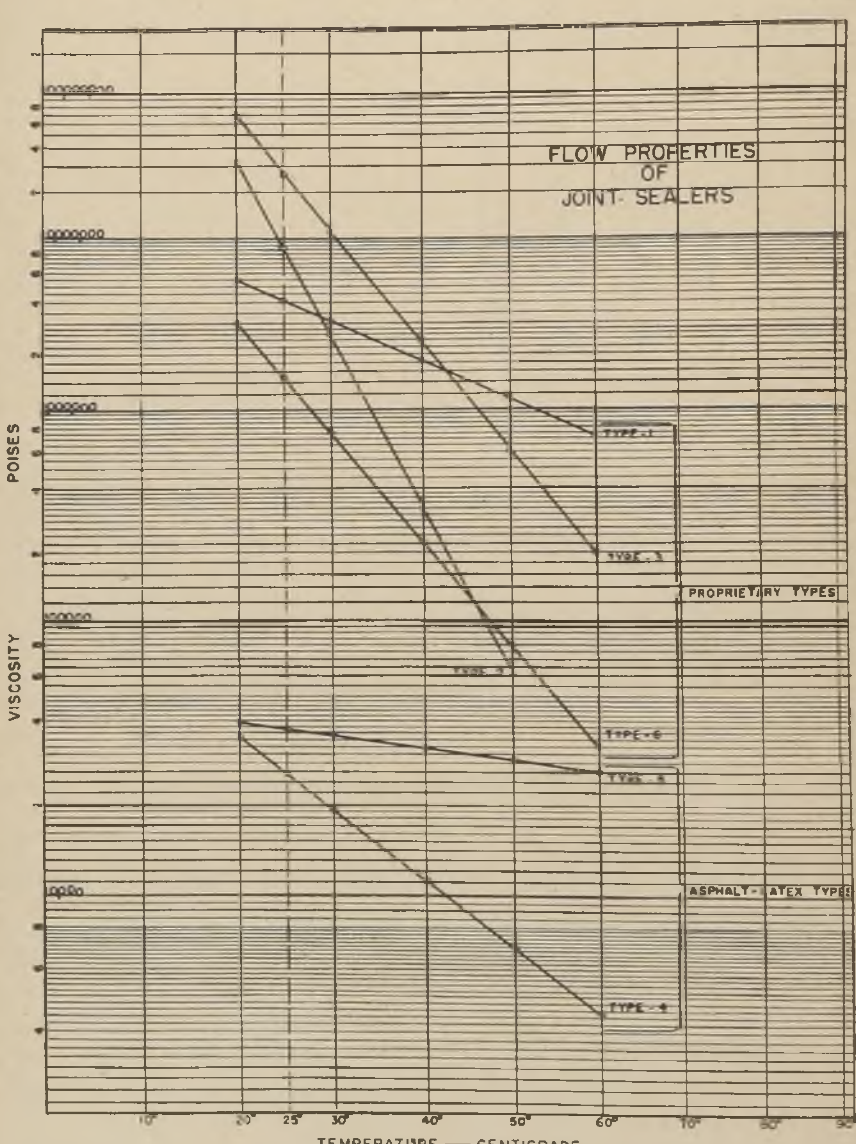

Fig. 7.—Flow properties of joint sealers.

The asphaltic oil shall be homogeneous, shall not foam when heated to the required temperature and shall conform to the following requirements:

	Minimum	Maximum
Specific gravity, 25/25 C.		1.0
Water, per cent by volume		0.5
Flash point, Cleveland open cup, deg. C.	120	
Loss on heating at 163 C., 50 gm., 5 hr., per cent		6
Viscosity, Saybolt Furol, at 60 C.	350	600
Solubility in carbon tetrachloride, per cent	99.5	
Oliensis spot test	Neg.	
Residue of 100 penetration:		
Per cent residue	70	
Ductility, 25 C., cm.	100	
Ductility, 4 C., cm.	7	
Sulfur content, per cent	2	4

Hydrated lime shall conform to the requirements for Mason's hydrated lime in the 1942 specifications for Hydrated Lime for Structural Purposes, A.S.T.M. Designation C 6.

The proportions by weight of the mixture shall be:

Lime, $Ca(OH)_2$	2 parts
Vulcanized Latex	30 parts
Asphaltic Oil	70 parts

Compounds made in accordance with the above procedures have given satisfactory performance in Michigan over a period of 4 years.

ACKNOWLEDGMENTS

The writer is indebted to Messrs. George A. Mansfield and Thaddeus Wolczynski, formerly of the Michigan State Highway Research Laboratory staff, for their early researches on important fundamental and practical aspects of asphalt-latex joint-sealing compounds.

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Title 42-23 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Petrography of Concrete Aggregate*

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Members American Concrete Institute

SYNOPSIS

Serviceability of a concrete aggregate depends upon the manner in which it joins with cement to determine the quality of concrete. Yet, standard acceptance tests do not measure properties which are directly responsible for performance of aggregates enclosed in concrete; new methods of aggregate investigation are needed. Experience shows that petrographic study can supply valuable information on a routine basis, and that, wherever possible, ordinary acceptance tests should be supplemented by examination by a petrographer familar also with problems of concrete. The significant properties of aggregates are discussed, and methods of petrographic study of aggregates are described.

An extensive bibliography is appended and referenced in the text for the benefit of readers, especially petrographers, wishing to explore further the concepts treated only briefly in this paper.

INTRODUCTION

Serviceability of concrete aggregates depends upon the strength and durability of the union which they establish with portland cement. Standard acceptance tests, currently applied independently to aggregate and to cement, do not permit satisfactory prediction of the quality of concrete. Such tests of aggregates may determine properties which make "good" or "poor" rock, but they contribute little to an interpretation of the rock's concrete-making properties. For example, tests on crushed quartz reveal how strong, impermeable, and durable it is, but they fail to indicate that it may nevertheless be an inferior aggregate. Durability or lack of durability, strength or weakness, or any other property of aggregate should concern the concrete engineer only to the extent that the property affects concrete. At this late date, service histories remain the only wholly reliable criteria of suitability. Conse-

*Received by the Institute, Jan. 29, 1946.
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quently, a reorientation of acceptance testing is necessary; practicable tests must be designed which will determine the properties affecting quality of concrete. These properties are reviewed in this paper.

Properties of aggregates comprise 1) those of the individual particles, and 2) those characterizing the entire assemblage. The latter, which comprise gradation, moisture content, and bulk weight are easily determined and will not be considered here. Properties of individual aggregate particles originate in their mineralogic compositions, textures, and structures. These are petrographic properties, and they may be determined by petrographic methods. Petrography has long been used in aggregate selection(1-10)* but its use should be extended.

PETROGRAPHIC PROPERTIES OF AGGREGATE

Properties of aggregate are numerous and complex, and they originate in several ways. The original nature of the rock may control its concrete-making qualities. Thus, rocks containing gypsum are always somewhat soluble, opaline materials react chemically with alkaline solutions, and shales are usually soft and absorptive. Conversely, many properties are not directly related to the rock type but were induced by weathering, solution, saturation, or encrustation by ground waters, by fracturing and jointing related to the geologic history of the region, or by some other natural alteration. Thus, granites may be either hard or disintegrated, and sandstones may be either friable or tightly cemented, with resulting variations in strength and porosity. Therefore, petrographic study of concrete aggregates must be dual: 1) properties inherent to the rock must be interpreted in relation to 2) the natural alteration which it may have experienced.

The most important properties of concrete aggregate are discussed below. It must be noted that these properties are commonly independent of rock type since rocks of any given type may have widely different concrete-making qualities. Effects of secondary alterations, of which weathering is the most common, are important only insofar as the essential properties of the aggregate are changed; where significant, the possible effects of weathering are indicated.

Chemical reactivity

Few, if any, rocks or minerals are chemically inert while enclosed in portland cement. There are four main ways that an aggregate may be chemically deleterious. First, it may contain water-soluble constituents which can be leached, with attendant loss of strength and increase in porosity. Leached material may form unsightly efflorescence or indirectly lead to surface scaling(11, p. 56). Second, soluble constituents or products of oxidation may retard or modify normal hydration of cement.

*Numbers in parentheses refer to bibliography at end of text.

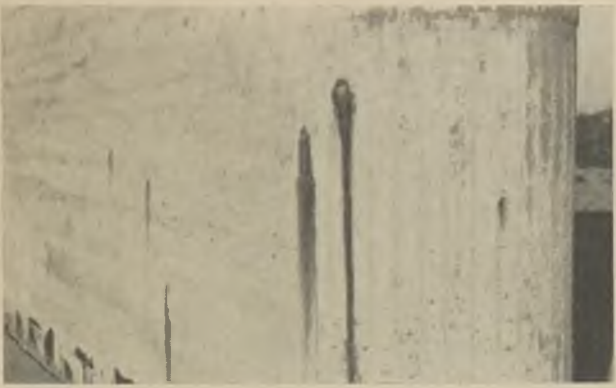

Fig. 1—Pop-outs and staining of concrete produced by oxidation of pyrite in particles of coarse aggregate.

Thus, oxidation of pyrite (FeS_2) in the aggregate resulted in pop-outs, local disintegration, and unsightly staining of the concrete at a dam in California[12] (Fig. 1).* Similar effects have been described by Litehiser[13]. Also in this category is the deleterious action of the mineral alunite ($K_2Al_6(OH)_{12}$ $(SO_4)_4$) in mortar bars[14]. In one month the bars expanded significantly, cracked, and warped, probably as the result of sulfate attack on the cement, since secondary calcium sulfoaluminate was abundantly produced. Similar effects might be expected from other aggregates containing soluble sulfates[15]. Gypsum ($CaSO_4.2H_2O$) in excessive amounts is deleterious to concrete[16, pp. 188, 194].

Third, certain aggregates are attacked by high-alkali cements[9, 17]. The reactions lead to internal expansion, external cracking, and decreased elasticity and strength (Fig. 2 and 3). Rocks and minerals known to be susceptible to this kind of reaction are: opal; many volcanic rocks of acid to intermediate composition (high to medium silica content); glass (artificial or natural, excluding the basic types such as basaltic glass); chalcedonic rocks (including many cherts and flints); some phyllites; and tridymite[9,10,12,14,17,18,19,20,21,22]. Ameliorants are being sought but at present, where such rocks must be used, specification of low-alkali cement is imperative. Limitation of alkali content to below 0.6 per cent (Na_20 + K_2O, expressed as soda-equivalents) has avoided or significantly retarded deterioration, even though the aggregates are highly reactive[9,20,24].

Fourth, certain common minerals (e.g., zeolites) are capable of base-exchange. In this process, alkalies in these minerals are exchanged for calcium in solutions permeating the concrete[9]. Released alkalies may then attack susceptible aggregate particles, and efflorescence may form on or near the surface of the concrete. Studies by the Bureau of

*However, pyrite in aggregate is not always susceptible to such decomposition; many concrete pavements in and about Colorado Springs, Colorado, contain pyritiferous aggregates which show no signs of alteration after 19 years of service.

Reclamation and others suggest that such base exchange, involving a granodioritic aggregate, may account for efflorescence and surface scaling of concrete on the downstream face of a thin dam in southern California[12]. Tests conducted in the Bureau of Reclamation laboratories indicate that most natural aggregates will extract calcium from alkaline solutions. Rengade, Lhopitalier, and de Fontmagne[23] and, more recently, Hansen[18] have shown that some natural aggregates release Na_2O and K_2O when embedded in cement.

Porosity, permeability, and absorption

Pore characteristics of aggregate are important controls of chemical and physical stability, and they strongly influence the bond with cement. Pore characteristics significantly affect the strength of any material, and they determine absorption and permeability. As a result, they control both durability under freezing and thawing conditions, and rate of chemical alteration. Three characters of pores determine physical and chemical stability of aggregate. They are 1) absolute pore volume, 2) size of the pores, and 3) their continuity. These characters are reflections of the

Fig. 2—Typical appearance of field concrete severely affected by reaction between aggregate and high-alkali cement. The concrete is 6½ years old.

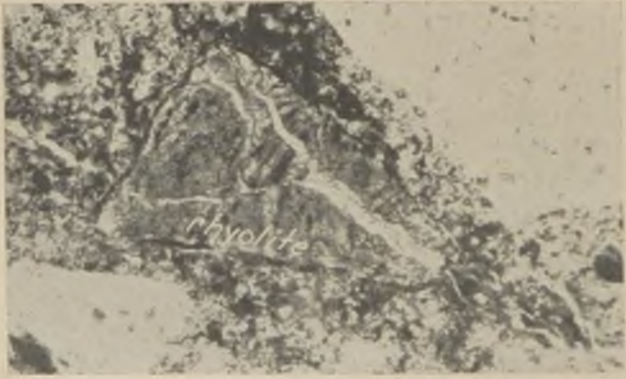

Fig. 3—Photomicrograph of concrete from Parker Dam, showing microfractures produced as a result of reaction between a particle of rhyolite and alkalies in cement.

Fig. 4—Fragment of quarried limestone in part unsound due to weathering action of ground waters. The white portion is highly absorptive, although the pores are microscopic in size.

texture of the rock, such as grain size, grain shapes, secondary cementation, and induration, and of structural features such as stratification and fractures. Original pore character may be changed by weathering, with resulting increases in porosity, permeability, and absorption; these changes may reduce durability and strength (Fig. 4). In spite of their importance these pore characters are not measured by any standard acceptance test.

Absorption is commonly used to predict freezing and thawing durability of aggregate. Investigators have variously obtained good to poor correlations between absorption and durability, the best relationships being found where similar rock types were used in the experiments. Where different kinds of rocks were used, only fair to poor correlations were derived. Wuerpel and Rexford(8), Cantrell and Campbell(25), and Sweet(26) obtained good relations between high absorptions and low durabilities of chert aggregates. But Hughes and Andersen(27), Garrity and Kriege(28), and others(29,30,31,32) did not find that durability consistently decreased as absorption of lithologically complex aggregates increased. Wuerpel(31) states that even with specifications which permit absorptions as high as 2 per cent, one-fourth of the rejected aggregates may be suitable for use in concrete.

The absorption test of bulk aggregates does not discriminate between absorption by few, highly porous particles or absorptions by many, moderately porous particles; for this reason, absorptions of lithologically complex aggregates do not show quantitative relationships to soundness. Wray and Lichtefeld[32] have demonstrated how slowly moisture is regained after desiccation, the rates of absorption varying with porosity and permeability. Thus, a standardized period of immersion may give deceptively low absorption for some porous, unsound aggregate of low permeability. Garrity and Kriege[28] long ago pointed out the divergence of absorption by simple immersion from potential absorption; one sample tested absorbed 2.60 per cent in 18 hours at 70 F but 6.47 per cent in 5 hours in boiling water, whereas a second material showed an increase from 1.55 per cent to merely 1.69 per cent under the same conditions.

Freezing and thawing breakdown is controlled by degree of saturation. On this basis, the saturation ratios of Krueger[33] and Schurecht[34], and modifications of these ratios have been widely used in durability studies of building stone[11,33], brick[34], concrete[35,36], and concrete aggregate[26,29]. Saturation ratios are calculated as the ratio between short-time and long-time absorption, and are therefore functions of permeability. For concrete aggregate these ratios are significant where rocks of similar type are tested; but where a wide range of types is studied, the ratios and rates of disintegration are little related. However, permeability is a fundamental factor in resistance to freezing and thawing[37]. Sweet[26] found a relationship between depth of penetration of a dye solution and durability of cherts.

The effect of an aggregate particle upon freezing and thawing durability of concrete depends largely upon the ability of the particle to attain a high degree of saturation while it is enclosed by cement. Measurement of the imbibition of water while the particle is immersed in a water bath (such as by absorption itself or by the Schurecht or Kreuger ratios) does not necessarily indicate the capacity of aggregate to attain and retain a high degree of saturation in concrete. Whether or not water will enter into or be drawn from an aggregate particle while it is enclosed in concrete depends in considerable part upon the relative size of the pores in the particle, on the one hand, and those in the cement paste, on the other. Water moving by capillarity will not enter aggregate containing only large voids, even if these voids are interconnected and penetrable. On the other hand, small voids will be penetrated; and if these openings are smaller than those of the cement paste, the water will be preferentially drawn into them from the paste. During periods of hydration or drying, while water is being withdrawn from the interstices of the cement paste, water will be drawn by capillarity from aggregate particles containing only voids which are larger than those in the paste; but the last residuals of water remaining in even relatively dry concrete may be concentrated in aggregate-voids smaller than those in the cement paste. Thus, rocks containing exceedingly small, interconnected voids are capable of attaining and retaining a high degree of saturation in concrete, and may be susceptible to disruption if repeated freezing occurs. Also, these materials will offer little opportunity for relief of hydrostatic pressures built up in the concrete by the progressive freezing postulated by Powers[37]. In view of the fact that most field concrete receives moisture through capillarity, these considerations are highly pertinent.

Extensive studies on building stones have shown a good relation between durability and content of voids smaller than 0.005 mm. (micropores), and that neither absolute porosity nor absorption is simply related to soundness[11, p. 33]. Moreover, rocks most commonly associated with freezing and thawing breakdown of concrete (such as argillaceous limestone, shale, and weathered chert) are characterized by exceedingly small voids. A soft, porous, and friable pumice with large voids, which was tested by the Bureau of Reclamation, showed 24.6 per cent absorption in 24 hours of immersion; yet concrete cylinders containing this pumice as both fine and coarse aggregate failed only after 350 cycles of freezing and thawing[38]. Many vesicular basalts, containing abundant large pores, have exhibited excellent durability in both field and laboratory concrete[12].

Bond and surface texture

Strength and permanence of bond between cement and aggregate are functions of surface texture and chemical stability of the aggregate. The integrity of original bond, however good, will be lost if certain chemical reactions, such as that between high-alkali cement and reactive aggregate, subsequently take place; but some types of chemical interaction may be beneficial in effecting a more intimate union. Surface rugosity of aggregate particles influences development of bond; but an even more important aspect of surface texture is the porosity, absorption, or permeability of the zone immediately underlying the surface. Penetration of the aggregate by cement slurry is conducive to good bond, but the porosity implied by very high penetrability may involve low tensile and shearing strengths of the aggregate, with net loss in strength of the concrete.

Surface texture depends upon original texture and structure of the rock, effect of weathering, and, in the case of natural gravels, upon processes of transportation and deposition which formed the deposit. The texture and structure of the rock are the most important. Certain minerals, such as quartz and feldspars, naturally break with smooth surfaces; and many common rocks, such as volcanic glasses and some cherts, do likewise because of dense structure. Effects of weathering are not altogether undesirable inasmuch as slight surficial alteration will increase absorption and rugosities of the aggregate particles, and may thus increase integrity of bond with cement.

Hughes and Andersen(27) have shown that fine aggregates with very low absorptions generally develop less strong bonds and produce less durable mortars than those with slightly higher absorptions. Meissner(39) attributed low freezing and thawing durability of concrete bars containing crushed quartz aggregate to poor bond. The interrelation of bond and absorption may account in part for poor correlation between durability of concrete and absorption, inasmuch as strength of bond increases as absorption increases whereas durability of concrete tends to decrease as absorption increases. However, absorption of aggregates, as such, cannot be adduced as a reliable indication of good bonding characteristics, for capillaries of extremely small size may not permit penetration of the cement slurry into the aggregate particles, even though they may permit considerable penetration of water. Therefore, from the standpoints of both durability and bond, penetrable voids of very small size are the least desirable.

Internal fractures

Rock may be internally fractured but nevertheless present an external appearance of soundness. Fractures are common in rock types which are naturally "brittle," such as cherts and slates, but may exist in any rock either as a result of weathering or of earth movements, such as folding or faulting, which affected the region from which the rock was originally derived. Cracks may be microscopically small, but they increase absorption and porosity, and decrease strength and durability(8). Moreover, they may become accentuated by temperature or stress changes within concrete.

Internal fractures are commonly produced or extended by crushing processes; and these changes generally result in higher losses in sodium sulfate and freezing and thawing soundness tests of the aggregate(2,12). Original fractures, not produced in crushing, may commonly be detected visually by the presence of secondary materials such as iron oxide (common in slates and cherts) or stingers of clay (common in impure limestone), in cases where no openings are otherwise apparent. Although fractures may be tightly sealed through metamorphic processes taking place in the earth under conditions of high pressure and temperature, secondary materials deposited during weathering or related alterations do not prevent entrance of water into the rock.

Particle shape

High proportions of flat or elongated particles in concrete aggregate are objectionable since they decrease workability and hence necessitate the use of more sand, cement, and water(24, p. 63). Also, they pack poorly, reducing bulk weight[40] and decreasing compressive strength[41]. Moreover, flat particles tend to orient themselves horizontally in concrete, permitting the accumulation of bled water beneath them, which prevents the development of good bond on their lower surfaces[42].

Internal structure largely controls particle shape. Thus, the laminated structure of schists, slates, and shales commonly gives rise to slabby forms. Characteristically, rocks without planar structures such as massive varieties of granite or limestone break down to roughly equidimensional shapes.

Thermal expansivity

Two important aspects of thermal expansivity pertinent to the selection of concrete aggregate are: 1) the average coefficient for the entire aggregate and 2) divergence of the coefficients of the aggregate particles from that of the cement.

In designing structural elements, allowance frequently must be made for thermal volume changes of the concrete, and these volume changes are strongly influenced by the average expansivity of the aggregate[24].

Differences between the thermal expansivities of aggregate constituents and that of the cement have been held responsible for poor durability of concrete in service[43]. Aggregate particles with unusually low coefficients are of more importance than particles with exceptionally high coefficients, because few rocks have coefficients significantly greater than those of portland cements. With declining temperature, particles of very low expansivity may create high tensile stresses in the enclosing cement, and disruption of the concrete can result[43].

Although expansivity of aggregate affects durability of concrete, the significance of different expansivities of different components of an aggregate is difficult, if not impossible, to evaluate in most instances. Measurement of the coefficient of expansion of an aggregate or of its constituents is not practicable on a routine basis. Although petrographic identification of constituents will assist in estimation of expansivity, similar rocks may have widely different coefficients (Table 1); in addition, many rocks and minerals have different coefficients in different directions[39,44,45,46]. Moreover, the effects of differences between the expansivities of aggregate and cement, unless very large, are commonly obscured by other variations (for example, aggregates with different coefficients of expansion usually also have different chemical compositions, surface textures, pore characteristics, etc.). For structural reasons, concrete possessing a low over-all coefficient of expansion may in specific instances be so desirable as to outweigh the advantages of matching the coefficients of the aggregate constituents with that of the cement.

TABLE 1—THERMAL COEFFICIENTS OF EXPANSION OF ROCKS*, CONCRETES, AND PORTLAND CEMENTS

(Expansion per degree Fahrenheit, in range—4 to 212 F)

Materials	Number of Specimens	Observed range in Mean Linear Thermal Expansion Coefficient $\Delta L x 10^6 L \Delta T$	References
Granites and Rhyolites	27	1.0 to 6.6	45,46,49,51
Diorites and Andesites	17	2.3 to 5.7	39,45,46,49
Gabbros, Basalts, Diabases	15	2.0 to 5.4	45,46,49,51,52
Sandstones	24	2.4 to 7.7	45,46,49,50,52
Quartzites	20	3.9 to 7.3	45,46,49,51,52
Dolomites	7	3.7 to 4.8	39,45,46
Limestones	65	0.5 to 6.8	39,45,46,49,50,51
Siliceous Limestones	6	2.0 to 5.5	45,46
Cherts	49	4.1 to 7.3	45,46,49,51,52
Marbles	29	0.6 to 8.9	43,47,48,49
Slates and Argillites	5	4.5 to 4.8	45,46,49
Portland Cements, neat	10	5.9 to 9.0	24,51
Concretes	27	3.6 to 6.8	24

*All coefficients of rocks were obtained on dry specimens.

Surface coatings

Surface coatings are generally detrimental because: 1) the strength of bond between the rock and coating is generally less than the internal strength of the rock, and may be very low, so that the union between the rock and the cement is weakened(15); 2) many natural coatings contain substances susceptible to reaction with alkalies in cement (for example, opaline silica); thus, aggregates which are inherently innocuous may become deleterious because of the surface coating(9); 3) coatings may contain salts of sodium or potassium which, if released, may participate in deleterious chemical reactions with susceptible constituents of the aggregate(9), or coatings of soluble salts such as gypsum ($CaSO_4.2H_2O$) may influence the course of cement hydration(16); and 4) if coatings are broken free during handling, and the residue not subsequently removed by washing, water requirement may be increased(53). In some instances, coatings may be essentially inert and so strongly bonded as to be innocuous, or they may actually increase the strength of the bond between aggregate and cement.

Properties affecting drying shrinkage

Carlson(54) has suggested that harsh aggregates, by increasing water requirement for given workability, may contribute to drying shrinkage, and that this shrinkage will be increased by particles which swell and contract upon wetting and drying. He tentatively concluded "that compressibility is the most important single property of aggregate governing concrete shrinkage," shrinkage increasing with increasing compressibility. He suggests that low shrinkage induced by more rigid aggregates probably results in a concrete more susceptible to breakdown by freezing and thawing through the formation of minute cracks in the cement paste.

Several aggregates containing high proportions of very coarse-grained, relatively unaltered granites have been found to produce concrete with low flexural strength and low resistance to breakdown under freezing and thawing, heating and cooling, and wetting and drying conditions [12,55]. These properties may result either from poor initial bond of the cement to the smooth, nonpermeable rock surfaces, from rupture of original bond due to differences in thermal coefficients among the several minerals of the granite or between the granite and the cement matrix, or, as may be implied by Carlson's experiments[54], from the formation of abundant minute cracks in the cement paste adjacent to the granite particles which have low compressibility.

Strength and elasticity

Although unusually soft aggregate particles reduce strength and elasticity of concrete, even under circumstances not involving durability, most rocks have considerably greater strengths and moduli of elasticity than even highest quality concrete. For example, average crushing strengths of many granitic rocks, gneisses, quartzites, marbles, sandstones, limestones, slates, and serpentines range from 10,500 to 28,700 pounds per square inch[44], whereas ultimate compressive strengths on the order of 5,000 to 8,000 pounds per square inch are considered quite satisfactory for most concretes. The strength and elasticity of sound aggregate are rarely factors controlling suitability. The strength and elasticity of concrete, containing sound aggregate and not affected by cement-aggregate reaction, are controlled largely by the bond established between cement and aggregate, the quality of cement, mix design, and placing and curing techniques.

Specific gravity

Specific gravity of aggregate influences the unit weight of concrete, and thus is of first importance under some conditions of service, such as in gravity sections of dams where maximum weights of concrete may be desired, or in flooring, where minimum weights are necessary[24]. In these circumstances specific gravity may be a determining factor in the selection of an aggregate. However, in general, the specific gravity of aggregate, other than certain artificial types, ranges between comparatively narrow limits, and the unit weight of concrete is not critical for most purposes. Of local importance are extremely light, unsound particles which segregate toward the surface, and thus contribute to low abrasion resistance and surface durability of the concrete[56].

Specific gravity determinations on aggregates are commonly utilized as an indirect measure of soundness, the conception being that soundness will decrease as the specific gravity decreases. Specific gravity is a function of the composition of the rock and its porosity; thus a rock with high absolute specific gravity will have a considerably higher porosity at a given apparent specific gravity than a rock with a low absolute specific gravity. For example, with an apparent specific gravity of 2.40, pure chert or quartzite would have a porosity of 9.4 per cent; whereas, with the same specific gravity pure dolomite would have a porosity of 14 to 17 per cent. Fear of low specific gravity is based upon fear of high porosity. Thus, the discussion in the section dealing with porosity, permeability, and absorption is pertinent in this connection.

The relationship between specific gravity and durability has commonly been considered in the selection of chert aggregates. Wuerpel and Rexford(8) observed that all chert fragments related to pop-outs in field concretes possessed apparent specific gravities (A.S.T.M. designation: bulk specific gravity, saturated-surface-dry) less than 2.40. On this basis and in view of experimental data, these authors recommend that cherts with less than this specific gravity be considered unsound. Cantrill and Campbell(25) have found that chert gravels from the Tennessee and Cumberland Rivers with apparent specific gravities (A.S. T.M. designation: bulk specific gravity) less than 2.33 and 2.44, respectively, are unsound in service. Sweet(26) concludes that unsound Indiana cherts are characterized by specific gravities (A.S.T.M. designation: bulk specific gravity, saturated-surface-dry) less than 2.50; but he mentions an unsound Ohio chert with an apparent specific gravity of 2.51, thus emphasizing the necessity for restricting a given specification to aggregates of one specific lithologic variety. Wuerpel and Rexford(8) point out that even if restricted to chert aggregates, their specification limit will exclude only 70 to 90 per cent of the unsound materials, largely because of the presence of impurities with high specific gravities. Walker(57) records the occurrence of 9.1 to 56.0 per cent by weight of chert particles with specific gravities less than 2.40 in four gravels supplied by long-established commercial producers. Lang and Hughes(58) in tests of aggregates of different lithologic types found no correlation between freezing and thawing durability and specific gravity.

Specific heat and thermal conductivity

The Bureau of Reclamation(59) has shown that thermal conductivity and specific heat of aggregate exert a strong influence upon the transmission of heat through and changes of temperatures in concrete. Where the following rock types were used as coarse aggregate, other conditions being identical, the specific heats of the concrete increased in this order: quartzite, granite, rhyolite, limestone, basalt, dolomite. The thermal conductivities of these concretes increased in the following order of aggregates: basalt, rhyolite, granite, limestone, dolomite, quartzite.

Calculations of diffusivity based upon the specific heats, densities, and thermal conductivities indicate that the different concretes will have significantly different rates of temperature change under equivalent conditions; the rates of temperature change increasing in the following order of aggregates: basalt, rhyolite, granite, dolomite, limestone, quartzite. Rates of heat transmission and temperature change must be considered wherever external or internal sources of heat are expected to produce significant thermal changes in volume.

SELECTION OF AGGREGATES

Standard acceptance tests of aggregates yield only meager information concerning the fundamental physical properties discussed above, and they largely ignore the pertinent chemical factors. If an aggregate fails these tests, the cause of failure may still be unknown. But the test data are amenable to quantitative evaluation in terms of a certain number of cycles, or weight loss, or some other definite quantity. These figures may be used arbitrarily in defining specification limits, although it is recognized that results of any given tests of this sort cannot be reliably correlated with service records(30). Indeed, some aggregates which consistently fail the standard tests for physical soundness possess excellent service histories.

A test which will quickly reveal the reactivity of proposed aggregates with alkalies in cement is badly needed. Many tests have been suggested, but all are unsatisfactory. The degree of reactivity can be determined by measurements of expansion or strength decline of mortar bars containing high-alkali cement; but the several months commonly required for conclusive results, limit the practicality of the method (9,17,18,20,21,60). Stanton(9,60) has suggested tests which are applicable to aggregates containing highly reactive opal, but they are inconclusive for less reactive materials. Leach tests(9,21,61,62,63) based on the chemical effects of alkaline solutions on aggregates are not yet perfected as indicators of potential cement-aggregate reaction. An etch test proposed by Parsons and Insley(64) is a modification of the leach test, involving microscopic examination of polished sections which have been treated with alkaline solutions; this test does not measure the particular kind of reactivity which causes deleterious expansions of mortars and concretes. Use of ultra-violet light and dyes for detection of cement-aggregate reaction is inadequate and such methods are unsatisfactory for predicting reactivity of aggregates(10,12,60).

Petrographic examination is primarily a visual appraisal of aggregate; any weaknesses are described as to character and extent; in a mixed aggregate any desirable or undesirable properties are identified with particular constituents; and relative qualities of different size grades are determined. The method does not lend itself to definition of specification limits because information so obtained is the result of subjective appraisal by the petrographer, and can be reduced to a numerical quantity only through his personal interpretation. On the other hand, difficulties attending the formulation of specifications based upon results of standard soundness tests have likewise been recognized[8].

In the Bureau of Reclamation procedure of petrographic analysis, aggregates are examined visually, and representative fractions are segregated into appropriate lithologic groups, based upon three considerations: 1) petrographic identity, 2) physical quality, and 3) chemical stability. For example, particles of granite may be placed in several categories depending upon the effects of varying degrees of weathering, or limestones may be classified as chemically innocuous or deleterious because of the absence or presence of an unstable minor constituent. In most instances, the study of natural gravels is satisfactory if only the 1½- to ¾-in. and ¾- to ⅜-in. fractions are analyzed in detail, and the other fractions are then qualitatively compared with them. Sands are examined in detail under binocular and petrographic microscopes. Where necessary, supplemental partial analyses, involving the segregation of physically or chemically deleterious particles, are made of any size-fraction of the aggregate. Typical summarization of a Bureau of Reclamation petrographic analysis is shown in Tables 2 and 3. These analytical data are always accompanied by a memorandum describing pertinent features of the aggregate and including specific recommendations regarding use of the aggregate.

The usual physical tests are applied to all aggregates investigated by the Bureau of Reclamation; and petrographic examination is used to supplement and interpret the results of these tests, or independently for preliminary estimates of quality. For example, in the sodium-sulfate soundness test (A.S.T.M. Designation: C 88-41T)[65], breakdown of platy particles may not be indicated by loss through the screens since, if the fracturing takes place by splitting parallel to the flat surfaces, no effective decrease in diameter is produced. This situation was encountered in tests of a phyllite aggregate proposed for use at Fontana Dam[12]. Low specific gravity of natural aggregates is commonly used as an indication of porous, soft, or deeply weathered materials. However, several natural aggregates having specific gravities of 2.44 to 2.56 from the vicinity of Provo, Utah, possess excellent service histories; these aggregates are composed predominantly of firm or hard but porous sandstones, and were quickly found by petrographic examination to contain merely insignificant proportions of clayey or friable materials,

TABLE 2—PETROGRAPHIC ANALYSIS OF A COARSE AGGREGATE

Typical tabular summary of a Bureau of Reclamation petrographic analysis of coarse aggregate.

Rock Types	Percentages by Weight			Quality	
	1½-¾ in.	¾-⅜ in.		Chemical	Physical
Granites	1.7	9.0	Slightly to moderately weathered, coarse-grained	Innocuous	Satisfactory
Rhyolite Porphyries	1.5	1.1	Slightly to moderately weathered, aphanitic, porphyritic	Innocuous	Satisfactory
Andesite Porphyries	29.5	17.2	Slightly to moderately weathered, aphanitic to fine-grained	Innocuous	Satisfactory
Weathered Andesite Porphyries	11.9	10.6	Moderately weathered, fractured, firm to soft	Innocuous	Fair
Andesites	9.8	3.9	Slightly to moderately weathered, aphanitic to glassy	Deleterious	Satisfactory
Basalts	18.1	29.3	Slightly to moderately weathered, aphanitic to glassy	Innocuous	Satisfactory
Weathered Basalts	21.9	21.8	Moderately weathered, fractured, firm to soft	Innocuous	Fair
Meta-Sandstones and Quartzites	5.6	7.1	Hard, massive to slightly schistose, fine-grained	Innocuous	Satisfactory

*See notes for Table 3.

TABLE 3—PETROGRAPHIC ANALYSIS OF A COARSE AGGREGATE

Typical tabular summary of the quality of coarse aggregate, based upon petrographic analysis.

	Degrees of Quality*	Percentages by Weight	
		1½-¾ in.	¾-⅜ in.
Chemical	Innocuous	90.2	96.1
	Deleterious (with high-alkali cements)	9.8	3.9
Physical	Good	....	....
	Satisfactory	66.2	67.6
	Fair	33.8	32.4
	Poor	....	....

*Explanation of terms used to describe the quality of aggregates.

Innocuous and deleterious

The classifications "innocuous" and "deleterious" refer to the chemical reactivity of aggregate constituents, particularly their reactivity with alkalies in cement. "Deleterious" materials produce adverse effects on concrete due to chemical reactions which take place subsequent to hardening of the cement. Substances designated as "deleterious" by reason of susceptibility to attack by alkalies in cement are non-deleterious if used with cements containing less than 0.60 per cent total alkalies (Na_2O plus K_2O, expressed as soda-equivalents). "Innocuous" constituents do not participate in chemical reactions harmful to concrete.

Good, satisfactory, fair, and poor

Four classifications are employed to describe the contribution made by the physical properties of the aggregate constituents to the quality of the concrete, as follows:

Good aggregate constituents contribute to superior strength, abrasive resistance, and durability under any climatic conditions.

Satisfactory aggregate constituents contribute to high or moderate strength, abrasive resistance, and durability of concrete under any climatic conditions.

Fair aggregate constituents contribute to moderate strength and abrasive resistance of concrete, but under rigorous climatic conditions they contribute to physical breakdown of concrete.

Poor aggregate constituents contribute to low strength and abrasive resistance of concrete under any climatic conditions, and cause physical breakdown of concrete under rigorous climatic conditions.

and so were recommended for use in spite of the fact that their specific gravities were below specification limits(12).

Periodic petrographic examination of a deposit during exploitation will disclose significant vertical or lateral variations. For example, gravel in the lower course of the San Joaquin River in the Sierra Nevada typically contains a highly reactive andesite(9); but certain deposits, or at least certain parts of some deposits, are virtually free from these andesites. The aggregate produced from these latter deposits might safely be combined with cements of moderately high alkali content. But, unless low-alkali cements are used, extreme care must be exercised in order that the limits of the andesite-free deposits are not transgressed. Again, near Pasco, Washington, many satisfactory, basaltic gravel deposits are underlain by deeply weathered, physically unsound gravels of similar composition[12]. Investigation of gravel deposits in this region must reveal the presence of these weathered zones and locate their limits; exploitation must be conducted so as to prevent admixture of the weathered materials with satisfactory aggregate.

Petrographic examination may be fruitfully employed in the search for new deposits of natural aggregate. The rapidity with which reliable examinations and comparisons can be made permits the evaluation of aggregate quality even when only limited time is available, or the inaccessibility of laboratory equipment renders impracticable the application of most standard tests.

CONCLUDING STATEMENT

Acceptance testing of concrete aggregates is unsatisfactory at the present time, since none of the standard tests yield information which properly correlates with service of concrete; the whole concept of aggregate testing needs revision and reorientation, so that these deficiences will be overcome. The first requirements are tests to evaluate 1) pore characteristics, 2) bonding characteristics (surface textures) and 3) chemical stability, particularly with regard to "cement-aggregate" reaction.

Petrographic methods represent a useful approach toward these and other objectives. The principle values of these methods are: 1) unique, fundamental information not derived from standard tests is obtained; 2) quick, preliminary estimates of quality can be made; 3) they permit interpretation and amplification of the results of other acceptance tests; 4) petrographic identification of chemically deleterious substances is the only reliable means by which chemical instability of new aggregates can be quickly estimated at the present time; 5) new aggregates may be compared with other aggregates for which service data are available; and 6) petrographic methods can be used to advantage in exploration for new sources and in exploitation of partially developed deposits.

The possibility of comparing new with previously tested or used aggregates is of great importance. Tests or service histories upon aggregates which are not completely described are of limited value, since they indicate the qualities of the one aggregate and no other. If aggregates are completely identified from a petrographic standpoint, results of tests or service are then of general interest, and can be utilized in selection of new aggregates of similar lithologic type, either in the same or in distant regions. It is unfortunate that so many aggregates are meticulously tested but yet are described in the literature as "crushed stone," "river gravel," "sand," or vaguely, as "trap," and so on.

Petrographic study of aggregates is not a panacea by which all the ills inherent to selection of concrete aggregates can be cured, but it does afford a means of evaluating significant factors not otherwise discovered. It is therefore recommended that, wherever possible, the routine testing methods be supplemented with examination by a petrographer familiar with problems of concrete.

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[To facilitate selective distribution, **separate prints** of this title (42-24) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946]

Title 42-24 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Entrained Air In Concrete

A SYMPOSIUM: contributions by:

W. A. CORDON
L. E. ANDREWS
ALEXANDER FOSTER, JR.
STANTON WALKER AND DELMAR L. BLOEM
HENRY L. KENNEDY
EDWARD W. SCRIPTURE, JR.
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FOREWORD

Entrained air in concrete provided a full-packed session at the Institute's 42nd annual convention in Buffalo in February; with one paper already published on laboratory work on the subject by the Corps of Engineers, U. S. Army* and 14 other contributions, several of them presented only in their highlights to be within the allotted 10-minute limitations of the session. These other contributions, most of them reporting on various aspects of the general theme, are here presented in a collected group.†

Concretes containing air-entraining agents are being extensively employed in present-day construction and, as entrained air alters many of the basic properties of concrete mixes, such as water requirements, sand requirements, workability, etc., a series of trial mixes was made in the laboratories of the Bureau of Reclamation to establish procedures for designing and adjusting concrete mixes containing entrained air. W. A. Cordon (p. 605) writes about the useful data that have been established from this set of tests which should facilitate the adjustment and design of air-entrained concrete and also establish a basis for further investigation.

L. E. Andrews, in his paper (p. 621) describes preliminary studies and current practice in the northeastern states in the reduction of frost action

*"Laboratory studies of concrete containing air-entraining admixtures", by Charles E. Wuerpel, ACI Journal, Feb. 1946.

†A separate publication, to be announced, will contain the symposium papers and other recent contributions to the present theme.

on concrete paving and other projects (roads, streets, airports, pavements and hangers, bridges and buildings); mix-design methods for determining air content and field control. Specifications tend to limit air content to 3 to 6 per cent rather than to fix the amount of air-entraining agents used.

Alexander Foster, Jr. reports studies by the Warner Co. over a 2½-year period with central mixed concrete using air-entraining cement. Following experimental work, more than 30,000 cu. yd. of air-entraining concrete, chiefly of low-slump, plastic mix, were used for pavements and highway work. Studies included the effect of truck mixer or agitator action, closed-drum and open-top equipment, on hauls up to 45 minutes. No significant differences in slump or air entrainment (2½ to 3 per cent) were found. The chief problem of the ready-mixed concrete people appears to be the added storage requirements necessary to meet specifications permitting no admixtures as well as those demanding air-entrained concrete (p. 625).

Stanton Walker and Delmar L. Bloem (p. 629) write of problems of air entrainment in concrete that have been particularly interesting to the ready-mixed concrete industry which has to meet a wide variety of specification requirements. Their paper describes exploratory studies in the research laboratory of the National Ready Mixed Concrete Association. Data are reported on the effects of entrained air on compressive strength and mixing water requirements. Other factors considered are: mixing time, grading of aggregate, temperature, ratio of sodium hydroxide to Vinsol resin, comparison of fresh and hardened concrete and air content at different depths of concrete.

Henry L. Kennedy (p. 641) reports studies of the homogeneity of air-entraining concrete by means of a test for resistance to abrasion. An air-entraining agent was used with no added accelerators, dispersing agents or gas forming materials. "Since air entrainment has a tendency to eliminate bleeding of concrete it is only reasonable to believe that such concretes are more homogeneous . . . . the top surface (of a slab) would have the same strength and density characteristics as the bottom surface." Plotted data show that there is progressive increase in abrasion loss in specimens as air content increases beyond 6 per cent.

Edward W. Scripture, Jr. (p. 645) discusses methods for and mechanisms of air entrainment in concrete mixes. Methods include use of aluminum and hydrogen peroxide for entrainment of hydrogen or oxygen, respectively; use of cement dispersing agents; perhaps protective colloids. Data are reported to record the relation of air content to durability as determined by freezing and thawing.

A. T. Goldbeck (p. 649) writes that a limestone sand which had been used with indifferent success in pavements and other structures, prompted a series of tests by National Crushed Stone Association, to improve workability and durability of concrete in which this aggregate was used. Results with and without Vinsol resin as an air-entraining agent were favorable to the use of the admixture. Data reported include materials, mix design, freezing and thawing tests.

W. H. Klein and Stanton Walker (p. 657) in their paper describe, with illustrations, the Klein Air Meter and a test procedure and calculation of air content based on the application of Boyle's law which has advantages over the methods now in use to determine percentage of air entrained in concrete. The paper is based on results of tests made by the Pennsylvania-Dixie Cement Corp. and the National Ready Mixed Concrete Association.

R. R. Kaufman (p. 669) describes an automatic dispensing apparatus designed for getting an admixture into the concrete mix at the mixer in any constituent of the mix, with all the accuracy desirable.

E. M. Brickett (p. 673) describes several devices for accurate measurement of liquid admixtures as introduced into the concrete batch at the mixer with special reference to air-entraining agents as used in ready-mix and concrete products plants and on paving jobs. Since the quantity of the solution is relatively small, accuracy is important for uniform results.

S. W. Benham (p. 677) reports the procedures and results of a method for field and laboratory for determining the amount of entrained air in a concrete mix.

W. F. Kellermann (p. 681) presents a part of the results obtained from an investigation of the durability of concrete by the Public Roads Administration using blends of portland cements with natural cements (86 per cent and 14 per cent by weight, respectively) and Vinsol resin-treated cements. Results presented in this contribution have a bearing on resistance to freezing and thawing tests, especially because of the unusual results of a prolonged interruption of the daily freezing and thawing cycle.

W. H. Herman (p. 689) relates the experiences of the Pennsylvania Department of Highways with air-entraining concrete in which 331,555 bbl. of normal strength portland cement containing Vinsol resin were used since 1940. The Pennsylvania Department's attitude on the subject of air entrainment is characterized by more concern with the particular admixture used than with the percentage of air entrainment and such use was inspired by difficulties with finer ground cements which prompted seeking an additive to improve pavement durability.

B. H. Wait (p. 697) suggests that air entrainment by admixtures is not the only method feasible for combatting disintegration due to frost action. Results are reported on numerous paving jobs in northeastern United States in support of the use of portland-natural cement blend as a means of reducing disintegration from frost action, where the air entrained averaged about 1 per cent only. Results were satisfactory and the weight of the concrete was higher than for straight portland-Vinsol resin mixes.

To facilitate selective distribution, **separate prints** of this title (42-25) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946

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Vol. 17 No. 6 7400 SECOND BOULEVARD 2, DETROIT, MICHIGAN **June 1946**

Entrained Air—A Factor in the Design of Concrete Mixes*

By W. A. CORDON†
Member American Concrete Institute

SYNOPSIS

Concretes containing air-entraining agents are being extensively employed in present day construction, and as entrained air alters many of the basic properties of concrete mixes such as water requirement, sand requirement, workability, etc., a series of 102 trial mixes was made to establish procedures for designing and adjusting concrete mixes containing entrained air. Useful data have been established from this set of tests which should facilitate the adjustment and design of air-entrained concrete and also establish a basis for further investigation.

INTRODUCTION

Many of the empirical rules and formulae heretofore used in concrete mix design must now be modified to allow for changes resulting from the entrainment of air. It has been established among other things that the addition of air to concrete (a) reduces the water requirement, (b) reduces the sand requirement, (c) increases the durability, (d) decreases the strength at any given W/C ratio and (e) improves the workability.

A series of 102 mixes was made in the Materials Laboratories of the Bureau of Reclamation for the purpose of establishing procedures which could be applied to the design of average concrete mixes containing nominal amounts of entrained air. The results of this series of mixes, which include aggregates from four widely separated sources, screened to 1½-in. maximum size, cements from three different mills, and two different air-entraining agents, are presented in this paper.

The results of the tests show the quantity of air entrained in a given concrete mix is dependent on a number of variables including (a) type

*Presented at Air Entrainment Session, ACI Convention, Feb. 19, 1946.
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and grading of aggregate, (b) amount and brand of cement, (c) consistency of mix, (d) type of mixing, (e) mixing time, and, of course, (f) type and amount of air-entraining agent. It is not within the scope of this paper to discuss these various ramifications of air-entraining properties in detail. The discussion is therefore purposely limited to the properties affecting basic mix design.

It is indicated from analysis of results obtained that the tests made were sufficiently conclusive in the range of normal air contents of from about 1 to 8 per cent for the materials used and for the relationships covered by the tests. The curves presented were fitted by the method of least squares and were found to be "highly significant" statistically[(1)]* indicating that the results would be reproduced 99 times out of 100 if comparable materials and methods were employed.

CONCLUSIONS

1. ACI Standard Recommended Practice for the Design of Concrete Mixes[(2)] can be readily modified to include concrete mixes containing nominal amounts of entrained air.

2. The water requirement of an average concrete mix is reduced approximately 6.0 lb. per cu. yd. with rounded aggregate and 8.0 lb. per cu. yd. with angular aggregate for each per cent of air entrained. (Fig. 1, 2, and 2A). This reduction in water at constant 0.55 W/C ratio reduces the cement content 11 lb. and 14.5 lb. per cu. yd., for natural and angular aggregate, respectively, for each per cent of air entrained.

3. The reduction in water content for each per cent increase in air is about the same for any given water-cement ratio, slump, or brand of cement. (Fig. 2A and 3).

4. Sand content, by weight of total aggregate, may be reduced by one per cent for each per cent increase in entrained air up to about 8 per cent, without any appreciable change in workability or slump. (Fig. 4). (Each per cent reduction in sand permits a 2.5 lb. per cu. yd. reduction in water in addition to that mentioned in conclusion 2 above)[(2)].

5. The workability of a concrete mix is improved as the amount of entrained air is increased. (Fig. 5).

6. Where the water-cement ratio is held constant the compressive strength of concrete will be reduced approximately 200 psi for each per cent of air entrained in the fresh concrete by Vinsol resin. (This value does not apply where accelerators are used in combination with the air-entraining agent). (Fig. 6 and 6A).

7. Where the cement content is held constant the change in strength ranges from practically no change for concretes of low cement content to a reduction of around 200 psi for each per cent of air entrained for concretes of high cement content. (Fig. 6A).

*The numbers in parenthesis refer to the references appended herto.

8. The modulus of elasticity of an average concrete having the same water-cement ratio and the same aggregate grading, will be reduced approximately 105,000 psi for each per cent of air entrained in the fresh concrete. (Fig. 7).

9. Poisson's ratio is not significantly affected by the entrainment of air.

10. The quantity of agent required to entrain a given amount of air decreases as the slump of the concrete is increased. (Fig. 8).

11. Crushed and angular aggregates will entrain more air than well rounded aggregates. (Fig. 9).

12. The quantity of agent required to entrain a given amount of air is influenced by the brand of cement; also a smaller amount of agent is required for lean mixes than for rich mixes. (Fig. 10).

DISCUSSION OF TEST RESULTS

The amount of water required to produce concrete of a desired slump is an important consideration in the design and adjustment of concrete mixes and as entrained air has a marked effect on the water requirements of a mix, considerable emphasis was placed on the determination of this effect. Fig. 1 shows that an average reduction of 6.4 lb. of water per cu. yd. of concrete results from each per cent increase in air content. Fig. 2, 2A, 2B, and 3 show a breakdown of the points plotted on Fig. 1. These figures show that this decrease in water for each increase in air is uniform for the same slump and water-cement ratio, and that the decrease in water for each increase in air will be slightly different for different aggregates and brands of cement. The most noticeable example of the difference caused by type of aggregate is shown in Fig. 2 where a reduction of about 6 lb. for each per cent of entrained air was obtained for rounded aggregate and 8 lb. for angular aggregate.

It should be noted that the values established in the foregoing discussion do not include the reduction in water content resulting from reductions in the sand percentage which may be realized because of increased workability caused by the entrained air. Table 8 of the Standard Recommended Practice for the Design of Concrete Mixes[2] shows a reduction in water of approximately 2.5 lb. per cu. yd. of concrete for each per cent decrease in sand. Therefore, if the workability of the concrete is held constant by reducing the percentage of sand by one for each per cent of air entrained as shown in Fig. 4, the total reduction in water content would average approximately 9.0 lb. for each per cent of air entrained. (Fig. 2B).

Effect of entrained air on the workability of concrete mixes

Entrained air improves the workability by increasing the slump, thereby permitting a reduction in water content, as indicated in Fig. 1. Where the slump is held constant through reductions in water and

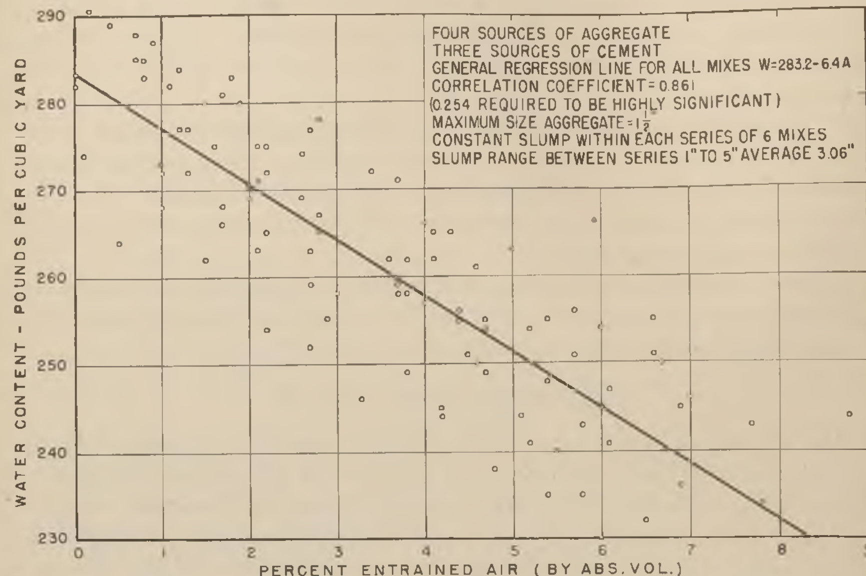


Fig. 1—An average reduction of 6.4 lbs. of water per cu. yd. of concrete results from each one percent increase in air content

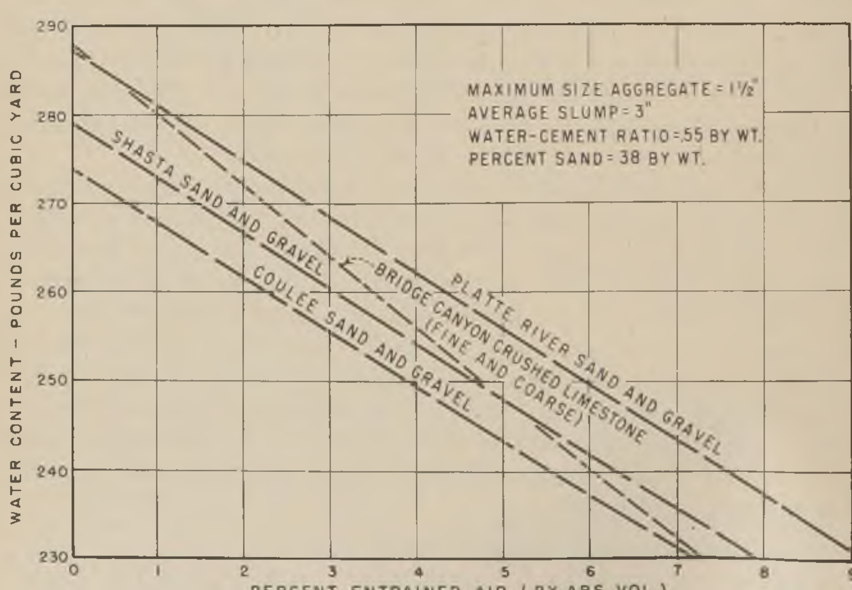


Fig. 2—The decrease in water content due to air entrainment is approximately the same for different natural aggregates, but is greater for crushed aggregate

cement as in Fig. 5, the workability was found to be better than that of the concrete of lower air content, as measured by "Power's Remolding Apparatus"[4]. Fig. 5 shows that the workability of a mix made with natural aggregates and containing 3 per cent air with a 1½-in. slump is about equal to that of a mix containing 1 per cent air with a 3-in. slump, even though the cement content has been reduced.

Decrease in strength with the entrainment of air

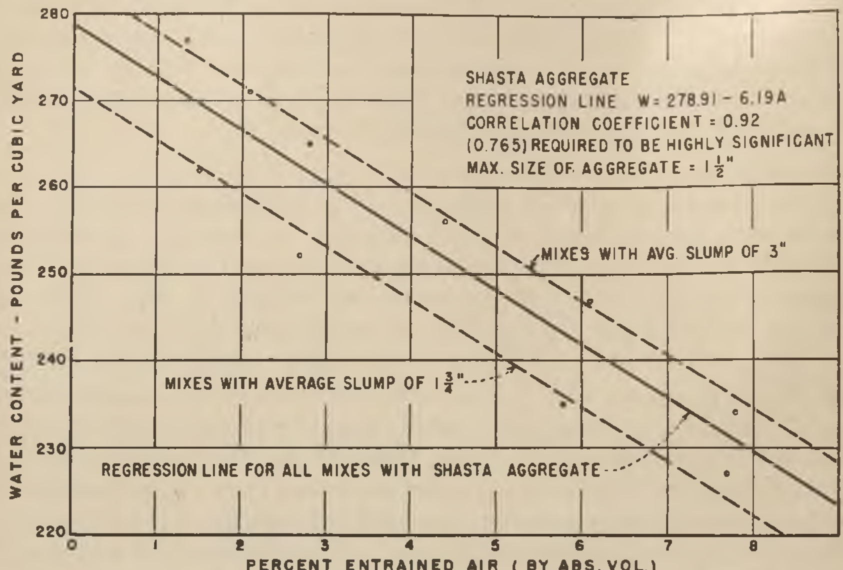
The strength of concrete as shown in Fig. 6 decreases uniformly with increases in air content of the fresh concrete. At constant water-cement ratio by weight the decrease in strength amounted to 195 psi (about 5 per cent average) for each per cent of air entrained. Also, where the W/C is held constant, Fig. 6A shows about the same reduction in strength for each per cent of air entrained for the higher constant W/C of 0.65 as for the lower constant W/C of 0.45. However, where the cement content is held constant, and the water-cement ratio is reduced through reduction in water content as a result of the entrained air, the change in strength for each per cent of air is not constant and ranges from a slight increase for mixes of low cement contents to about 200 psi reduction for each per cent of air for mixes of high cement content. It should be noted that there is little advantage in maintaining the cement content constant when using more than 6 sacks per yard, since the reduction in strength as the air is increased is about the same as that for constant W/C.

Decrease in modulus of elasticity with entrainment of air

Although there was close agreement on the reduction of the modulus of elasticity within each group of tests containing the same aggregate, Fig. 7 shows the marked difference between two different types of aggregate. The slopes of the regression lines correspond with the average of all tests, however, and at the same water-cement ratio a loss of approximately 105,000 psi static modulus of elasticity for each per cent of air entrained can be expected for average concrete mixes.

TEST PROCEDURE

A large number of tests were made in order that the influence of the numerous variables which affect the air-entraining properties of a concrete mix could be established with a reasonable degree of certainty. A range of slumps, water-cement ratios, air percentages, brands of cement and types of aggregate were included in the series of 102 concrete mixes. The percentage of entrained air by absolute volume was used as the independent variable throughout the entire investigation, and an effort was made to vary the amount of air in each set of 6 mixes by increments of 1 per cent from approximately 0 to 6 per cent. Agent was added to all mixes to obtain a pre-established percentage of air regardless of the amount of agent required. A standard solution of Vinsol resin was made



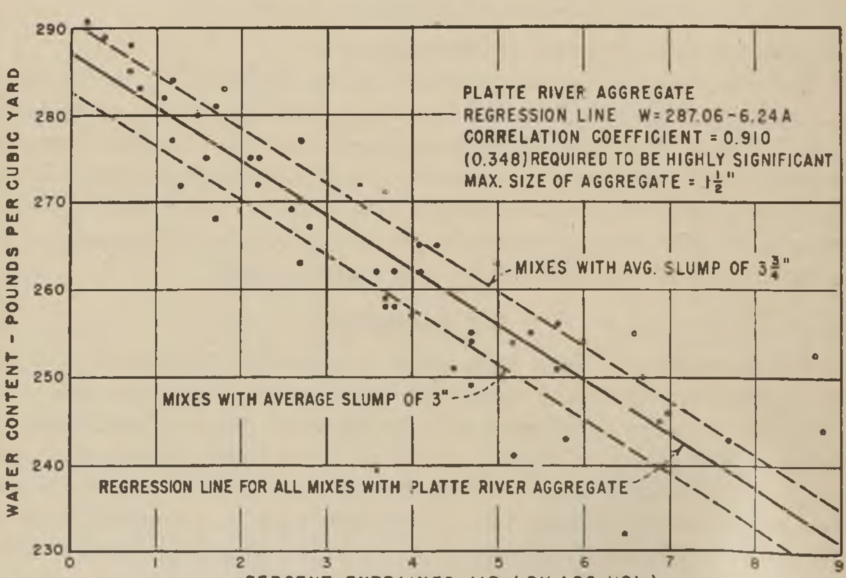
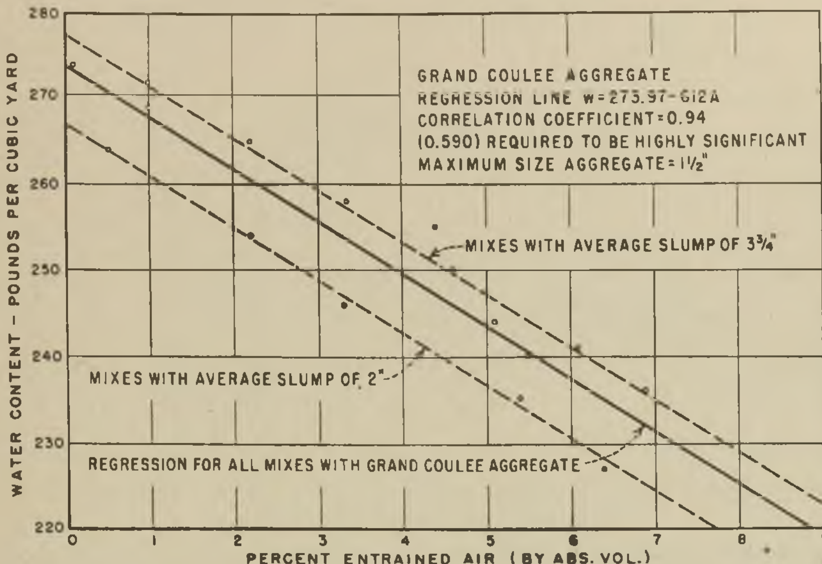
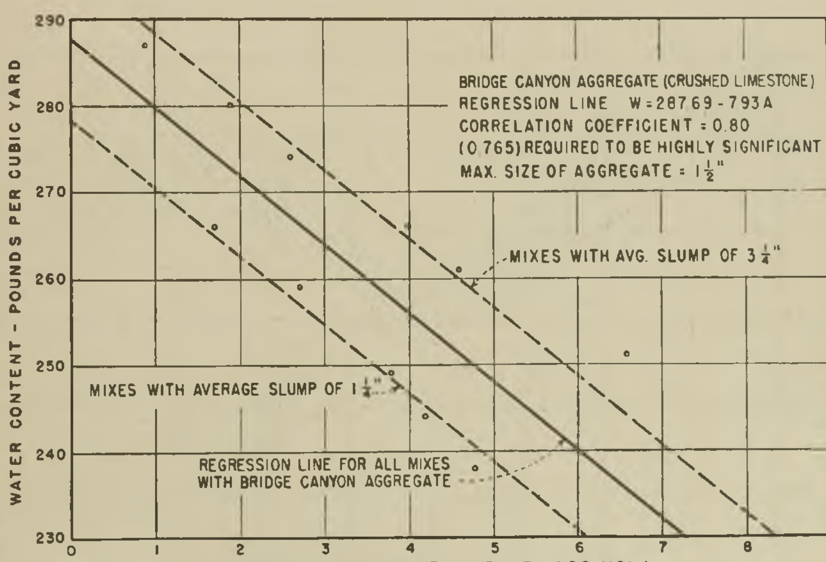


Fig. 2a—The decrease in water content due to air entrainment





is approximately the same for different slump

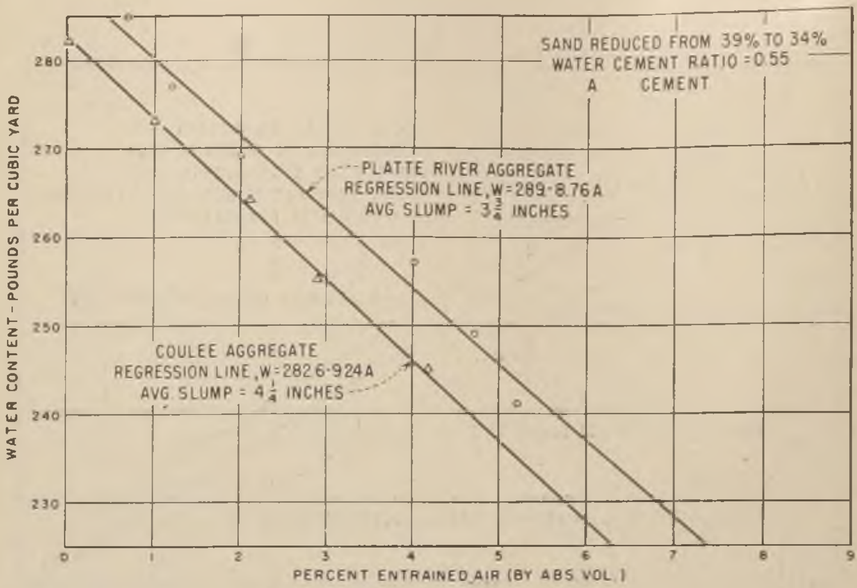


Fig. 2b—If the percentage of sand is reduced by one for each per cent increase in air content, a reduction of 9.0 lb. of water per cu. yd. is realized for each per cent of air entrained

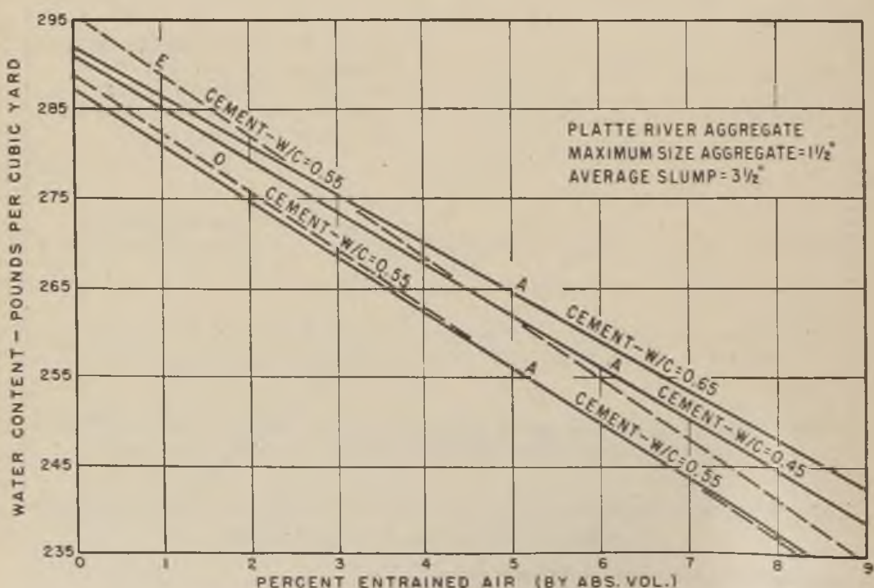
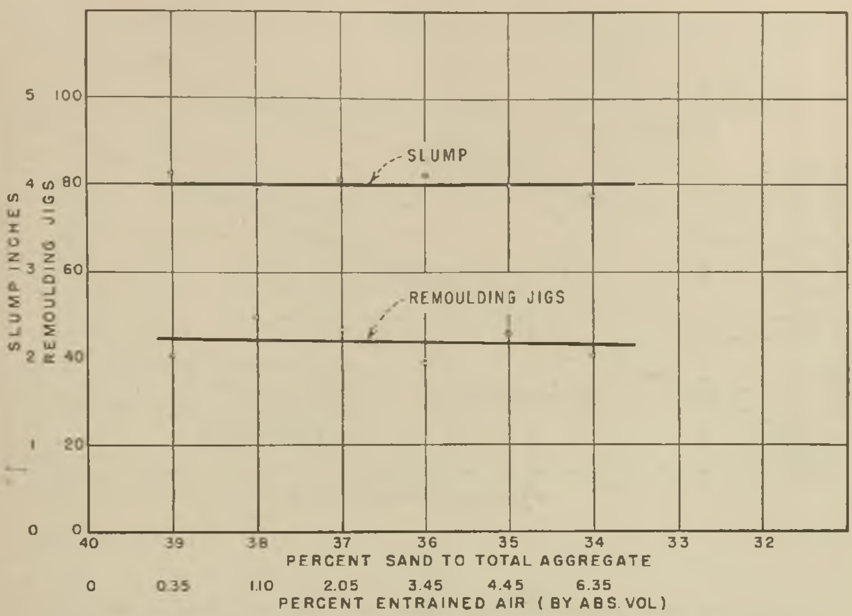


Fig. 3—The reduction in water content for each increase in air entrained is about the same regardless of the water-cement ratio or brand of cement



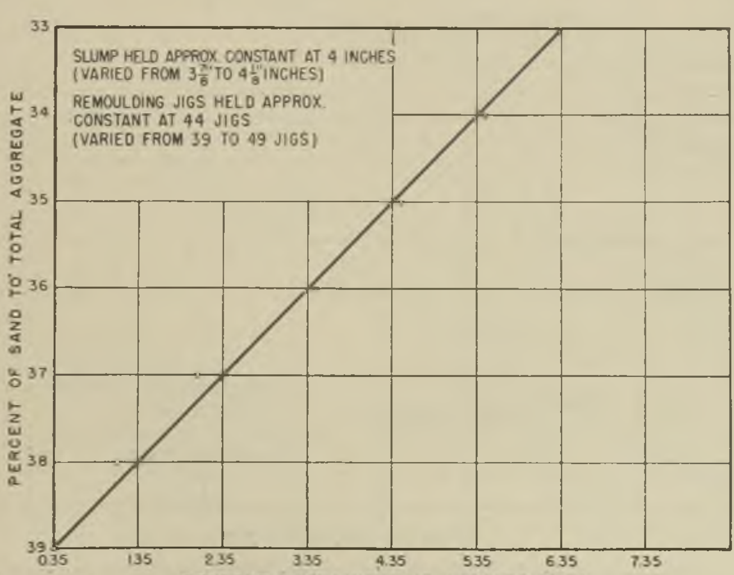
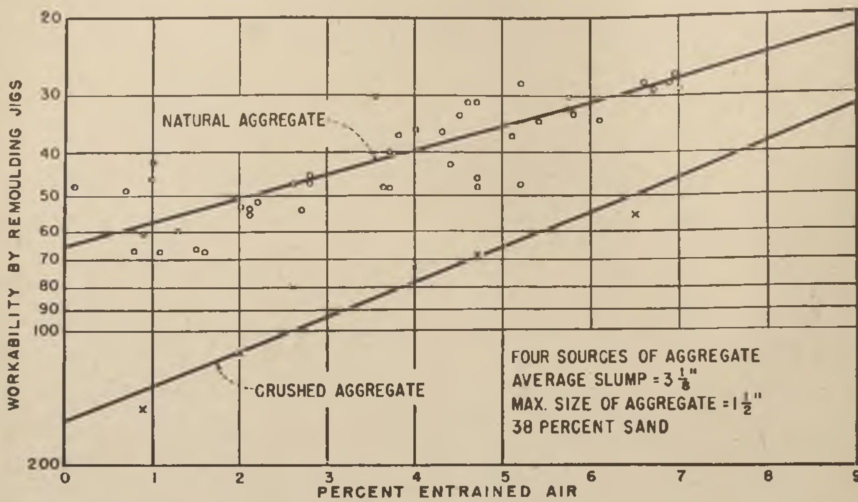


Fig. 4—The workability and slump remain about the same if the percentage of sand is reduced by one for each per cent increase in air content



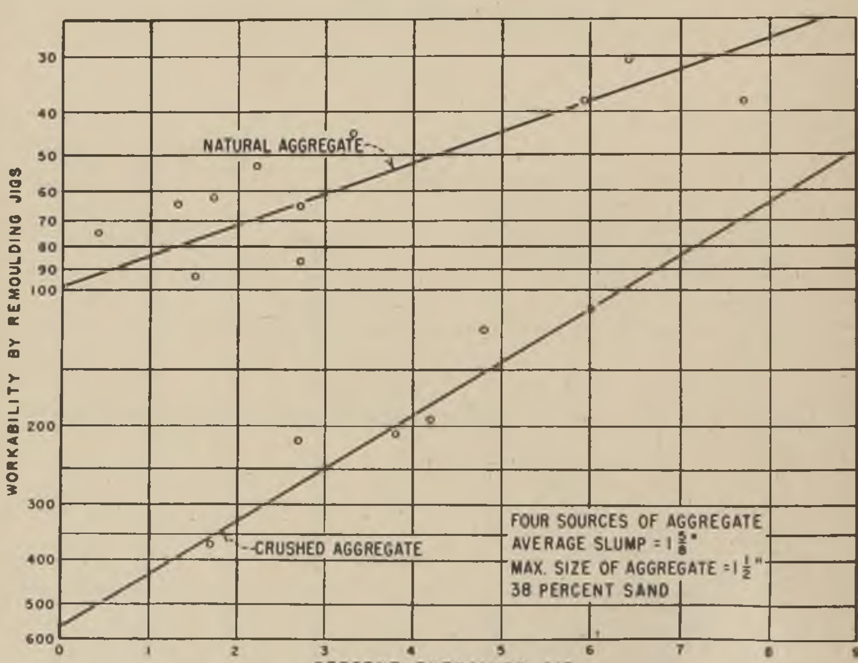


Fig. 5—At the same slump the workability of concrete is increased as the percentage of air is increased

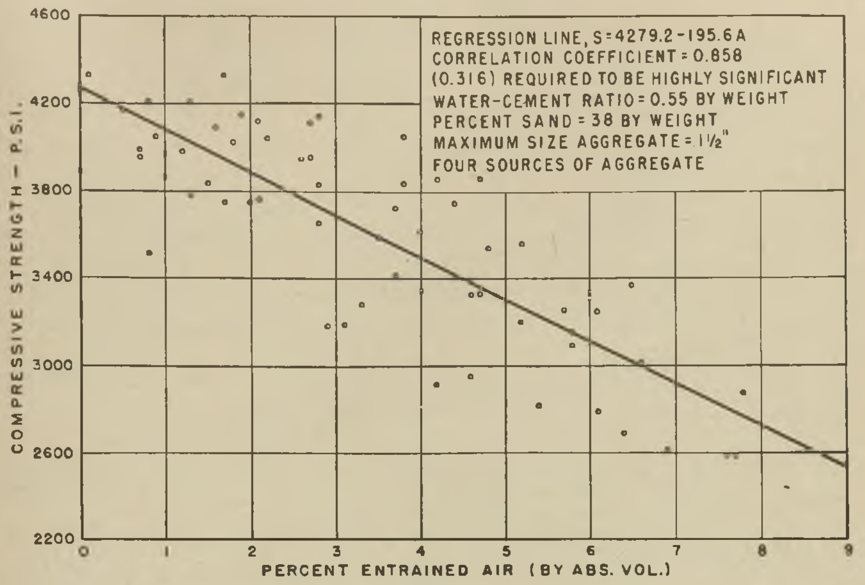


Fig. 6—Concrete having the same water-cement ratio and the same aggregate grading is reduced in strength approximately 195 psi for each percentage increase in air content

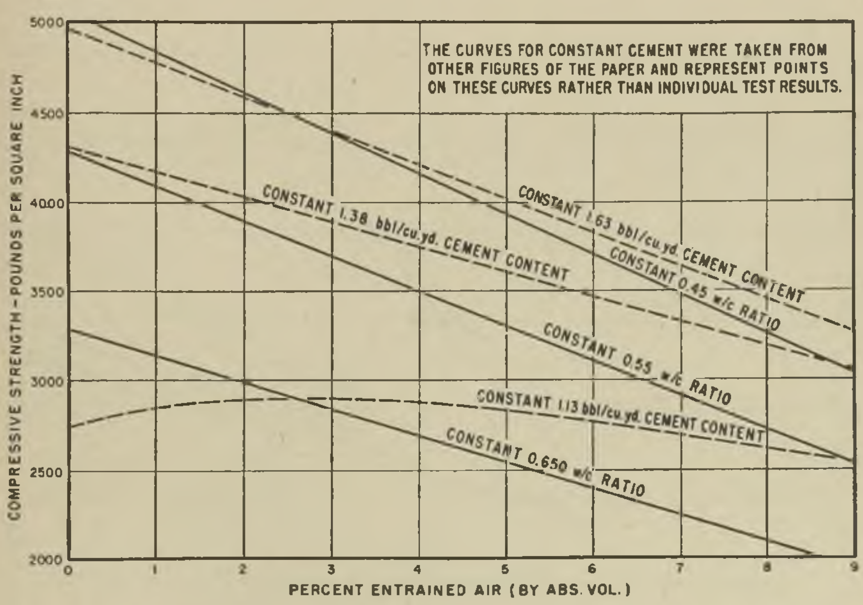


Fig. 6a—The decrease in strength as the air content increases is about the same for different water-cement ratios but for constant cement contents is influenced by the amount of cement in the mix

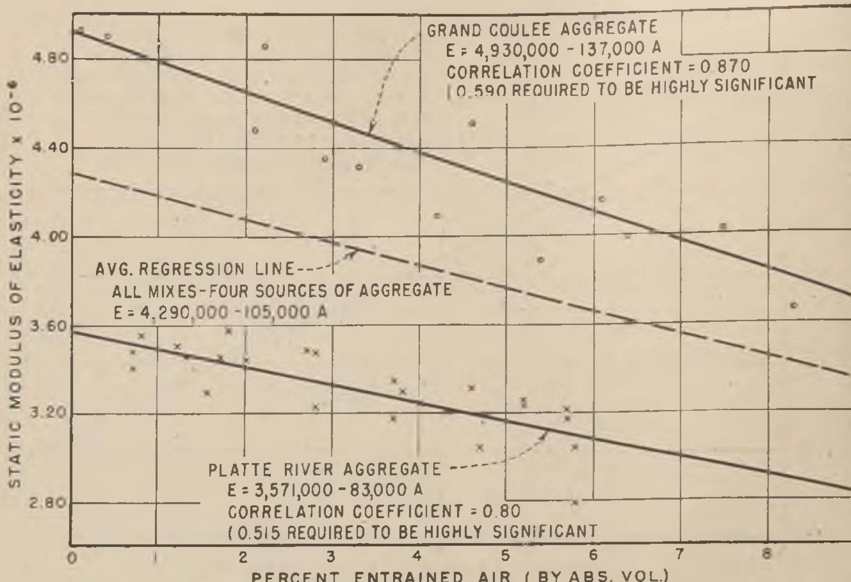


Fig. 7—Modulus of elasticity is decreased 105,000 psi for each percentage increase in air content

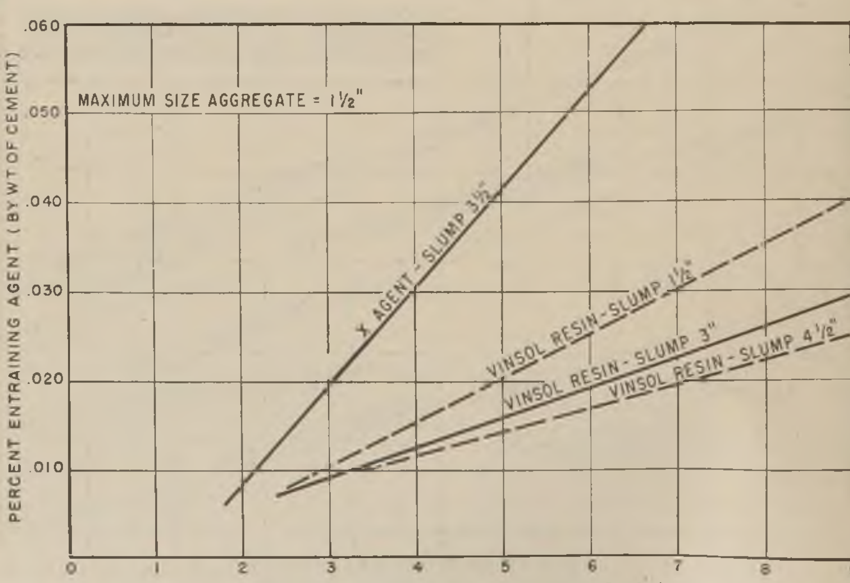


Fig. 8—The amount of air entrained in a given mix, is influenced by the slump and the kind of agent used

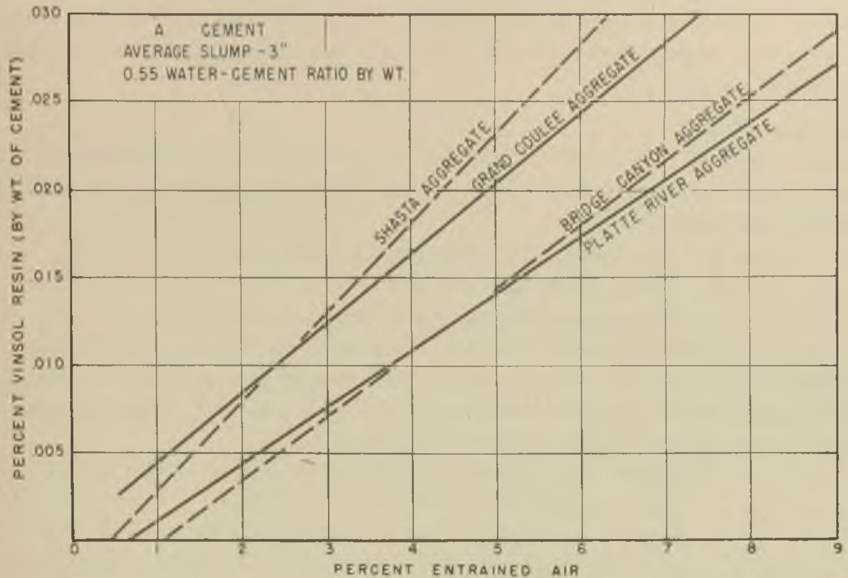


Fig. 9—The amount of air entrained in a given mix, is influenced by the aggregate used

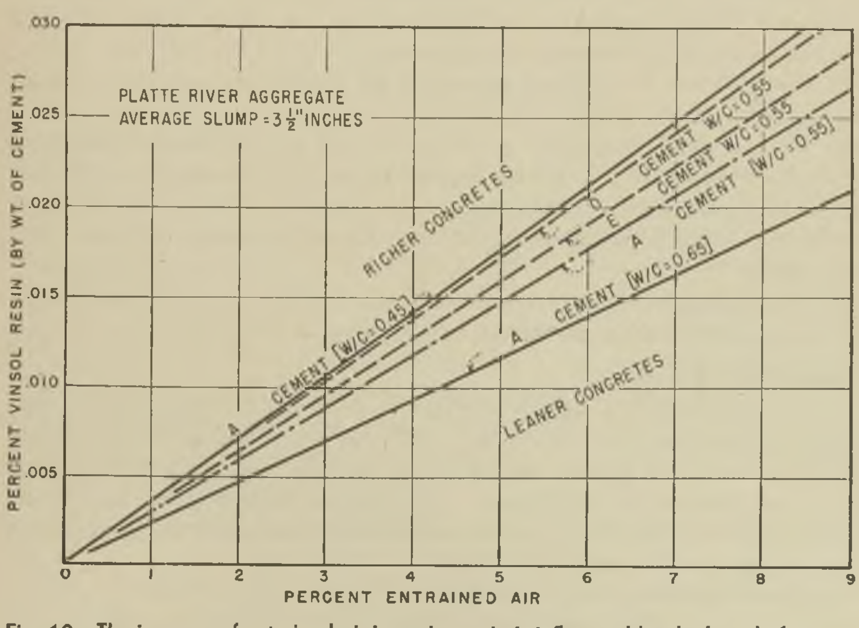


Fig. 10—The increase of entrained air in a given mix is influenced by the brand of cement and the water-cement ratio

before the mixing program started, and this supply was used throughout, with the exception of 12 mixes in which another commercial air-entraining agent was used.

Mixing procedure

All mixes of this study were made in a $1\frac{1}{2}$ cu. ft. tilting laboratory mixer with a mixing period of 5 minutes. The charging sequence of the mixer was as follows:

1. One half of the mixing water and agent
2. Sand, gravel and cement combined
3. Remaining half of mixing water and agent.

The air-entraining agent was combined with the mixing water before introduction into the mixer. After the mixing period, the mixer was discharged into a large pan and re-worked with a shovel. The following tests were made on each batch of fresh concrete:

Slump test—A.S.T.M. Des. (C143-39)
Unit weight—A.S.T.M. Des. (C138-44)
Powers Remolding Test for Workability[4].

At 28 days three 6 x 12 in. cylinders from each batch were tested for compressive strength, A.S.T.M. Des. (C192-44T), and elastic properties.

ADJUSTMENTS AND COMPUTATIONS

Table 5 of the ACI Standard Recommended Practice for the Design of Concrete Mixes[2] could be slightly modified for air entrained concrete by including the following information:

(a) For concrete containing entrained air reduce the sand percentage by one for each per cent of air entrained.

(b) For concrete containing entrained air reduce the water content 8.5 lb. for well rounded natural aggregate and 10.5 lb. for angular aggregate for each per cent of air entrained.

The following adjustment can be placed as a footnote to Table 2 of the standards:

(a) For the same W/C ratio reduce strengths shown in table by 200 psi for each per cent of air entrained in concrete.

Computation of trial mixes

It is very simple to slightly modify the trial-mix computations as outlined in the recommended practice and allow for entrained air by simply treating the air as another ingredient of the concrete mix which replaces an equal volume of aggregate. The sample computations shown on pages 657 and 658 of the above mentioned standards can be modified for air-entrained concrete as follows:

(Assume 3.0 per cent air).

$$\text{Cement content} = \frac{\text{net water content}}{\text{water-cement ratio}}$$

$$= \frac{305}{0.53} = 575 \text{ lb. per cu. yd.}$$

$$= \frac{575}{94} = 6.12 \text{ sacks per cu. yd.}$$

Absolute volume water + cement

$$= \frac{\text{water content}}{62.4} + \frac{\text{cement content}}{\text{specific gravity} \times 62.4}$$

$$= \frac{305}{62.4} + \frac{575}{3.15 \times 62.4} = 7.81 \text{ cu. ft. per cu. yd. concrete}$$

Absolute volume air = per cent air × cu. ft. per cu. yd. of concrete
= 0.03 × 27 = 0.81 cu. ft. per cu. yd. of concrete

Absolute volume total aggregate
= 27 − absolute volume (water + cement) − absolute volume air
= 27 − 7.81 − 0.81 = 18.38 cu. ft. per cu. yd. of concrete

Absolute volume sand = per cent sand × absolute volume total aggreate = 0.42 × 18.38 = 7.72 cu. ft. per cu. yd. of concrete

Absolute volume coarse aggregate
= absolute volume total aggregate − absolute volume sand
= 18.38 − 7.72 = 10.66 cu. ft. per cu. yd. of concrete

Sand content = absolute volume × specific gravity × 62.4
= 7.72 × 2.65 × 62.4 = 1,277 lb. per cu. yd. of concrete

Coarse aggregate content
= 10.66 × 2.55 × 62.4 = 1,696 lb. per cu. yd. of concrete

$$\text{Trial-mix proportions} = \frac{575}{575} : \frac{1{,}277}{575} : \frac{1{,}696}{575}$$

$$= 1 : 2.22 : 2.95, \text{ say } 1 : 2.2 : 3.0$$

Computation of air content

The computation of the air content of freshly mixed concrete naturally becomes an important function in the design of concrete mixes in which air-entraining agents are used. A.S.T.M. Des. (C173-42T) "Test for Air Content of Concrete" is not recommended for mortar containing soap forming materials and is not widely used for air-entrained concrete. The most popular method of determining the percentage of entrained air is A.S.T.M. Des. (C138-44) which assumes that the difference between the unit weight computed from the absolute volume of the cement plus water plus aggregate and the unit weight determined with the freshly mixed concrete is due to the amount of entrained air in the mix. The percentage of entrained air is therefore equal to

$$\text{Per cent air} = \frac{(\text{Theoretical unit weight} - \text{measured unit weight}) \times 100}{\text{theoretical unit weight}}$$

Using weights computed in the foregoing trial-mix computation, the per cent air can easily be determined as follows: (Measured unit weight = 142.7 lb. per cu. ft.)

Theoretical unit weight

$$= \frac{\text{weight cement} + \text{weight water} + \text{weight aggregate}}{\text{solid volume of cement} + \text{water} + \text{aggregate}}$$

$$= \frac{575 + 305 + 2{,}973}{27 - 0.81}$$

$$= \frac{3{,}853}{26.19} = 147.1 \text{ lb. per cu. ft.}$$

and

$$\text{Per cent air} = \frac{147.1 - 142.7}{147.1} \times 100 = 3.0 \text{ per cent}$$

Designation 24 of the U. S. Bureau of Reclamation Concrete Manual outlines a general method for the computation of per cent voids (per cent air) based on the actual mix parts of any combination of cement, water and aggregate. A simplification of this method is presented in the following general formula:

$$\text{Per cent air} = 100 - \frac{\left(\frac{1}{G_C} + W/C + \frac{P_A}{G_A}\right) 100\ W}{62.4\ P_T}$$

where

P_T = Total parts (cement, aggregate, water) in mix (by weight)
P_A = Parts of aggregate (by weight)
G_C = Specific gravity of cement
G_A = Specific gravity of aggregate
W = Measured unit weight of fresh concrete (lb. per cu. ft.)
W/C = Water cement ratio by weight.

ACKNOWLEDGMENT

The tests were made in the Denver laboratories of the U. S. Bureau of Reclamation under the direction of Robert F. Blanks, Chief, Engineering and Geological Control and Research Division and were supervised by W. H. Price, Head of the Materials Laboratories. The writer is indebted to Mr. Price and to L. H. Tuthill for many valuable suggestions and criticisms throughout this investigation and to the laboratory personnel for valuable assistance in conducting the tests.

All design, construction, and research work of the Bureau are under the supervision of Walker R. Young, Chief Engineer; and all activities of the Bureau are under the direction of Michael W. Straus, Commissioner.

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To facilitate selective distribution, **separate prints** of this title (42-26) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced.
Discussion of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946

Title 42-26 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Recent Experiences with Air-Entraining Portland Cement Concrete in the Northeastern States*

By L. E. ANDREWS†

SYNOPSIS

Describes preliminary studies and current practice, in the northeastern states in the reduction of frost action on concrete paving and other projects (roads, streets, airport pavements and hangars, bridges and buildings); mix design, methods of determining air content and field control. Specifications tend to limit air content to 3 to 6 per cent, rather than fix amount of air-entraining agent used.

The first activity with air-entraining concrete in the northeastern states area was started in 1938. The next 4 years, up to the war period, embraced the construction of experimental projects by numerous states and other agencies. For example, the highway departments of New York, Pennsylvania, Massachusetts, Maine and Vermont used air-entraining portland cement, in addition to other cement variables, for comparative performance purposes in these experimental jobs. The results with A-E concrete proved to be highly satisfactory.

New York was the first state in this group to start such experimental work. Most of these projects were built in 1939-40. Last year I had occasion to make close observations on this work by walking the entire length of these jobs and noting all details of existing condition. What we believe to be more complete information today, in the light of past experiences, was naturally not known or available at the time these pavements were built. Nevertheless, all of the sections observed on this inspection were in excellent condition showing no scaling or disintegration of any kind. However, in many instances, considerable scaling was observed on adjacent sections where A-E portland cement was not used.

In fact, our observation is that where it is known that at least 3 per cent air entrainment was secured in the fresh concrete, and normal

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Regional Highway Engineer, Portland Cement Association, New York City.

materials and construction methods used, there has been no subsequent scaling or other disintegration.

During the war period great impetus was given to the use of A-E portland cement concrete in military installations and particularly at airports for paving and, in fact, also for much structural work. The War Department became one of the largest users and extensive studies were made by the Central Concrete Laboratory of the Corps of Engineers.

Since V-J day and the advancing of postwar programs, particularly in the paving field, attention has been focused on the desirability of benefitting from this previous experience with A-E concrete by providing for its use on current and future work by definite specification adoption.

At this time most of the northeastern states have either adopted specifications for A-E concrete for paving work or are in the process of preparing such specifications.

During the early use of A-E portland cement concrete in this area there were a few instances where too much air was developed in the mix. This was under the old specification where fixed limits were placed upon the amount of A-E agent interground with the cement.

It was soon recognized that the same percentage of A-E agent in different cements would not necessarily induce similar amounts of air in the same class of concrete mixture. It then became more than ever apparent that the important feature is control of the air content and this could not be insured by rigidly fixing the limits for the amount of A-E agent used in the manufacture of the cement.

CURRENT PRACTICE

Now we have the improved specification represented by A.S.T.M.: C175-46T which provides for a permissible range of 12 to 20 per cent air entrainment in a standard mortar test.

It is expected that this latest revision will preclude any tendency toward less than minimum air content in the fresh concrete. It therefore appears that the possibility of securing occasional mix combinations resulting in air content outside the desired range of 3 to 6 per cent has been narrowed considerably.

The principal field of use for A-E concrete in the northeast has been that of pavements for airports, roads and streets. It has also been introduced into many other types of structures including hangars, buildings, bridges, piling, flood control facilities, and other miscellaneous uses. The U. S. Engineer Office in this area, reported last year that A-E portland cement concrete had been used on 170 projects of all kinds in their various activities.

More than a dozen major air bases in which A-E portland cement concrete was used for the paving, embracing considerably more than 3,000,000 sq. yd., were constructed here during the war years 1942-45. One of the largest of these was Stewart Field, West Point, with a total of 1,300,000 sq. yd.

Numerous A-E concrete paving projects have also been constructed during these years in Pennsylvania, New Jersey, New York, Connecticut, Massachusetts, Vermont and Maine, and practically without exception they show no scaling or disintegration.

CONCRETE MIX DESIGN

Concrete mix design and control on all of this work has been simple. Generally, the mix set-up has been based on proportions ordinarily used with normal portland cement for similar work but with sand and water content reduced by an amount to compensate for volume occupied by the increased air-content thus maintaining constant yield.

Such adjustments allow for the same cement factor and about the same consistency. Workability of the resulting concrete is much improved, segregation is minimized and bleeding is considerably reduced. This latter quality allows finishing operations on concrete paving to keep close behind the paver.

On paving work where gravel coarse aggregate is used, water ratios have ranged from about 4.75 to 5.25 gal. per bag of cement with sand-aggregate ratios (by absolute volume) from 29 to about 34 per cent. Where crushed stone coarse aggregate is used the water ratio range has been from about 5.00 to 5.50 gal. per bag while sand-aggregate ratios have been about 33 to 38 per cent. The slump has ranged from about 1½ to 2½ in.

DETERMINING AIR CONTENT

To date, air entrainment has generally been determined by the gravimetric method using the standard unit weight test, comparing actual unit weight of the concrete with calculated air-free unit weight. This method requires accurate data on specific gravities, free moisture, absorbed moisture, water added at paver, and batch weights.

In some instances where normal portland cement has been available on the job at the start of paving operations, it has been used in establishing a unit weight of concrete for basis of comparison with the unit weights secured later with A-E portland cement. In such cases it has been assumed that the air content for the normal cement mix is 1 per cent. Usually, however, where A-E portland cement is specified there is no other cement on the job and the calculated air-free unit weight must then be determined.

Considerable interest has been expressed relative to the "Indiana Method" of determining air content by use of the hook gage in combination with the usual equipment. If close agreement is shown with the standard gravimetric field method in determining volume of air, it would appear that this method may receive wide use because it affords a direct measurement.

FIELD CONTROL

It is always of particular interest to know what was done to correct conditions resulting in air content falling outside the desired range. These occurrences have been few in number, and, as noted above, it is believed that current specification changes will tend to prevent repetition of such instances. However, on a railroad job last year it was necessary to add a neutralized Vinsol resin solution to the batch in order to increase the air content from less than 2 per cent to slightly above 3 per cent. The quantity added was calculated to be about 0.002 per cent which brought the total in this case to about 0.034 per cent.

In another instance, on a highway job, it was possible to increase the air content about 1 per cent by increasing the sand-aggregate ratio (gravel coarse aggregate) from 29 per cent to about 36 per cent.

On a prewar airport project where excessively high air content occurred, it was necessary to blend off with normal portland cement in order to reduce the air to desired limits. Such instances have been the exception and not the rule.

INCREASED DURABILITY

The use of air-entraining concrete in the northeastern states has resulted in greatly increased durability, that is, resistance to freezing and thawing and salt action in all cases where sufficient air-entrainment was obtained.

Practically all specifications now require that the entrained air in the fresh concrete, as determined by the standard yield test, shall come within the range of 3 to 6 per cent. This requirement, with the selection of proper materials, careful mix design and conscientious field control gives assurance, based on experience over the past 8 years, that scale free concrete pavements are an accomplished fact in our northern states.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN June 1946

Experiences with Air-Entraining Cement in Central-Mixed Concrete*

By ALEXANDER FOSTER, Jr.†
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SYNOPSIS

Reports studies by Warner Co. with central-mixed concrete using air-entraining cement over a 2½-year period. Following experimental work, more than 30,000 cu. yd. of air-entraining concrete, chiefly of low-slump, plastic mix were used for pavements and other highway work. Studies included effect of truck-mixer or agitator action, closed drum and open top equipment, on hauls up to 45 minutes. No significant differences in slump or air entrainment (2½ to 3 per cent) was found. The chief problem is in added storage requirements to meet specifications permitting no admixtures and those demanding air entrainment.

Warner Company has done a considerable amount of work with air-entraining cement over the past 2½ years. Prior to committing the Company to furnish concrete containing entrained air commercially, experimental work was carried out to familiarize ourselves with its peculiarities. Subsequently, over 30,000 cu. yd. have been delivered, mostly for concrete pavements and highway structures. Consequently, the major part of our experience has been with relatively low-slump, but plastic concrete—in general, with slumps of less than 3 in.

Our experimental work was inspired by some early experiences in the industry which had suggested that excessive amounts of air would be entrained in the ready-mixed concrete operation, with consequent difficulties of control. It was variously claimed that the slow, folding, mixing action of a truck mixer, or agitator, the prolonged mixing incidental to transportation, and the air-tight characteristics of the horizontal drum type of mixer or agitator caused the entrainment of greatly more air than for the usual job mixer. A series of tests was conducted

*Presented at Air Entrainment Session, 42nd ACI Convention, Feb. 19, 1946.
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by the Company's concrete engineer, H. J. Knopel, to develop information on these and other points. The cooperation of representatives of the Pennsylvania Highway Department and Stanton Walker of the National Ready Mixed Concrete Association was had in the outline and conduct of the investigation. The experience of these tests, reported elsewhere*, did not support the claims that pre-mixed concrete entrained more air than that produced by job mixing. The Company accepted contracts to furnish concrete to several large highway jobs, with satisfactory results to all concerned.

The experimental work, conducted in 1944, had shown that the use of some cements would result in the entrainment of too much air. It became clear that mixing time, but *not* the nature of the mixing action or of the mixing drum, tended to increase the amount of air entrained in some cases. Apparently that was due, according to the generally accepted explanation, to the universal use, at that time, of the dry flake material (not preneutralized) for intergrinding with the cement. Whatever the cause of this variable the air-entraining properties of the cement were investigated in concrete in the laboratory before being accepted for commercial work. This was in addition to the usual test to determine conformity with A.S.T.M. specifications. That practice resulted in the rejection of some cement but avoided complications on the job. As a further precaution weight per cu. ft. tests of the concrete from which air content could be computed, were made throughout the progress of all work. Even today, before cement is accepted for a large road job, samples of the specific brand to be used are obtained and tests are run in the laboratory with the actual amount of cement, water and particular quantities and grading of fine and coarse aggregate to determine the air entrainment by means of the wet weight of the resulting concrete.

With the adoption of limitations on the amount of air entrained in a standard mortar test (A.S.T.M. Method C 185), and perhaps with changes in technic of cement manufacture, no difficulties have been experienced with excessive air entrainment. On the contrary, the tendency has been in the direction of entraining an amount of air near or slightly below the generally accepted minimum.

The need for paving several areas in one of the Company's distributing yards, afforded an opportunity to conduct experiments with full-sized batches of concrete. Centrally mixed concrete, in 5¼ cu. yd. batches, was transported for various periods up to 90 minutes in both a horizontal drum and a high-discharge type of truck agitator. For another paving job we used an open-top dump body. Concrete, in 3 cu. yd. batches, was transported for periods up to 45 minutes.

*A.S.T.M. *Bulletin*, October, 1944; and ACI Journal, June, 1944, *Proceedings*, V. 40, p. 547.

The truck agitators were revolved at the slow speed of 2 r.p.m.—our standard practice. For the periods investigated, no significant effect on either the amount of air entrained or the slump was found. With normal atmospheric temperatures the recession in slump seems to be less with air-entraining cement. However, in hot weather, or when heated concrete is provided in cold weather, recession in slump for air-entraining-cement concrete requires the usual compensating amounts of water. It should be pointed out, however, that although an air-entraining cement was used, an amount of air between only about 2½ and 3 per cent was entrained. A further observation was that no important differences were shown as between the two types of drums. It did appear that these very stiff mixes were discharged somewhat more readily from the horizontal drum than from the high-discharge drum, but the difference was not enough to be considered of practical importance.

The question of the use of non-agitating equipment (open-top dump bodies) has come very much to the fore with the advent of air entrainment. Our work was done with a water-tight body having a sloping rear and no tail-gate. It might be described roughly as being something of the nature of a bath tub. The results with this method of transportation were not encouraging. It is true that the entrainment of air reduced segregation and packing of the concrete, but it did not eliminate them. The top, middle, and bottom strata of the concrete had a decidedly different appearance. Difficulty was experienced in discharging the load from the body. Perhaps more satisfactory results would have been obtained with a better designed container, but our experience suggests that, except for special cases, the need for agitation during transportation is not eliminated by air entrainment.

The experimental field concrete just referred to included concrete made with normal cement and concrete of a wide range in consistency and cement content. Some of the loads with the open-top dump body were made to contain excessive air, by use of sodium hydroxide solution of Vinsol resin as an admixture. Standard cylinders were molded for compression tests at 7 and 28 days. Opportunity has not been had to analyze these data thoroughly and, further, they would be too voluminous to report here.

However, something can be said concerning the effect of air entrainment on strength, based on our routine tests. Mr. Knopel not only designs and controls our concrete mixes, but he makes regular tests of representatives batches of the different classes of concrete each day. A comparison of the average level of strengths for concrete made with air-entraining cements as compared with that for standard cement indicates, for the pavement type of mix, about 80 per cent of normal. These pavement mixes tend to be under-sanded under usual circumstances and, as air is entrained, we are required to reduce the sand content to compensate for the volume of air. For structural concrete with the somewhat leaner cement and with proportionally higher sand and water content the level of concrete strength for air-entraining cements averages about 90 per cent of the normal cement concrete strength.

Questions have been raised concerning difficulties in handling air-entraining cement in bulk. We have taken pains to observe if there is an unusual tendency of the cement to leak from openings in bins, gates, chutes, screws, etc. No handling difficulties have been encountered which do not appear to be common to all cements.

Questions have also been raised concerning the "dischargeability" of concrete made with air-entraining cements as compared with standard cements. Because of some difficulties which we thought we encountered (they turned out to be for another reason than air entrainment) we paid particular attention to this problem and we also made a survey of experiences over a fair cross-section of the industry. The consensus of our own experience and of the survey is that such differences as exist are negligible and, if anything, favor the concrete containing entrained air. With the air-entraining cement, the concrete seems to have less tendency to cling to the drum surfaces than normal-cement concrete—and, in general, the problem of keeping mixers clean may be somewhat less.

The introduction of air-entraining cements has presented one outstanding difficulty to the ready-mixed concrete producer. It lies in facilities for storage. Under normal conditions, minimum storage facilities for two or three cements are required—Type I or II, and sometimes both, and the high-early-strength Type III. That condition commonly is still further aggravated by the necessity of maintaining special bins for particular jobs. The additional complication presented by the addition of Types Ia and IIa, with Type IIIa in the wind, will be readily recognized. An immediate solution would be to use air-entraining admixtures, with which we have had some limited and, on the whole, satisfactory experience. However, in our area, admixtures are not permitted by certain specifications.

To sum up, Warner Company's management looked with fear on the use of air-entraining cements. Today that fear is gone, although the recognition of new problems of control exist. While we have suffered some loss in strength due to entrained air, we have, nevertheless, been able to maintain the required strengths. Important difficulties of handling either the cement or the concrete have not been encountered. On the other side, the complications introduced by additional storage requirements are serious.

To facilitate selective distribution, **separate prints** of this title (42-28) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946

Title 42-28 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Studies of Concrete Containing Entrained Air*

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SYNOPSIS

Problems of air entrainment in concrete have been particularly interesting to the ready-mixed concrete industry which has to meet a wide variety of specification requirements. This prompted exploratory studies in the Research Laboratory of the National Ready Mixed Concrete Association. Data are reported on effect of entrained air on compressive strength and mixing water requirements. Other factors considered are: mixing time, grading of aggregate, temperature, ratio of sodium hydroxide to Vinsol resin, comparisons of fresh and hardened concrete, and air content at different depths of concrete.

Entrainment of air in concrete has introduced new problems in the control of concrete quality. These problems have been of particular interest to the ready-mixed concrete industry, whose members are required to produce concrete meeting a wide variety of requirements. To develop information on the effects and control of air entrainment, helpful to the ready-mixed concrete manufacturer, a number of studies have been made in the Research Laboratory of the National Ready Mixed Concrete Association at the University of Maryland. Much of that work has been exploratory in nature to obtain information indicating trends.

The more comprehensive tests have dealt with effects of entrained air on strength and mixing water requirements. Other factors studied have included: time of mixing, grading of aggregates, temperature of concrete, make-up of Vinsol resin solution, relationships between fresh and hardened concrete, and miscellaneous phases of the general problem. This paper presents brief summaries of most of these studies. For some

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of them, more detailed reports are available and may be obtained on request to the authors.

CONDITIONS OF TESTS

The data summarized are derived from a number of independent investigations. However, the conditions under which they were made are sufficiently similar that a general statement on procedures should be sufficient.

For the tests reported air-entraining admixtures were used, instead of air-entraining cements, to provide for a wide range in air content. Most of the work was with the flake Vinsol resin; some tests were made with the sodium resinate (N.V.X.) and with Darex. The first two are products of the Hercules Powder Co., and the last a product of the Dewey and Almy Chemical Co. The flake Vinsol resin was used in a sodium hydroxide solution and the sodium resinate in a water solution; the Darex was used as received.

The concrete was made with Type I cement and well graded sand and gravel of 1 in. maximum size. Unless otherwise noted it contained 5.5 sacks of cement per cu. yd. It was designed to produce mixtures of good workability and of a consistency represented by a slump of 3 to 4 inches. It was mixed in 0.5 cu. ft. batches in a small tilting mixer of 1 cu. ft. capacity. Except as otherwise noted the mixing time was 6 minutes, at 16 r.p.m., representing a peripheral speed of 90 ft. per minute. With the entrainment of air, the volume of sand was reduced to maintain, in so far as practicable, the same absolute volume of cement and coarse aggregate.

Where strength tests were involved specimens were cured in a standard moist room until tested. Air contents reported were determined by the gravimetric method—i.e., computed from the absolute volumes of the ingredients of the batch. The weight per cu. ft. measurements were made in a 1/5 cu. ft. measure.

EFFECT OF ENTRAINED AIR ON COMPRESSIVE STRENGTH

Fig. 1 summarizes results for 7 and 28-day compressive strengths of concrete containing 4.5, 5.5, and 6.5 sacks of cement per cu. yd. Vinsol resin was added in amounts of 0, 0.005, 0.010, 0.015, and 0.020 per cent of the weight of cement, resulting in air contents up to about 9 per cent. It will be seen that, for the 5.5 and 6.5 sack concrete, strength was reduced as air was entrained. For the lean 4.5 sack mix the strength was slightly increased for air contents up to about 6 per cent. This beneficial effect on the lean mix, it seems reasonable to suppose, was due to the improvement in workability—in spite of the fact that, according to usual standards, the air-free mix was workable. For 5 per cent of added air, an average desirable amount, the results may be summarized as in Table 1.

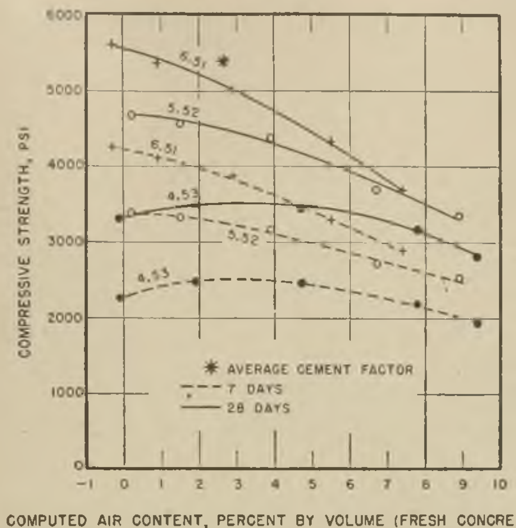


Fig. 1—Relationship between air content and strength (series 66).

TABLE 1

Cement sacks per cu. yd.	Percent change in strength due to 5 per cent added air	
	7-day	28-day
4.5	+9	+4
5.5	−12	−16
6.5	−17	−20

Entrained-air to strength relationships reflect not only the effects of the air but also the reduction in mixing water necessary to maintain the same consistency. The authors, and other writers, have suggested that air affects the strength exactly as so much water. The present data, and other data not reported herein, support the thought that such an assumption affords a good, and generally conservative, basis for design, but that the assumption is not true. Fig. 2 shows 28-day compressive strengths plotted in relationship to the gallons of water *plus* air per sack of cement. Curves, derived by interpolation from the original data, are shown for 0, 2, 4, 6, and 8 per cent of air. A family of curves is revealed, with their slopes decreasing as the percentage of air increases.

Fig. 3 shows the conventional water-cement-ratio-strength relationship for the same data as used in Fig. 2. It may be seen, for example, that concrete containing 4 per cent of air, and having a specified strength, can be obtained by reducing the water-cement ratio about 1 to 1½ gals. per sack below that required to produce the same strength in

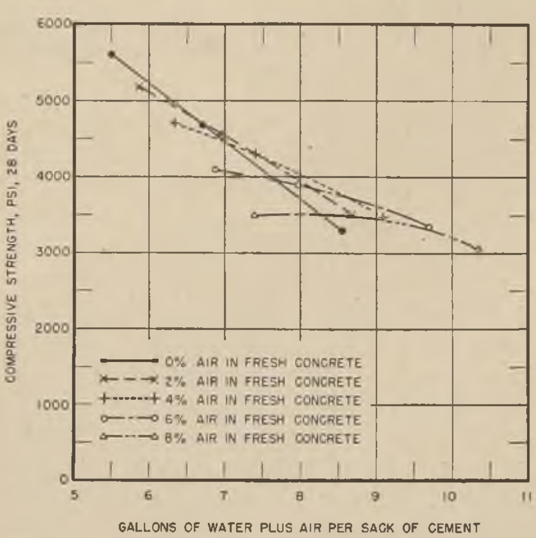


Fig. 2—Strength, water plus air relationships (series 66).

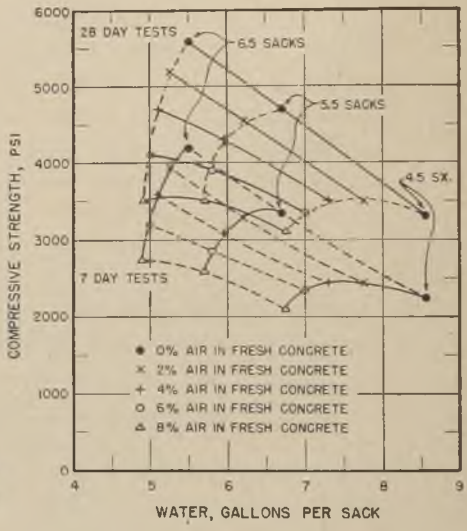


Fig. 3—Relationship between water ratio and strength for various air contents (series 66).

normal cement concrete. Penalties in cement content of from none, for the lower strengths, up to 1 sack per cu. yd., for the higher strengths, are indicated.

EFFECT OF ENTRAINED AIR ON MIXING WATER REQUIREMENTS

As pointed out, requirements for mixing water are reduced as air is entrained. The relation between the volume of air and the reduction in mixing water is so orderly that there can be little doubt but what a

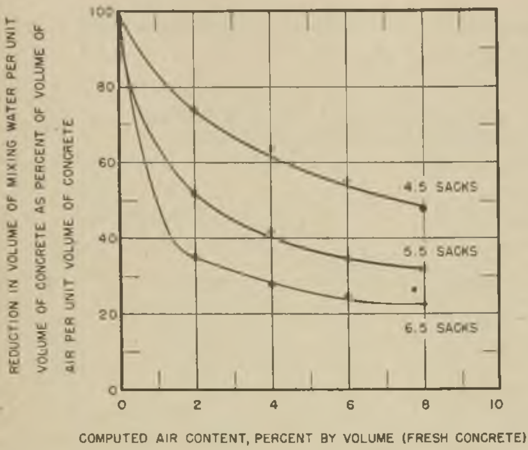


Fig. 4—Effect of entrained air on mixing water (series 66).

rational relationship exists. However, thus far, it has eluded the authors. For the same data as used in earlier diagrams, Fig. 4 shows the amount which the mixing water may be reduced for different cement contents, with the reduction expressed as a percentage of the volume of air entrained in a unit volume of concrete.

The diagram shows that the reduction in mixing water ranged, for the 4.5 sack concrete, from 74 per cent of the air for 2 per cent of air to 48 per cent for 8 per cent of air; for the 5.5·sack concrete the range was from 52 per cent to 32 per cent; and for the 6.5 sack concrete, from 35 to 23 per cent. That is to say, as more air was entrained it became progressively less efficient in reducing mixing water; also, the air was progressively less efficient as the cement content was increased.

EFFECT OF MIXING TIME

In early experiences with air-entraining cements there were some unfortunate cases of the entrainment of excessive air contents, attributed to prolonged mixing. It seems likely that most, if not all, of these cases occurred when using cements containing interground Vinsol resin which had not been pre-neutralized. More recent experiences suggest that, with air-entraining cements as now made, the danger of excessive air contents, due to prolonged mixing as in a ready-mixed concrete operation, is not a serious problem. In fact, some experiences suggest that the air content may be decreased significantly by prolonged mixing.

In view of the absence of quantitative data, tests were carried out with flake Vinsol resin, sodium resinate (N.V.X.), and Darex. The concrete was mixed for periods of 6, 12, 30, and 90 minutes. No significant differences in results were found for the three air-entraining admixtures and the average results are shown in Table 2.

TABLE 2—EFFECT OF MIXING TIME ON CONCRETE MADE WITH AIR-ENTRAINING ADMIXTURES

Mixing Time	Concrete without Admixture		Concrete with Admixture		
	Compressive Strength	Slump	Compressive Strength	Slump	Air Content
6	98	97	94	93	94
12	100	100	100	100	100
30	98	74	96	74	86
90	98	35	101	35	58

While the data are too few to permit of drawing conclusions some observations on them may be of interest. The results of the slump tests, as well as measurements of air content, suggest that, for this mixer, about 12 minutes were required for adequate mixing. After 12 minutes, both the slump and the air content were decreased by additional mixing. The slumps of both the standard-cement concrete and the air-entraining concrete were affected similarly with mixing time. This result was contrary to expectation, it being reasoned that the reduction in air content would increase the requirements for water and, for that reason alone, the slump would be reduced more when air was entrained than when the concrete was air free.

The data referred to herein, while not conclusive, suggest that, whatever the air-entraining agent, attention to mixing time is required to maintain a uniform air content, when opportunities for wide ranges in mixing time exist. However, after adequate mixing has been done, the relationship is not so critical but what little difficulty should be encountered in exercising the necessary control.

EFFECT OF GRADING OF AGGREGATE

The grading of the aggregate has been shown to affect the amount of air entrained, and this factor was investigated. Principal attention was paid to fine aggregate. Tests of both mortar and concrete were made for sand gradings ranging in fineness modulus from 1.7 to 3.6. These gradings were obtained by adding to relatively fine concrete sand appropriate quantities of a very fine sand (finer than the No. 30 sieve) and small gravel graded between the No. 8 to No. 3 sieves.

A good relationship was found between fineness modulus and amount of air entrained. The quantity of air increased from the lowest fineness

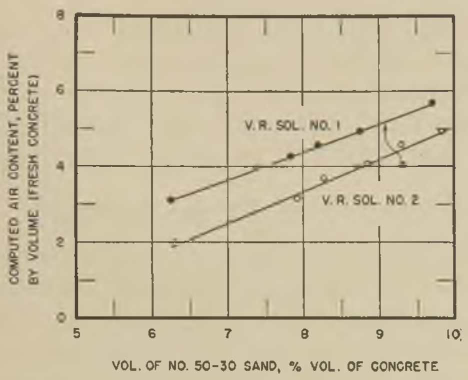


Fig. 5—Effect of grading of sand on air entrainment in concrete (series 63).

modulus to a peak at about 2.5 and, thereafter, decreased sharply. These data are not shown, since other studies suggest that the relationship is not general and that it would differ for other gradings of the same fineness moduli.

The test results were studied along lines suggested by Henry L. Kennedy in his contribution to the 1944 Symposium on Air Entrainment in Concrete*. The proportions of various grain sizes in the sand, expressed as a percentage of the volume of concrete or mortar, were studied in relation to the amount of air entrained. The most satisfactory relationship—or, in fact, the only satisfactory one—was found for air content expressed as a function of the amount of the No. 50 to No. 30 size. The data for concrete, using two different Vinsol resin solutions, are shown in Fig. 5. An equally good relationship, not shown, was found for the mortar.

Only a few tests have been made on the effect of grading of coarse aggregate, but the limited data available indicate that the coarse aggregate has little effect, except in so far as it affects the amount of sand required.

EFFECT OF TEMPERATURE OF CONCRETE

The temperature of the concrete at the time of mixing was found to have a significant effect on the amount of air entrained. By controlling the temperature of the ingredients, concrete was mixed which had temperatures, after mixing, ranging from 46 to 106 F. Vinsol resin, sodium resinate, and Darex were used as admixtures. Results of tests are shown in Fig. 6 and 7.

Fig. 6 shows that, in the case of each admixture, the amount of air entrained decreased as the temperature of the concrete increased. Fig.

*ACI Journal, June, 1944; *Proceedings* V. 40, 1944.

7 is of principal interest. In it, the amount of air is expressed as a percentage of the amount at 70 F. It is interesting that, in spite of the different admixtures and the different amounts of air entrained by the quantities of them used, all points fall on the same curve. At 50 F, the amount of air was 130 per cent and, at 100 F, 77 per cent of that at 70 F.

EFFECT OF RATIO OF SODIUM HYDROXIDE TO VINSOL RESIN

The question has been raised as to how critical is strict adherence to recommended proportions of sodium hydroxide to Vinsol resin in making up the solution for use as an admixture. Tests were made with solutions with the ratio of sodium hydroxide to the resin varied from 0.1, the usual recommendation, to 5.0. The amount of Vinsol resin was maintained at 0.009 per cent of the cement. The results of the data are shown in Fig. 8. It appears that, for the condition of these tests, the activity of the solution increased up to the point where a weight of the sodium hydroxide about equal to that of the Vinsol resin was used. Beyond this point the solution was less active, probably due, as suggested in correspondence with the Hercules Powder Company, to a "salting-out action" caused by the excessive amount of the sodium hyrdoxide.

COMPARISONS OF FRESH AND HARDENED CONCRETE

Interesting relationships between fresh and hardened concrete have been developed. The air content of the hardened concrete was computed from absolute volumes of the ingredients and specific gravity determinations of the concrete. These values were invariably about 20 per cent lower than those for the fresh concrete, also computed from absolute volumes. In search for an explanation, specific gravities of the cement were determined in water. Apparent values were found, ranging from 3.15 after 15 minutes in the water, to 3.35 after 24 hours and to 3.65 after 270 hours, when the tests were discontinued. The use of a specific gravity of cement indicated for 72 hours, about the period at which the displacement of the concrete cylinders was determined, would reduce the differences between the apparent air content of the fresh and hardened concrete to an amount explainable by losses in water in the molding process.

Consideration should be given to the question of whether or not differences in air content indicated by the gravimetric and volumetric methods may not be accounted for, in part at least, by differences in specific gravities of cement determined by conventional methods and by displacement in water.

The differences in the apparent volume of fresh concrete and hardened concrete are an indication, at least, of the shrinkage which takes place on

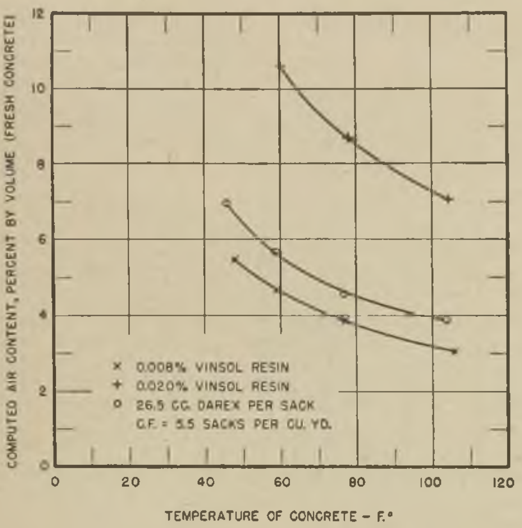


Fig. 6—Effect of temperature on air entrainment in concrete (series 78).

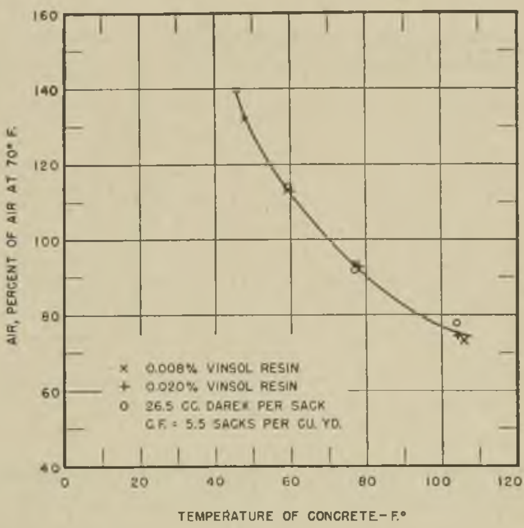


Fig. 7—Ratio of air content at different temperatures to that at 70 F. (series 78).

hardening. These differences increase significantly with increase in air content, as may be seen from Fig. 9. These data are presented to stimulate thinking without attempting to formulate conclusions.

AIR CONTENT AT DIFFERENT DEPTHS OF CONCRETE

Exploratory tests have been made on the effect of depth of section on the air content of the concrete. Columns 6 in. in diameter and 4 ft.

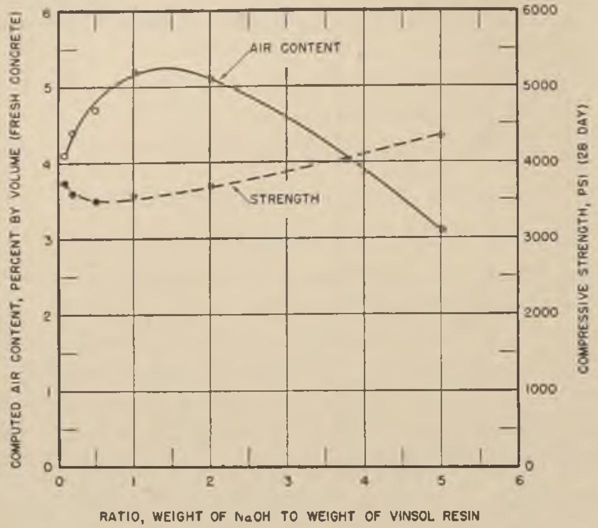
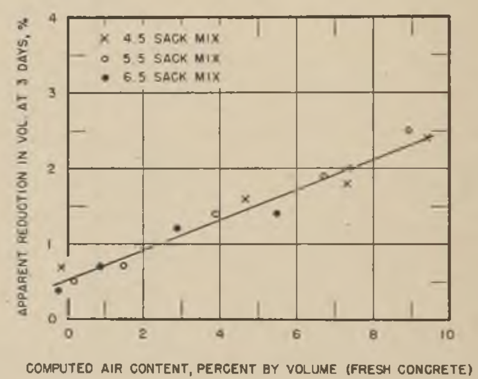


Fig. 8—Effect of amount of sodium hydroxide used to neutralize Vinsol resin on air content and compressive strength of concrete (series 79).

Fig. 9—Apparent reduction in volume of fresh concrete after 3 days (series 66).



high were molded. The concrete was placed in 12 in. "lifts" each of which was rodded 25 to 30 times as required in molding 6 by 12 in. cylinders. After the concrete had hardened, the column was broken into four pieces and the air content of these computed. There was a decrease in the indicated amount of air with distance from the top of the column. However, it appears that these differences may be satisfactorily accounted for by the compression of the air due to the weight of the superimposed concrete. That is to say, the data suggest, contrary to an earlier statement by the senior author, that there is probably no significant movement of air from the bottom to the top of the section for the method of placement used.

CLOSURE

While the tests reported are fairly limited in scope, they do lead to the following observations:

1. Entrainment of air decreased the strength of rich and moderately rich mixes but, in reasonable quantities, increased the strength of lean mixes.

2. Entrainment of air reduced requirements for mixing water. As a substitute for mixing water in maintaining consistency, air was progressively less efficient as the amount of air was increased and as the amount of cement was increased.

3. The amount of entrained air was decreased by prolonged mixing; the slump of air-free concrete and concrete containing entrained air was similarly affected by mixing time.

4. The amount of air entrained was significantly affected by the grading of the fine aggregate and appeared to be principally a direct function of the amount of the No. 50 to No. 30 grain size.

5. More air was entrained by concrete of low temperature than by concrete of high temperature.

6. Increasing the ratio of sodium hydroxide to Vinsol resin up to 1 resulted in increased air entrainment.

7. The differences between apparent volumes of fresh and hardened concrete increased with amount of entrained air.

8. The volume of air at different depths of concrete was affected by the weight of the superimposed concrete.

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Title 42-29 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Homogeneity of Air-Entraining Concrete*

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SYNOPSIS

Reports studies of the homogeneity of air-entraining concrete by means of a test for resistance to abrasion. An air-entraining agent was used with no added accelerators, dispersing agents or gas forming materials. "Since air entrainment has a tendency to eliminate bleeding of concrete it is only reasonable to believe that such concretes are more homogeneous . . . . the top surface (of a slab) would have the same strength and density characteristics as the bottom surface." Plotted data show that there is progressive increase in abrasion loss in specimens as air content increases beyond 6 per cent.

Our most recent work in connection with air-entraining agents has involved a study of the homogeneity of air-entraining concrete, utilizing abrasion resistance as a criterion. A common, everyday air-entraining agent was used for these studies—no added accelerators, dispersing agents, or gas-forming materials.

Since air entrainment has a tendency to eliminate bleeding of concrete, it is only reasonable to believe that such concretes are more homogeneous. For example, the top surface would have the same strength and density characteristics as the bottom surface.

Fig. 1 was made in connection with our earlier studies of the homogeneity of high and low bleeding concretes with no air entrainment. Reduced bleeding is accompanied by an increase in homogeneity as measured by compressive strengths.

Tests of air-entraining concretes indicate that there is little difference between the homogeneity of an air-entrained concrete and a relatively high-bleeding, normal concrete, despite the fact that in the case of air-

*Presented at Air Entrainment Session, 42nd ACI Convention, Feb. 19, 1946.
†Dewey & Almy Chemical Co., Cambridge, Mass.

entraining concrete the bleeding is reduced greatly or eliminated altogether. The range in strength and density of many low-bleeding air-entraining concretes has been found to be practically the same as in higher bleeding concretes in the absence of air-entrainment.

A lower density on the upper surface of a concrete pavement would indicate that the surface was more resistant to freezing and thawing, and hence the apparent lack of homogeneity is a favorable factor. In reinforced concrete, however, particularly continuous beams, where the negative reinforcing is near the top surface of the beam, this situation will demand more study. It represents a probable explanation for differences in bond strength between top and bottom of a beam made with air-entraining concrete.

As a means of studying homogeneity, the Dewey & Almy Chemical Co. Laboratories have developed an abrasion test which seems to bring out the differences in relatively shallow slabs, differences which are not readily discernable in strength and density tests.

The apparatus used is a Reumelin sand blast cabinet, except that iron filings are used instead of sand. The air pressure is held constant at 60 psi. The specimen is set 1½ in. away from the nozzle and the abrasion applied for a total of twenty seconds, over an area 1¼ in. by 1½ in. Eight such tests are made on each surface by use of a template.

Fig. 2 shows the effects of abrasion on the top, side and bottom of a cast specimen of an air-entraining concrete containing 10 per cent air.

The figure shows a discernable difference between the resistance to abrasion between the top, side and bottom of the specimen. The actual weight loss in grams, due to abrasion, were as follows: Top 48; Side 33; Bottom 17. On the side view the top of the specimen as cast is to the left. Note the difference in abrasion between the top and bottom of slab even in a 4 in. depth.

Actually, densities were not measured at the top and bottom of this beam. It would have been difficult to discern the difference in density, but the difference in abrasion loss indicates a difference in the concrete from top to bottom as cast.

The abrasion loss noted on the corresponding blank specimen, having a total air content of 0.6 per cent, was: Top 24; Side 13; Bottom 12.

The significant figures are those represented by the abrasion loss on the side of the specimen, as compared with the bottom of the specimen, since both were cast against a steel form. The top of each specimen was simply trowelled smooth. Hence the top of one specimen may be compared with the other, but the homogeneity should be evaluated only on the basis of side and bottom of specimen.

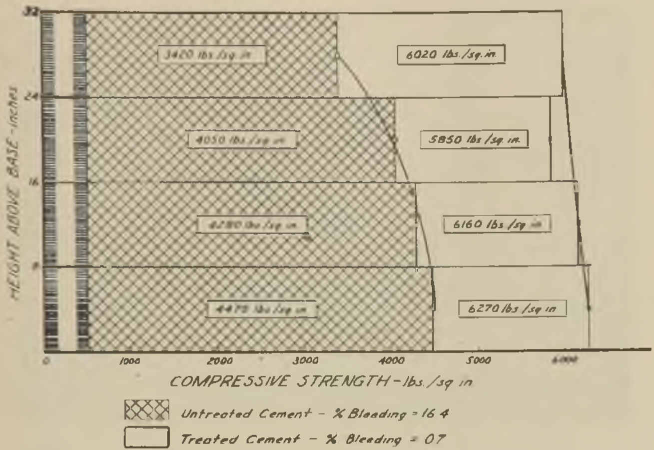
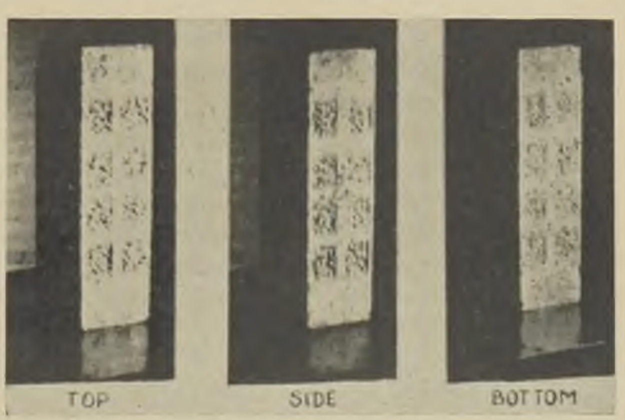


Fig. 1—Air entrainment tends to increase hemogeneity in concrete.

Fig. 2—Abrasion on top, side and bottom of cast specimen — 10 percent air.



It will be noted that in the case of the blank there is practically no difference in abrasion loss between the bottom and the side of specimen, whereas in the case of the air-entraining concrete, the abrasion loss on the side was practically twice that of the bottom.

Fig. 3 showing these same four concretes, presents an interesting study of the danger of not controlling air and cement content. It shows the top surfaces of the four concretes after the abrasion tests had been made. The differences are so great as to be easily noted merely by a visual inspection. The specimen on the left is a plain concrete having somewhat less than 1 per cent of air. The second specimen made with a straight

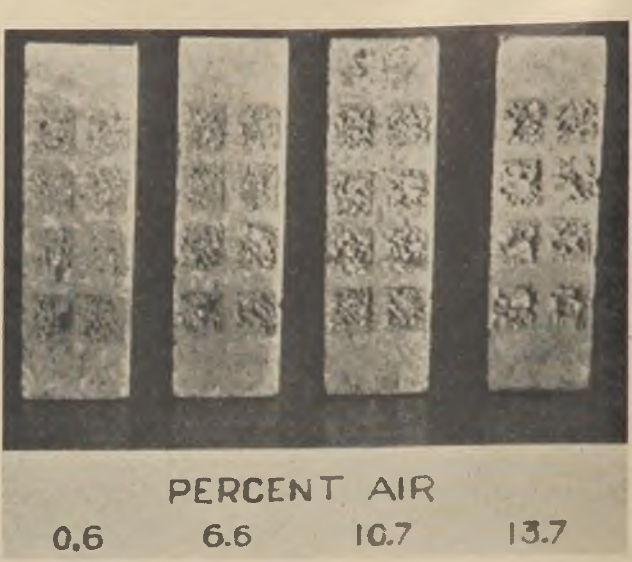


Fig. 3 — Abrasion tests of specimens with different percentages of air.

air-entraining agent contains 6 per cent, the third 10 per cent, and the last 13 per cent of air.

It will be noted that there is a progressive increase in the abrasion loss as the air content increases. It will also be noted that when the air content is kept below 6 per cent and cement content held constant, the abrasion loss is very little more than that obtained without the entrained air. It is suggested, however, that entrained air of over 10 per cent is actually dangerous from the standpoint of abrasion, even though the concrete would be quite remarkable from the standpoint of resistance to freezing and thawing.

Incidentally, this project involves approximately 1200 test specimens and one year's laboratory work. The specimens are being tested under various conditions of mix, cement content, air content and curing.

All of this work points the same way—do not be deceived by the very fatty (whip cream) mixes—be suspicious of these, because it has been demonstrated time and again that they can be an indication of excess air and may give concretes low in strength and abrasion resistance. Hold and control the air to a maximum of 6 per cent. Check and insist on maintenance of cement factor. The plasticity at this air content will be very satisfactory, as will be strength and other desirable properties.

To facilitate selective distribution, **separate prints** of this title (42-30) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946

Title 42-30 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Methods of Entraining Air in Concrete*

By E. W. SCRIPTURE, Jr.†
Member American Concrete Institute

SYNOPSIS

Discusses methods for and mechanisms of air entrainment in concrete mixes. Methods include use of aluminum and hydrogen peroxide for entrainment of hydrogen or oxygen, respectively; use of cement dispersing agents; perhaps protective colloids. Data are reported to record the relation of air content to durability as determined by freezing and thawing.

From a practical point of view it might not seem important to know the mechanism whereby air entrainment is effected, especially since it does not appear to make much difference, as far as durability is concerned, how the air is incorporated in the concrete. Hope for improvement of methods and materials for air entrainment and for control of this phenomenon, however, should lie in a knowledge of the mechanism or mechanisms. There appears to be a tendency to think only in terms of one means of air entrainment, that of employing surface active compounds which lower the surface tension of water, known variously as wetting or foaming agents, and including soaps. The mechanism whereby these compounds incorporate air in the concrete mix is well-known and has been described in several places in the literature. That this is the only air entrainment mechanism is manifestly incorrect and it is by no means certain that it is the most satisfactory.

Aluminum and hydrogen peroxide have been used to incorporate air, or rather hydrogen and oxygen, respectively, in concrete mixes. These function by generating the gases in situ by reaction with constituents of the cement and bear no relation to surface tension reducing compounds.

There is still a third known method of introducing air, above the normal amount, into concrete and that is by use of a cement dispersing

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Director of Research, The Master Builders Co., Cleveland, Ohio.

agent. These are surface active chemical compounds which are preferentially adsorbed by cement, endowing the cement particles with electrostatic charges which make them mutually repellent. These compounds do not lower the surface tension of water to a marked degree and do not form stable foams with water alone, although what they may do in a cement suspension is something else. They are definitely not wetting or foaming agents and would not be applicable to those uses of wetting or foaming agents which depend on surface tension reduction. The mechanism whereby they entrain air in concrete is evidently not the same as that of foaming agents although just what that mechanism is may be open to some question. It may be suggested that it is related to increased effective surface area of the cement and the finer effective size of the cement particles in the dispersed state.

While these three methods of entraining air are the best known, it by no means follows that they are the only ones. For example, the use of protective colloids may offer some possibilities. Solving the problem of air entrainment may lie in an understanding of how it works.

Undoubtedly cement dispersing agents normally entrain some additional air. A curious phenomenon in this connection is that, unlike the wetting agents, the amount of air entrained with a fixed ratio of dispersing agent to cement appears to be relatively constant regardless of the mix or the brand of cement used, within reasonable limits. Data on a series of 39 mixes using 13 different cements ranging from 4½ to 6 sacks of cement per cu. yd. and including slumps from 2 to 6 in. are shown in Table 1. It will be noted that for each type of mix the averages for the 13 cements range only from 3.1 per cent to 3.8 per cent of air. The extreme variation with the different cements and the different mixes is from 2.0 to 5.0 per cent but only 7 air contents are over 4 per cent and only 3 below 2.5 per cent, and only 7 others below 3.0 per cent. It may be doubted that the extreme values are significant. It is not suggested that

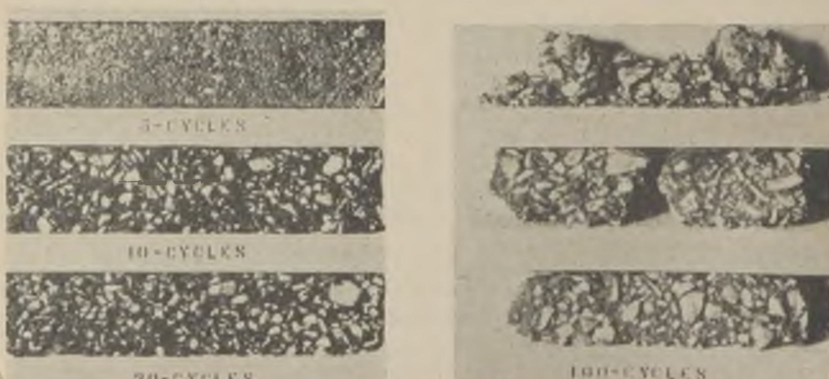

Fig. 1—Control Mix

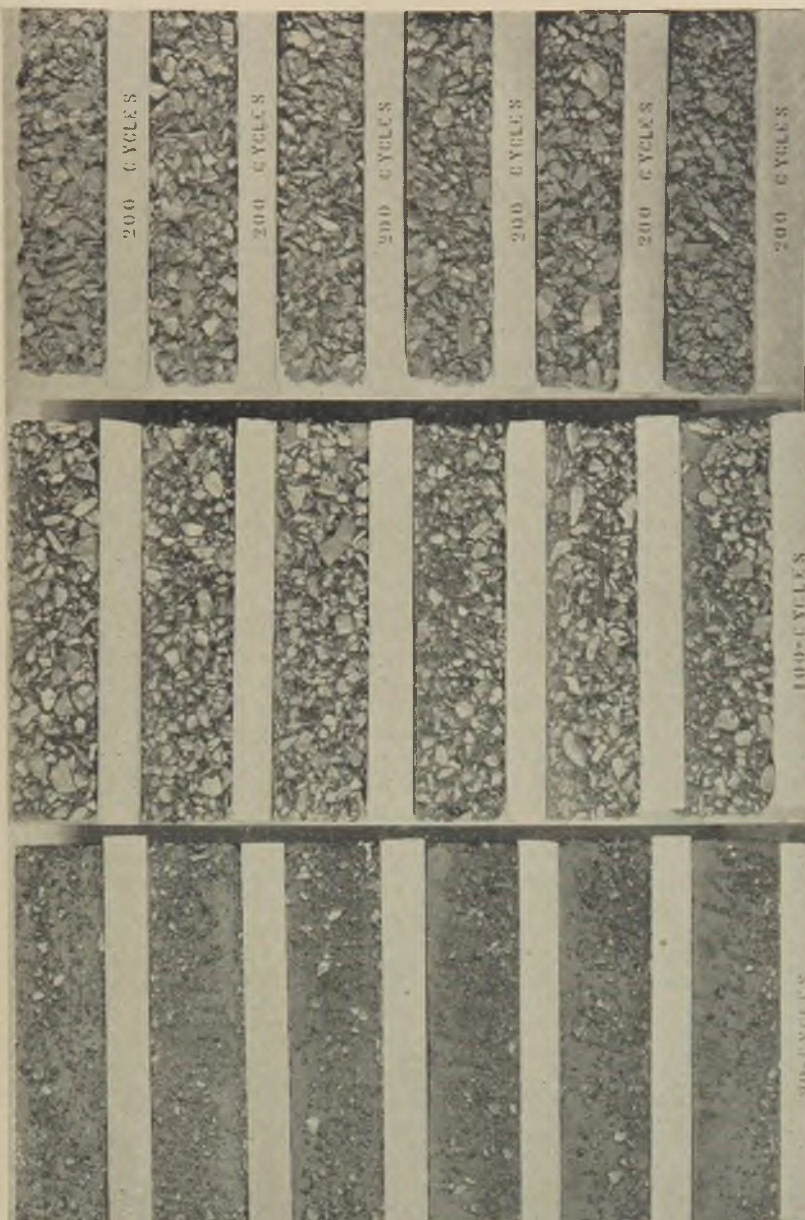

Fig. 2—3.5 per cent air—20 cycles.

Fig. 3—3.5 per cent air—100 cycles

Fig. 4—3.5 per cent air—200 cycles.

TABLE 1

Cement	Total Air Content—Per cent by Volume		
	6 Sack Nominal Mix		4½ Sack Nominal Mix
	Slump 3½ in.	Slump 6 in.	Slump 2 in.
A	3.7	4.0	3.5
B	3.5	3.6	2.9
C	3.8	3.7	3.8
D	3.7	4.6	3.4
E	2.8	3.1	2.0
F	2.7	3.2	2.9
G	3.3	4.4	4.3
H	2.4	2.8	3.0
I	3.3	3.7	2.9
J	3.6	4.4	4.5
K	2.7	3.0	2.4
L	3.9	3.8	3.5
M	3.8	5.0	4.1
Average	3.1	3.8	3.3

TABLE 2

Addition	S/A Ratio per cent	W/C gal. per sack	C.F. Sk. per cu. yd.	Total Air per cent	Comp. Str. 6 mo. Lb. per sq. in.
None	40	6.01	6.01	0.2	6870
Dispersing Agent + Foaming Agent	36.5	5.16	5.98	3.6	7265
Foaming Agent	36.5	5.61	5.95	3.4	6257
Dispersing Agent	36.5	5.20	6.00	3.5	7460
Dispersing Agent + Foaming Agent	40	5.36	5.97	3.5	7511
Foaming Agent	40	5.66	5.91	3.7	6270
Dispersing Agent	40	5.31	5.97	3.7	7612

it is not possible to vary the air content, especially by changing the sand-total aggregate ratio, when using a cement dispersing agent, but that with a fixed quantity of the cement dispersing agent per sack of cement a very constant air content, in the range generally considered most desirable, will be secured automatically in most normal mixes.

To determine the relation of durability to air content, regardless of the means used for air entrainment, a series of mixes of equal cement factors was made up to approximately constant air content, except for the control mix without addition, and subjected to freezing and thawing. The mix data, arranged in the order of illustrations, (Fig. 1 to 4) are given in Table 2. It may be of interest to note that the six-month strengths of the dispersing agent or dispersing plus foaming agent mixes average about 9 per cent higher than that of the control mix, although the air content has been increased from 0.2 to 3.5 per cent, whereas the foaming agent mixes exhibit about an equal loss in strength. The course of disintegration is shown in Fig. 1 for the control mix at 5, 10, 20, and 100 cycles and in Fig. 2, 3, and 4 for the mixes containing approximately 3.5 per cent air, however attained, at 20, 100, and 200 cycles.

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Title 42-31 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Effect of Air Entrainment on Stone Sand Concrete*

By A. T. GOLDBECK†
Member American Concrete Institute

SYNOPSIS

A limestone sand which had been used with indifferent success in pavements and other structures, prompted a series of tests by National Crushed Stone Association, to improve workability and durability of concrete in which this aggregate was used. Results with and without Vinsol resin as an air-entraining agent were favorable to the use of the admixture. Data reported include materials, mix design, freezing and thawing tests.

The following tests were made as a portion of a series of tests in an effort to improve the workability and durability of concrete made with a particular limestone sand which had been used in concrete structures and pavements with indifferent success. Previous tests included studies of the effect of changing the shape of particle, the effect of cement factor and investigations to determine the effect of the addition of different percentages of limestone dust.

The present report covers only what might be considered the successful portion of the investigation, namely, that involving the use of Vinsol resin as an air-entraining agent.

Six mixes were made; one with Potomac River sand with no Vinsol resin added, and five with limestone sand having Vinsol resin contents of 0.0, 0.009, 0.013, 0.017, and 0.025 per cent based on the weight of cement in the mixture. The Vinsol resin was dissolved in a 3 per cent solution of sodium hydroxide and this solution was added to the mixing water in the above amounts.

MATERIALS

Cement

One brand of type 2 cement was used, with a constant cement factor of 6 bags per cu. yd. in all of the mixes.

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Engineering Director, National Crushed Stone Association, Washington, D. C.

Sand

The natural sand used in one of the mixes for comparison purposes was from the Potomac River and its physical characteristics were as follows:

Bulk specific gravity 2.61
Weight per cu. ft. solid, lb. 162.9
Absorption, per cent 1.0
Gradation:

Sieve No.	Total per cent passing
4	100
8	85
16	75
30	50
50	15
100	4

Fineness Modulus = 2.76

The characteristics of the limestone sand were as follows:

Bulk specific gravity 2.64
Weight per cu. ft. solid, lb., ... 164.7
Absorption, per cent 1.1
Gradation:

Sieve No.	Total per cent passing
4	100
8	91
16	64
30	36
50	18
100	8

Fineness Modulus = 2.83

Shape of stone sand particle

A measure of the shape of stone sand particle was obtained in the manner described in N.C.S.A. "Useful Information" No. 2-28. It consists in determining the percentage of voids in a loose condition of the No. 8-16, No. 16-30, and No. 30-50 sieve fractions. The higher the percentage of voids, the more angular is the shape of particle. As determined by the above mentioned method, the percentage of voids equals 51.5. The indications are that this sand has a fairly cubical shape of particle.

When sand is prepared by special means, such as the Kent mill or the rod mill, it is possible to reduce the percentage of voids to about 50.0. On the other hand, stone screenings with no processing will ordinarily run about 53.0.

Coarse aggregate

A crushed limestone coarse aggregate with the following characteristics was used in these tests:

Source—West Virginia
Bulk specific gravity 2.72
Weight per cu. ft. solid, lb. 169.7
Weight per cu. ft. dry, rodded, lb. 103.7
Absorption, per cent 0.3

Gradation:

Sieves	Total per cent passing
2 in.	100
1½ in.	98
1 in.	67
¾ in.	52
½ in.	30
No. 4	2
No. 8	0

Design of concrete mixes

All mixes were designed in accordance with the method given in N.C.S.A. *Bulletin No. 11*, or *Stone Briefs No. 2*. The cement factor was held at 6 sacks per cu. yd. for all of the mixes and since the sand had a fineness modulus of about 2.80, the value for b/b_o, was taken at 0.77. An estimate was made of the volume of air entrained in the mixtures and this volume was considered as a component of the concrete. A slump of 2 to 3 in. was maintained.

Mixing, molding and testing

The concrete was mixed for 5 min. in a tilting drum, fixed blade type of mixer of approximately 3 cu. ft. capacity. One beam and two cylinders were cast from each mix. Twenty-four hours after molding, the specimens were removed from the molds and placed in the moist room where they were cured under standard conditions of moisture at 70 F temperatures until they were tested at the age of 28 days. Two compression and two flexural tests were made from each mix. Each beam was tested twice.

Freezing and thawing tests

Owing to the limited capacity of the freezing chamber, specimens were prepared from the broken ends of beams by sawing them to the dimensions of 3 x 3 x 12 inches. Gage plugs were set on 10-inch centers and a Whittemore strain gage was used to measure the permanent set after a number of alternations of freezing and thawing, so that curves could be drawn showing the expansion which took place as the result of freezing. Permanent expansion is a measure of the durability of concrete and it gives indications of durability parallel with those shown by dynamic modulus of elasticity. The specimens were kept completely immersed in water during both the freezing and the thawing periods.

Results of the tests

A summary showing the average of the three mixes is as shown in Table 1.

Fig. 1 shows the effect of different percentages of air entrainment on both the compressive strength and the modulus of rupture of the stone sand concrete. Fig. 2 shows the effect of Vinsol resin and air entrainment

TABLE I—AVERAGE VALUES OF THREE MIXES

Type of Fine Aggregate	Natural Potomac R. Sand	Stone Sand				
		A	B	C	D	E
Vinsol resin, percent	0.000	0.000	0.009	0.013	0.017	0.025
Parts per unit vol.						
Cement	.106	.106	.106	.107	.106	.107
Water	.156	.163	.158	.155	.151	.146
Air	.012	.011	.021	.026	.040	.065
Sand	.257	.250	.244	.239	.232	.210
Stone	.469	.470	.471	.473	.471	.472
Total	1.000	1.000	1.000	1.000	1.000	1.000
Slump, in.	2.6	2.5	2.4	2.4	2.5	2.8
Flow, per cent	71.	64.	61.	62.	69.	67.
Cement factor, sacks per cu. yd.	5.98	6.00	6.01	6.04	6.00	6.02
Entrained air, per cent	1.2	1.1	2.1	2.6	4.0	6.5
Sand, per cent	35.5	34.7	34.2	33.6	33.0	30.9
W/C by volume	.70	.73	.71	.69	.68	.66
Ratio (vol. water + vol. air to apparent vol. cement)	.76	.77	.81	.81	.86	.95
Compressive strength, psi	5950	5310	5250	5090	4890	4090
Modulus of rupture	860	870	830	810	805	685
Freezing and thawing tests (specimens from third set of mixes)						
Permanent expansion in. per in., 150 cycles	Failed	Failed	.00074	.00049	.00044	.00039

on the unit expansion which took place as the result of cycles of freezing and thawing.

The significant results of this series of tests can best be observed by studying the two charts. It will be noted that air entrainment produces a reduction in both the compressive strength and in the modulus of rupture. Assuming that the curves of strength shown in the chart fairly represent the average values, the effect of air entrainment on strength is shown in the following table.

Air	Compression		Modulus of Rupture	
0 per cent	5300	100 per cent	860	100 per cent
3 per cent	5000	94.5 per cent	840	98 per cent
5 per cent	4600	87 per cent	790	92 per cent

It will be noted that with 5 per cent of air entrainment which is frequently taken as a desired maximum, there is a reduction of 13 per cent in compressive strength and 8 per cent in modulus of rupture. The strength values in both compression and modulus of rupture are very high. Referring to the chart showing unit expansion with different cycles of freezing and thawing, it will be noted that all of the specimens containing air produced through the use of Vinsol resin, show very little expansion and, in fact, the expansion of specimen 3B with only 2.1 per cent of air in it, was no greater at 150 cycles than in the specimen containing no Vinsol resin, measured at less than 10 cycles.

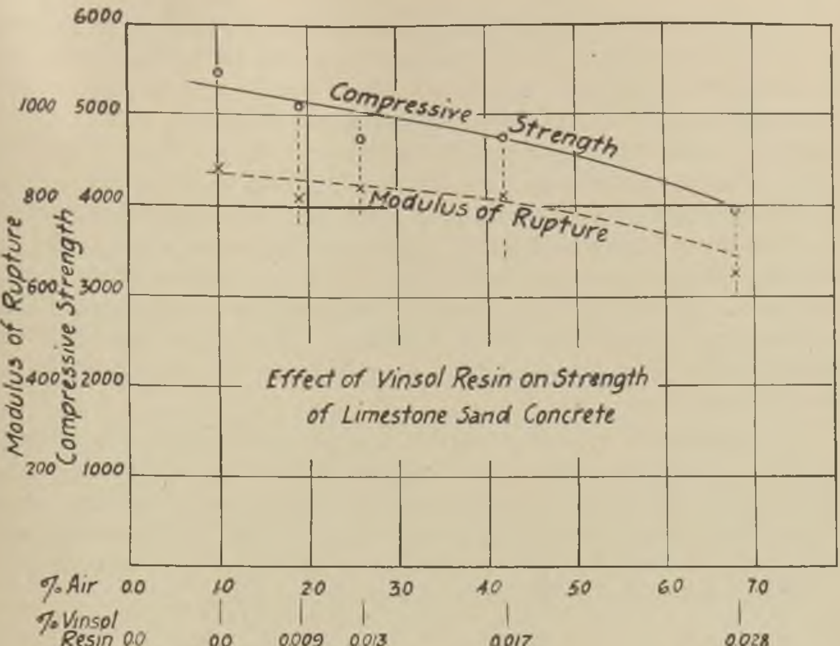

Fig. 1—Effect of Vinsol resin on strength of limestone sand concrete

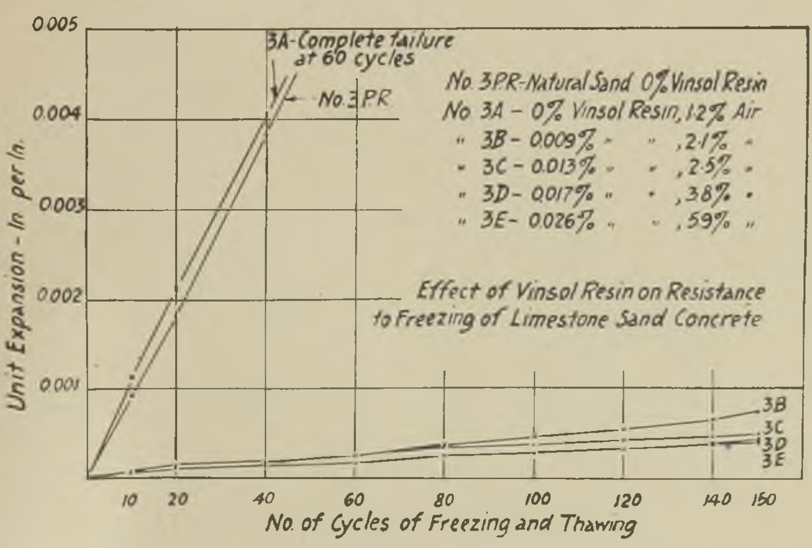

Fig. 2—Effect of Vinsol resin on resistance to freezing of limestone sand concrete

The effect of the Vinsol resin inclusion on the workability of the concrete was extremely marked. Even specimen 3B, with only 2.1 per cent of air and the minimum amount of Vinsol resin, was very plastic and, furthermore, this mix showed practically no water-gain. This was in marked contrast with specimen 3A which contained no Vinsol resin. All of the specimens showed surface scaling at the end of 150 cycles, but the specimens containing no Vinsol resin were unsound as indicated by their partial breaking apart and by the looseness of the coarse aggregate in the matrix. Furthermore, they had a very dead sound when struck with a hammer. On the other hand, all of the specimens containing Vinsol resin were in an entirely sound condition. The coarse aggregate was tightly bound and when struck with a hammer the specimens had a metallic ring.

Air entrainment evidently will not entirely prevent surface scaling of concrete upon severe exposure, but it does effect a very material improvement in the durability of concrete and in its resistance to scaling. It is believed that these tests indicate very strongly that the limestone sand used will make very durable concrete, provided from 3 to 5 per cent of air is entrained by the use of an air-entraining agent.

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Title 42-32 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

A Method for Direct Measurement of Entrained Air in Concrete*

By W. H. KLEIN† and STANTON WALKER‡
Members American Concrete Institute

SYNOPSIS

Since the amount of air entrained in concrete is of major importance and the methods now in use to determine that amount have inherent objections, the pressure method, by application of Boyle's law, has advantages. The Klein air meter is described, following tests by Pennsylvania-Dixie Cement Corp. and National Ready Mixed Concrete Association. The paper presents a description, with illustrations, of the Klein air meter, the test procedure, calculation of air content and calibration of meter and presentation of data on the use of the method.

The amount of air in concrete is of major importance when air-entraining agents are used. If too little is entrained the potential benefits are not realized; if too much, serious reductions in strength and other disadvantages result. Methods in general use for the determination of the quantity of entrained air in concrete have inherent in them certain objections—either from the points of view of accuracy or of convenience. This paper describes a procedure for the direct measurement of air which offers promise of overcoming some of these objections.

Two general methods for measuring air in concrete have been used. The most convenient procedure, and the one best adapted to field use, is patterned after that described in A.S.T.M. Method C 138—*Standard Method of Test for Weight per Cubic Foot, Yield, and Air Content (Gravimetric) of Concrete.* By it, the air-content is calculated from knowledge of the weight per unit of volume of the concrete and the quantities and specific gravities of the ingredients. The second method depends on the determination of the weight of concrete per unit of volume and on the displacement in water of a weighed sample of the concrete, after elimi-

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Vice-President, Pennsylvania-Dixie Cement Corp.
‡Director of Engineering, National Ready Mixed Concrete Association.

nation of the entrained air from the sample while immersed in water. A specific procedure applying that principle is described in A.S.T.M. Method C 173, *Tentative Method of Test for Air Content (Volumetric) of Freshly Mixed Concrete.* The method described herein permits direct determination of air content without the necessity of knowledge of any characteristics of the batch.

The gravimetric method depends upon the accuracy with which the required data are known. If the proportions of ingredients differ from those assumed, an error is introduced; a principal source of error is lack of precise knowledge of the specific gravities and moisture content of the ingredients of the batch. Small errors in these factors lead to relatively large errors in the results.

The volumetric method is tedious of execution and, in addition to common errors of measurement, is subject to such errors as may result from failure to remove all of the entrained air. The authors believe it, however, to be basically sound.

PRESSURE METHOD FOR DETERMINING AIR

The method described in this paper depends on a simple application of Boyle's Law. Neither the weight per unit of volume nor the proportions or specific gravities of the ingredients are required. Pressure is applied to a known volume of concrete and the reduction in volume measured. Since the air entrained in the concrete is the only significantly compressible ingredient, whatever reduction in volume results is due to its compression. From a knowledge of the pressure applied and the reduction in volume, the amount of air can be calculated readily.

The idea for the procedure originated with the senior author. On being discussed with the junior author it aroused his interest and the desire to investigate its possibilities. Exploratory work was done in both the Central Laboratory of the Pennsylvania-Dixie Cement Corporation, Bath, Pa., and the Research Laboratory of the National Ready Mixed Concrete Association at the University of Maryland, College Park, Md. At Bath, the work was done by L. C. Hawks, Chief Chemist; at Maryland by D. L. Bloem, Associate Research Engineer.

At the Central Laboratory of the Pennsylvania-Dixie Cement Corporation an apparatus of one liter capacity was constructed. Pressure-method tests were made on mortar and, also, on concrete with aggregate up to ¾ in. maximum size. For mortar, results were compared with determinations made in accordance with Tentative Method of Test for Air Content of Portland Cement Mortar (A.S.T.M. Designation, C 185-44T); for concrete the comparisons were with Tentative Method of Test for Air Content (Volumetric) of Freshly Mixed Concrete (A.S.T.M. Designation, C 173-42T). Satisfactory agreement was found in both cases. A large number of routine tests have been made comparing the pressure method with the two standard methods, also with satisfactory agreement.

At the University of Maryland tests were conducted with improvised equipment and with especially constructed apparatus of one-half cu. ft. capacity, suitable for concrete, and described later. Comparisons were made with the gravimetric method with satisfactory results. The apparatus and tests described in this paper are based on the work done at the University of Maryland.

The tests conducted were not extensive. However, they were adequate to convince the authors that the method offers definite promise and deserves of description to permit others to compare it with currently used procedures. It is hoped that such work may lead to a standardized procedure which will serve, at the least, as a basis for "calibrating" the convenient gravimetric field method.

THE KLEIN AIR METER

Fig. 1 is a schematic layout of the appratus used in the investigation. It consists essentially of a source of air pressure (the Pressovac pump), a pressure gauge (the mercury manometer) and a container identified as the Klein Air Meter. The "surge tank," consisting of a standard glass carboy, is incidental, being required because of the over-capacity pump used. In the field, pressure would be applied by some such simple device as an ordinary tire pump, which has been used with complete satisfaction. For the pressure gauge, there would be substituted for the somewhat clumsy manometer one of several standard dial gauges of suitable sensitivity, which are readily available on the market.

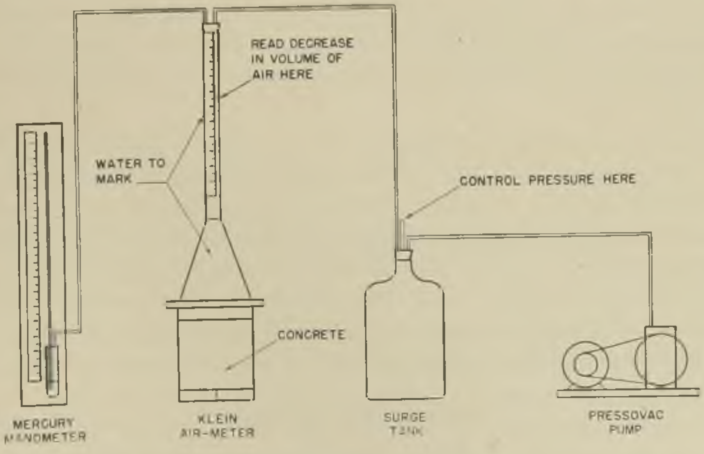

Fig. 1.—Schematic layout of Klein Air-Meter in operation in laboratory.

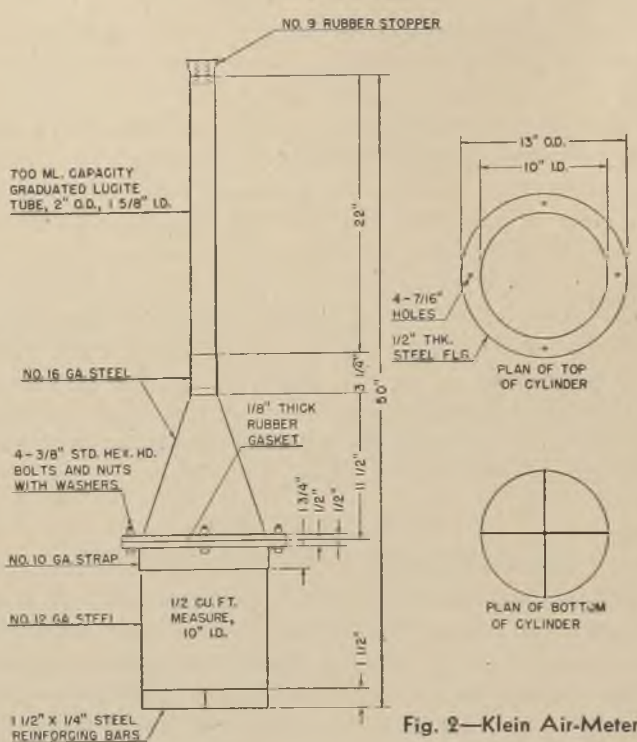

Fig. 2—Klein Air-Meter

Fig. 2 shows the essential details of the air meter. The one illustrated is a purely experimental model manufactured under the authors' direction by C. A. Hogentogler, Jr. It lacks a number of refinements which would be built into any subsequent model. It consists of a standard ½ cu. ft. measure (see A.S.T.M. Method C 29) on which an assembly consisting of a metal truncated cone equipped with a graduated lucite tube is mounted, with a suitable rubber gasket to make the connection water-tight.

Among the refinements which would be incorporated in later models are a drain-cock near the base of the truncated cone and a threaded top, with suitable outlets, to replace the indicated rubber stopper. A source of pressure and a pressure gauge would also be furnished. Further, it seems likely that a more convenient method for attaching the cone assembly to the measure, than the bolts shown, can be devised. While this apparatus would permit of more detailed description, what has been said should be sufficient to make clear its fundamental features.

TESTING PROCEDURE

The procedure for making the test may be described briefly as follows:

(1) Fill the ½ cu. ft. measure with concrete in a standard fashion (see A.S.T.M. Method C 138).

(2) To avoid turbulence (the source of which will become clear later) place on top of the concrete a glass plate or a thin rubber sheet, or some similar device, which is slightly less in diameter than the measure.

(3) Attach the cone and graduated tube assembly in such a manner as to be assured that the connection is water-tight.

(4) Introduce water into the cone assembly to the zero mark on the graduated tube. To avoid turbulence and the inclusion of casual air, the water should be introduced slowly through a hose, the end of which is near the covered surface of the concrete in the container. (In a later model, the water would probably be introduced through the drain cock).

(5) Cap the tube with a stopper having two outlets, one to a source of pressure and the other to a pressure gauge.

(6) Apply pressure and read the amount of pressure applied and the reduction of volume of water in the graduated tube. (In our practice, pressures have been restricted to various values up to about 40 centimeters of mercury, or a little more than ½ atmosphere).

That is all there is to the testing procedure, and experience with it demonstrates that it works for a wide range of mortar and concrete mixtures. Typical data will be discussed later.

CALCULATION OF AIR CONTENT

As stated previously, from a knowledge of the reduction in volume, as determined by reading the graduated tube, the volume of air can be calculated by the straight-forward application of Boyle's Law. With certain minor reservations of no interest here, that law is that the volume occupied by a given mass of any gas at constant temperature varies inversely as the absolute pressure to which it is subjected. Accordingly we can write,

$$\frac{V}{V-r} = \frac{P_2}{P_1} \qquad \text{(1)}$$

where,

V = volume of air in the concrete

r = reduction in volume due to pressure

P_1 = initial absolute pressure

P_2 = final absolute pressure

The initial pressure consists of the barometric pressure plus pressure incidental to the procedure and the final pressure consists of the gauge pressure plus the initial pressure and other pressure incidental to the procedure. It seems worth while to derive the applicable formulae; for that purpose assume the following nomenclature:

V_1 = Volume of air in fresh concrete in container.
V_2 = Volume of air in fresh concrete in container after water has been brought to mark in cone assembly.
V_3 = Volume of air in fresh concrete in container after application of pressure.
V_4 = Volume of concrete in container.
h_1 = Average pressure exerted by concrete (taken as head of concrete to mid-height of measure).
h_2 = Pressure exerted by column of water brought to mark in cone assembly.
h_3 = Pressure exerted by column of water in cone assembly after application of pressure (h_3 is less than h_2, closely enough, by the amount which the height of the column of water is reduced in the calibrated tube when pressure is applied.)
B = Barometric pressure.
P = Gauge reading on application of pressure.
R = Reduction in volume of water in calibrated tube on application of pressure P.

Then,

$$\frac{V_1}{V_2} = \frac{B + h_1 + h_2}{B + h_1} \qquad (2)$$

$$\frac{V_2}{V_3} = \frac{P + B + h_1 + h_3}{B + h_1 + h_2} \qquad (3)$$

$$V_3 = V_2 - R \qquad (4)$$

Combining equations 2, 3, and 4, we have,

$$V_1 = \frac{R(B + h_1 + h_2)\ (P + B + h_1 + h_3)}{(B + h_1)\ (P - h_2 + h_3)} \qquad (5)$$

The percentage of air is, of course,

$$A = 100 \frac{V_1}{V_4} \qquad (6)$$

CALIBRATION OF AIR METER

While barometric pressures vary sufficiently from locality to locality (for example, New York to Denver) so that they should be taken into account, the variation in any one general locality is not likely to be sufficient to cause appreciable error. Consequently, if a mean barometric pressure is assumed and an applied gauge pressure is selected, the tube may be calibrated to read percentages of air directly. Further, this same calibration can be made to apply for other barometric pressures without appreciable error if appropriate applied pressures are selected.

Equations 5 and 6 can be combined and written,

$$R = \frac{A\ V_4\ (B + h_1)\ (P - h_2 + h_3)}{100\ (B + h_1 + h_2)\ (P + B + h_1 + h_3)} \qquad (7)$$

$$A = \frac{100R}{V_4}\ \frac{(B + h_1 + h_2)}{(B + h_1)}\ \frac{(P + B + h_1 + h_3)}{(P - h_2 + h_3)} \qquad (8)$$

Equation 8 has been written to indicate the most convenient subdivision of steps in making calculations. Knowing the cross sectional area of the tube (C), R can be expressed in terms of h_2 and h_3. For illustration, expressing all dimensions in centimeters and all pressures in terms of mercury, which has a specific gravity of 13.6, we have,

$$R = 13.6\ C\ (h_2 - h_3) \qquad (9)$$

From a knowledge of the dimensions of the air meter, R can be calculated for selected percentages of air A for fixed values of P and B, and thus the graduations on the tube can be made to show percentages of air directly. For any other barometric pressure B than that assumed in the preceding, calculations based on these equations will show that the same calibration can be maintained, without significant error, by using another value for gauge pressure P.

At the risk of belaboring this point, let us, for illustration, calculate R for this apparatus for 5 and 10 per cent of air, when the applied gauge pressure is 40 cm. and the barometric pressure is, closely enough, 76 cm. of mercury. The volume of the measure is ½ cu. ft., or 14160 cu. cm. The weight per cu. ft. of the concrete will not vary enough to have a significant effect and, assuming it at 145 lb., h_1, for the dimensions shown, becomes 2.5 cm. of mercury. Pressure h_2 depends on the position of the "zero mark" (not shown on the drawning) and, for the apparatus used, is 6.6 cm. of mercury.

The application of equations 7 or 8 and 9 shows R to be 204 cu. cm. for 5 per cent of air and 402 cu. cm. for 10 per cent, when the barometric pressure is 76 cm. and the applied pressure is 40 cm. of mercury. From these same equations it can be found that a reduction in volume of 204 c.c. can also be made to indicate 5 per cent of air for other barometric pressures if the correct applied pressure is used. For a barometer of 60 cm., the application of 33.8 cm. of pressure will accomplish that result. Calculation will show that the application of this same pressure to concrete containing 10 per cent of air will give the correct result within less than 0.1 per cent.

Such a refinement in calibration as described above is intriguing but somewhat academic. It probably is not worth the trouble, and, moreover, it detracts from flexibility in use of the meter; it ties the user to one applied pressure. The authors believe the advantages and disadvantages balance in favor of calibrating the tube in terms of volume—most conveniently, cubic centimeters.

The computations need not be laborious. Equation 8, combined with equation 9, is so nearly a straight line, when plotted, that a simple table can be constructed which gives factors which may be used to calculate the percentage of air by the simple division of the reduction in volume R by the factor K. Table 1 shows factors for this apparatus, assuming a barometer of 76 cm. and an applied pressure of 40 cm. of mercury. Similar factors can be calculated readily for other barometric and applied pressures.

To illustrate the use of the table, consider a reduction in volume R of 225 c.c. The factor for 200 will represent adequate accuracy. The per cent of air would be $\frac{225}{40.9}$ = 5.5 per cent. The precise application of

TABLE 1—FACTORS FOR COMPUTING AIR CONTENT

Per cent air $= \frac{R}{K}$, where R = reduction in volume of water column in cu. cm. and K = factor for barometric pressure of 76 cm. and applied pressure of 40 cm. of mercury.

Using the value of "K" nearest to reading "R" will yield percentages accurate to within 0.02. If greater accuracy is desired, other values for "K" may be obtained by interpolation.

Reduction in Volume (R, cu. cm.)	Factor K
100	41.5
200	40.9
300	40.4
400	40.3
500	39.9
600	39.6
700	39.1

the basic equation would permit of no different reading on a 20 in. slide rule.

TEST DATA

The operation of the air meter is relatively simple. Some troubles are encountered with casual air introduced with the water, but they can be minimized by using water that is relatively air free and by exercising care in filling the cone assembly. When air is included with the water it should be removed by gentle tapping of the container. If vigorous methods are used, air in the concrete also will be displaced.

Relatively few tests suitable for reporting have been conducted although many preliminary trials were made in developing technic. A typical group of data, for concrete having a wide range in air content, are shown in Table 2. The concrete contained 5.5 sacks of cement per cu. yd. and had a slump of 3 to 4 in.; well-graded sand and gravel with a maximum size of 1 in. was the aggregate. The specific gravities of the ingredients were known with accuracy and the results of the air meter are compared with those by the gravimetric method (i.e., calculated from absolute volumes).

With the air meter, it will be observed, 10 to 15 readings were taken for each determination. It should not be assumed that such detail is suggested for standard use. The procedure was to take readings at applied pressures of 20, 30, and 40 cm. of mercury both as the pressure was applied and released, and to repeat this operation one or two times. In all cases the water in the column did not return to the zero mark and, before starting the next run, water was added to mark.

The results for the air meter are, with one exception, higher than those arrived at by computations based on the gravimetric method, the differences averaging about 0.6 per cent of air. These differences are

TABLE 2—DETERMINATIONS OF ENTRAINED AIR IN CONCRETE BY AIR METER AND GRAVIMETRIC METHODS

Klein Air Meter illustrated in Fig. 2 used; weight per cu. ft. determined in the measure forming part of the meter.

Concrete contained 5.5 sacks of cement per cu. yd.; slump of 3 to 4 in.; aggregate, well graded sand and gravel with maximum size of 1 in.

Concrete mixed six minutes in tilting mixer of 1 cu. ft. capacity.

Applied pressure cm. of *Hg*.	Air Contents for Different Batches						
	1 (0.000)*	2 (0.010)	3 (0.010)	4 (0.015)	5 (0.015)	6 (0.020)	7 0.020)
1st Run							
20	2.0	7.3	6.8	9.8	8.9	11.2	12.1
30	2.1	7.2	6.8	9.9	8.8	10.8	12.2
40	2.1	7.1	6.8	9.7	8.6	10.8	12.6
30	2.1	7.3	6.8	9.9	9.1	11.6	12.4
20	2.1	7.3	6.8	9.9	9.1	11.5	12.1
Average	2.1	7.2	6.8	9.8	8.9	11.2	12.3
2nd Run							
20	2.1	7.2	6.8	9.8	9.2	11.4	12.0
30	2.1	7.2	6.8	9.9	9.2	11.4	12.0
40	2.1	7.1	6.8	9.5	8.8	11.0	12.1
30	2.0	7.2	...	9.9	9.2	11.4	12.9
20	2.1	7.3	...	9.9	9.0	11.4	12.2
Average	2.1	7.2	6.8	9.8	9.1	11.3	12.2
3rd Run							
20	2.1	7.1	6.9	9.8	9.1	11.5	...
30	2.1	7.2	6.7	9.9	9.1	11.4	...
40	2.1	7.2	6.8	9.4	8.8	11.1	...
30	2.1	7.3	6.8	9.9	9.1	11.4	...
20	2.1	7.3	6.8	9.9	9.1	11.4	...
Average	2.1	7.2	6.8	9.8	9.0	11.4	...
Grand Average	2.1	7.2	6.8	9.8	9.0	11.3	12.2
Gravimetric Method	1.3	6.5	6.1	8.6	9.1	11.2	11.8

*Vinsol resin per cent by weight of cement.

consistent with those found by others between the gravimetric and volumetric methods; J. C. Pearson, in his paper in the 1944 *Proceedings* of the American Society for Testing Materials, reported an average difference between the two methods of 0.8 per cent.

A study of the range in readings for each determination will give an idea of the reproducibility of the results. For reference numbers 1, 2 and 3, with air contents of less than 7 per cent, the variations are negligible, generally being of the order of 0.1 per cent. For reference numbers 4, 5, 6 and 7, with air contents of 9 to 12 per cent, somewhat greater variations were experienced, but none excessive. Experiences suggest that, with a later model of the air meter, even these small variations would be reduced. It is noteworthy that determinations at applied pressures of 20 and 30 cm. of mercury appear to be at least as reliable as those at 40 cm. There are indications that the use of the lower pressures is preferable.

Prior to the construction of the air meter, tests of the air content of mortar were made by adaptation of a flask (the Montreal Moisture Meter), designed for determinations of specific gravity and surface moisture of aggregates. The results of these preliminary tests are shown in Table 3. Here again it will be observed that the air contents shown by the pressure method differ from those by the gravimetric method in about the same degree as has been found for the volumetric procedure.

The question has been raised as to whether or not this procedure is applicable in the case of extremely harsh mixtures. In an endeavor to answer it, some tests have been made, only recently and incompletely, with under-sanded concrete using blast-furnace slag as a coarse aggregate. Experiences revealed no particular difficulties due to the harshness of the mix. The results with the air meter were in all cases higher than those arrived at by computation.

TABLE 3—DETERMINATIONS OF ENTRAINED AIR IN MORTAR BY PRESSURE AND GRAVIMETRIC METHODS

Weights per unit of volume measured in a 520 cc. container.

Determinations by pressure method made in Montreal Moisture Flask containing 1475 cc. of mortar.

Mortar 1 part cement to 2 parts sand by weight using Standard No. 20 to 30 Ottawa sand. W/C = 0.6 by weight. Vinsol resin in NaOH solution used as air-entraining agent.

Test No.	Percent Air	
	By Unit Weight Method	By Pressure Method
1	12.0	13.0
2	10.2	11.5
3	11.6	12.0
4	8.7	9.1
5	11.5	12.1
6	10.0	10.3
7 (Sand only)	24.7	25.7

However, a complication was encountered, probably because of the porosity of the slag. The data are shown in Table 4. The spreads between determinations by the air meter and by the gravimetric method were considerably greater than for the sand and gravel concrete. Comparing the "grand average" air meter results with the gravimetric

TABLE 4—DETERMINATIONS OF ENTRAINED AIR IN SLAG CONCRETE BY PRESSURE AND GRAVIMETRIC METHODS

Klein Air Meter illustrated in Fig. 2 used; weight per cu. ft. determined in the measure forming part of the meter.

Concrete contained 4.4 sacks of cement per cu. yd.; slump about 1 in.; aggregate, sand and blast furnace slag with maximum size of 1 in.

Concrete mixed six minutes in tilting mixer of 1 cu. ft. capacity.

Applied Pressure cm. of *Hg*.	Air Contents for Different Batches			
	1 (0.010)* (52)†	2 (0.020) (51)	3 (0.000) (40)	4 (0.015) (36)
1st Run				
20	9.9	13.7	3.0	10.1
30	10.2	14.0	3.7	11.8
40	9.9	...	5.2	11.9
30	11.4	...	4.0	12.9
20	12.0	15.9	4.5	13.6
Average	10.7	14.5	4.1	12.1
2nd Run				
20	11.2	15.1	4.5	12.6
30	11.0	14.5	4.6	12.5
40	10.6	...	...	12.5
30	11.8	...	...	12.9
20	12.0	16.1	...	13.1
Average	11.3	15.2	4.6	12.7
3rd Run				
20	11.2	15.1	4.4	12.5
30	11.1	14.7	4.6	12.7
40	10.6	...	4.5	12.6
30	11.6	...	4.6	13.1
20	11.4	16.0	4.9	13.4
Average	11.2	15.3	4.6	12.9
Grand Average	11.1	15.0	4.4	12.6
Gravimetric Method	8.1	12.9	2.5	9.0

*Vinsol resin per cent by weight of cement.
†Sand, per cent of total aggregate.

results, the average spread was 2.8 per cent as compared with the 0.6 per cent stated earlier. A part of this discrepancy may be accounted for (not surely) by inadequate knowledge of the specific gravity of the slag—a determination difficult to make with assurance. The complication referred to is represented by the progressively larger amounts of air found in the same sample by successive determinations, and the lower amount generally revealed by the highest pressure used.

The data are presented as a matter of interest and for whatever they may be worth. The need for additional work is recognized, particularly with lower air contents. However, explanations, which appear logical, present themselves for the apparent inconsistencies. The slag, in a dry condition, was soaked in water for 24 hours before being incorporated in the concrete. Obviously, that was not adequate time for complete saturation. It is reasoned, therefore, that as pressure was applied water was forced into the slag with the release of a corresponding amount of air to the concrete, revealed by subsequent determinations. That some such phenomena occurred is confirmed by tests made on slag alone, with the material dry, soaked for 24 hours, and saturated under a vacuum—with the first two indicating the presence of air and the latter showing no air on any run.

The lower value of air (sometimes) for the highest pressure might be reasoned as resulting from particle interference of the aggregates.

Both of these "complications" suggest the desirability of using a lower pressure, particularly with non-saturated porous aggregates, to minimize the additional absorption and, also perhaps, to minimize the effects of particle interference.

CLOSING REMARKS

The method for the direct measurement of the amount of entrained air in concrete described in this paper is recommended as being worthy of favorable consideration. The basic theory is sound. Experiences with the method convince the authors that the application of the theory is workable. The relatively few data cited are convincing; even more convincing are the preliminary tests made by both authors in developing the technic. The authors' excuse for reporting the procedure without complete investigation is their desire to make it available to other investigators at an earlier date than would otherwise be possible.

No rights are reserved by the authors with respect to the use of the procedure or the device described. However, with respect to the device, they wish to acknowledge assistance received from C. A. Hogentogler, Jr., who constructed the one used in most of the tests and who has since built a more refined model.

[To facilitate selective distribution, **separate prints** of this title (42-33) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946]

Title 42-33 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT, MICHIGAN **June 1946**

Automatic Dispensing Equipment for Air-Entraining Agents*

By R. R. KAUFMAN†
Member American Concrete Institute

SYNOPSIS

Automatic dispensing equipment is described as a means of getting an admixture into the concrete mix, at the mixer, with all the accuracy desirable.

Others have indicated the great improvements in concrete structures resulting from the use of certain admixtures and the need to introduce such admixtures uniformly at the mixer if desired results are to be achieved. The desirability of utilizing automatic dispensing equipment for this purpose has long been recognized by our company and we devised and used such equipment over five years ago.

The control of air entrainment and the other properties of the concrete is extremely difficult unless the addition is made at the mixer and in variable quantities. The addition should be at least as automatic and accurate as the measurement of any other constituent of the mix. To meet this need we have perfected a dispensing device which is accurate, rugged, easily regulable, and automatic.

Our problem in designing dispensing equipment was to provide an apparatus which would be rugged and easily regulable for different batches with varying cement contents over a wide range—from ½ yd. to 6 yds.—one which was at least as accurate as the other batching devices used in batching or ready-mix plant operation for sand, aggregate and cement.

In our experimental installations we found meters or other apparatus now available for this purpose wholly inadequate and unsatisfactory. Frequent plugging of the equipment by foreign materials such as cement and sand occurs. Equipment which depends upon close clearances not

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Chief Engineer, The Master Builders Co., Cleveland, Ohio.

only becomes easily plugged but soon shows excessive wear, rendering the equipment inaccurate after a period of operation under conditions encountered in batching and ready-mix plants.

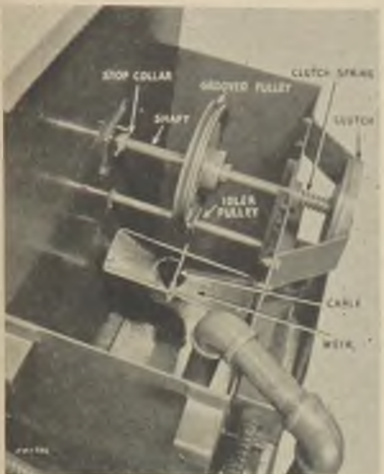
Fig. 1 shows the general arrangement of the installation of the automatic dispensing device at the batching plant. It consists of storage tanks, pump and automatic dispenser. The storage tanks for the cement dispersing agent or other type of air-entraining agent in liquid form are usually located at ground level. Two tanks are generally provided so that one can be filled while the other is being used, resulting in continuous operation of the batching plant. The cement dispersing agent solution is of such concentration that 2 quarts are required per bag of cement. Solutions of foaming type air-entraining agents likewise are used in similar concentration because these relatively dilute solutions lend accuracy to the operation in contrast to the measurement of the amount in cubic centimeters. A pump connected to the storage tanks elevates the solution to the automatic dispensing unit. This is located at any place convenient for the batching plant operator.

The automatic dispensing unit is a rugged instrument built for heavy duty service. It has a large, easily readable dial calibrated in pounds as well as in bags of cement. It is only necessary to set the dial to correspond to the amount of cement used in the mix; then when the start button is pressed the dispenser will deliver the required amount of the reagent solution for the batch.

A 2-in. pipe connected to the outlet of the dispensing unit conducts the solution to desired delivery point. In central mix plants delivery of the solution is generally direct to the concrete mixer, while in transit mix operations it is usually into the mixer trucks. When mixing starts, practically immediately after charging either the mixer or the truck equipment, there is no measurable variation in results regardless of the sequence in which the reagent solution and other constituents of the mix are discharged. If the operation is so conducted that mixing is deferred until arrival at the job site, then the reagent solution is discharged with the sand and aggregate, and the cement is discharged last so that it is on the top of the load. When dry batching into compartment trucks from a central batching plant, the reagent solution is discharged simultaneously with the sand and aggregate and the cement is batched on top of the aggregate.

Fig. 2 is a view looking into the top of the dispensing unit. The construction is extremely simple and very accessible. It consists of a calibrated tank with a constant level overflow and an inverted weir which is regulable by the knob in front of the dial.

The automatic operating mechanism of the dispensing unit is shown in Fig. 3. When the start button is pushed it causes the discharge solenoid valve to open and simultaneously starts the timer. This allows the reagent solution to flow out of the dispenser until it reaches the level



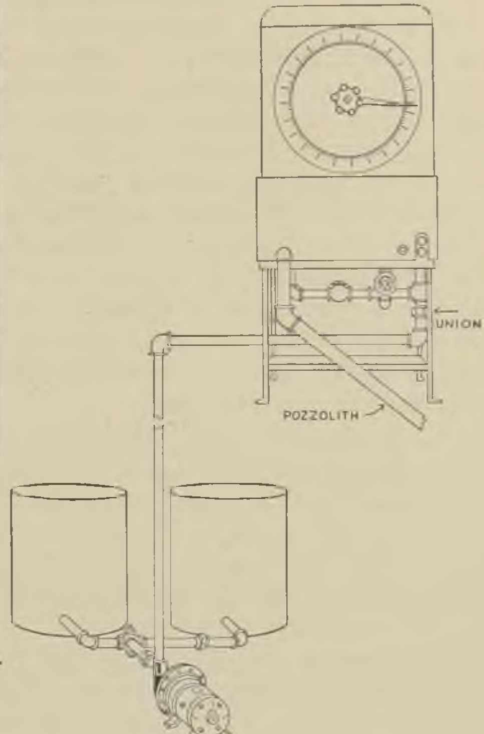


Fig. 1 (right)—Diagramatic sketch of Model A Pozzolith dispensing device installation.

Fig. 2 (above)—Interior of the dispensing unit.

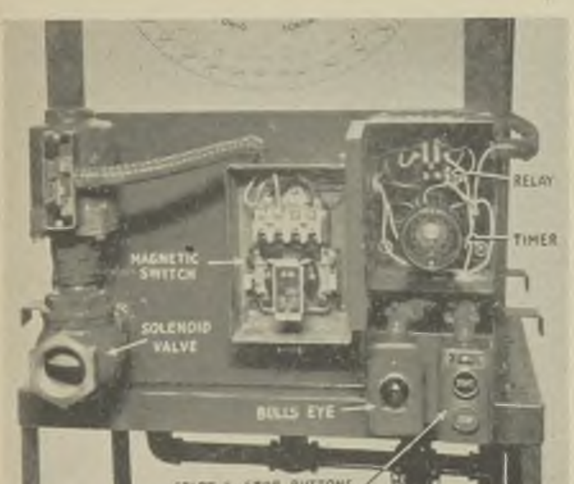


Fig. 3—Dispenser operating equipment.

of the top of the weir shown in Fig. 2. The time that the valve is open is determined by the setting of the timer. Normally 30 seconds is sufficient to empty the entire capacity of the dispenser. If the dispenser is to be used to dispense the reagent into dry batch trucks having charges of 1 cu. yd. and carrying two or more charges per truck, the timer may be set for 10 to 12 seconds which will provide ample time for the dispenser to deliver the small quantity of liquid required. After the time cycle, corresponding to the setting on the timer, has elapsed the solenoid valve closes automatically, starting the motor and pump connected to the storage tanks. The pump rapidly fills the dispenser with the reagent solution to the constant level overflow located at the back. Overflow of the solution opens the circuit on the overflow switch and stops the pump. During the entire cycle of operation the red bulls eye on the front of the dispenser is lighted. This indicates when the equipment is in operation and, if lighted after the normal time cycle, it usually indicates that the storage tank is nearly empty.

The dispenser has a dial calibrated both in pounds and bags of cement to make it easy for the operator to set the instrument without calculations to deliver the proper quantity of reagent solution. This arrangement is satisfactory for cement dispersion where the same amount is required per sack of cement regardless of the amount of cement per cu. yd., slump, design of mix, or other variable properties. In the case of foaming type air-entraining reagents which require variable quantities to produce the desired results, the dispensing unit has a variquantity attachment. When the variquantity attachment is used, the quantity of the foaming type air-entraining reagent may be varied from 66⅔ per cent to 133 per cent of the standard amount. The solution in the storage tanks is made of such concentration that 1½ quarts of the solution contain the predetermined amount of the reagent to produce correct results under controlled standard conditions. The variquantity attachment is then re-set by the concrete technician from time to time as materials or conditions vary on the job. After the concrete technician re-sets the variquantity attachment then it is necessary only for the batch plant operator to set the dial of the dispenser for the pounds of cement used in the mix and push the start button to discharge the proper amount of reagent solution into the concrete mix. The dispenser may be equipped with an autographic recording device.

In some more modern batching and ready-mix plants which use automatic dial scale equipment for proportioning their mixes, it is possible to connect and synchronize the automatic dispensing unit with the cement scale.

Automatic dispensing units are now installed in more than 70 leading ready-mix plants throughout the United States alone.

[To facilitate selective distribution, separate prints of this title (42-34) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. Discussion of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946]

Title 42-34 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE Vol. 42

JOURNAL
of the
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN June 1946

Mechanical Dispensing Devices for Air-Entraining Agents*

By E. M. BRICKETT†

SYNOPSIS

Several devices for accurate measurement of liquid admixtures as introduced into the concrete batch at the mixer are described and illustrated, with special reference to air-entraining agents as used in ready-mix and concrete products plants and on paving jobs. Since the quantity of the solution is relatively small, accuracy is important for uniform results.

In the production of air-entraining concrete by the introduction of air-entraining agent at the mixer, a metering device is essential to the accuracy and simplicity of the operation.

We have had some interesting experiences in the design and operation of several metering devices in the introduction of air-entraining agents in products plants, ready-mix plants, and on paving jobs.

The quantity of solution to be added to a batch is relatively small. The volume required ranges from one to one and one-half fluid ounces of "agent" per bag of cement. There are occasions when the required quantity is only a fraction of an ounce, and others when the figure is greater than two ounces per bag, but, in general, the figures of one to one and one-half ounces apply.

The simplest method of adding the air-entraining agent is, of course, by means of a dipper. A small tin can, cut down to the required capacity, and fitted with a handle, makes a very convenient tool for measuring and applying the agent. A pail of the solution is placed in a handy location and the agent is dipped from the pail into the batch in the weigh hopper or directly into the mixer.

A very simple measuring device which now is used in some products plants and ready-mixed concrete plants is shown in Fig. 1.

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Dewey & Almy Chemical Co., Cambridge, Mass.

In this particular meter a measuring glass is located between two cocks. The supply comes in from the left and with the right cock closed the supply is admitted through the left cock until the desired quantity shows in the calibrated glass. With both cocks closed the agent is held at the meter until required. Thus, by opening the right cock the agent is discharged from the meter to the batch.

In Fig. 2 is shown a proportioner developed particularly for use in ready-mixed concrete plants, though its application is not limited to such plants. This meter is filled and discharged through a three-way cock. In operation, the cock is set to the filling position and the meter is completely filled, the liquid finally filling the vent tube to the level of the liquid in the supply reservoir. A section of lucite tubing in the vent permits the operator to know when the meter is filled. Then, by turning the three-way valve a quarter turn to the discharge position, the liquid flows from the meter to the concrete batch. The amount discharged is governed by the setting of the calibrated quadrant. The discharge is rapid and has a sufficiently sharp cut off. The three-way valve prevents any possible discharge of liquid direct from the reservoir to the batch.

This meter is simple and inexpensive to build. There is no machine work required in its construction; it can be built in a welding shop.

Fig. 3 shows such a meter installed in a ready-mixed concrete plant. The meter is small and lends itself easily to installation in a convenient position. In this illustration the meter is mounted on the scale-beam housing where it is handy for the operator. In this particular installation the liquid is discharged into the sand in the weigh hopper. There are other installations in which the liquid agent is discharged into the water line and carried into the batch with the gauging water.

In ready-mix plants where the volume and proportions of successive batches are continually changing, it is a simple matter to set the meter for any desired quantity of air-entraining agent. The meter fills and discharges a measured quantity merely by a quarter turn of the three-way valve; the operator does not have to stand by to close the cock at a given moment.

In Fig. 4 is shown a meter which has been used on paving mixers to meter the air-entraining agent into the batches. This meter was designed and built by our Seattle distributor, the Charles R. Watts Co. It is entirely automatic in its action and insures a measured quantity of agent in every single batch. This meter is in effect a pump with an adjustable stroke. As the aggregate skip of the paver is raised it bears on the bar at the left and swings it in a counter-clockwise direction. Through the connecting link and the bell crank this forces down the plunger of the pump, forcing the liquid agent through the rubber hose to the water line

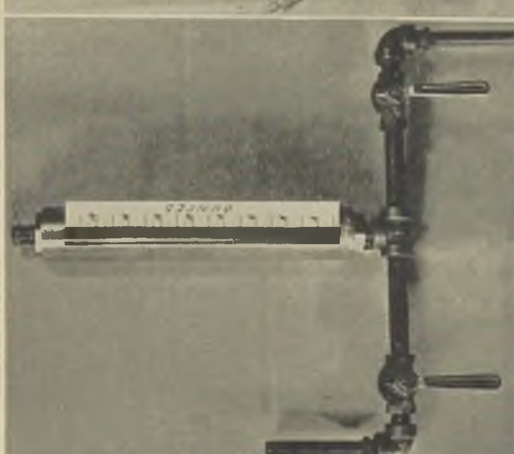


Fig. 1.—Simple measuring device.

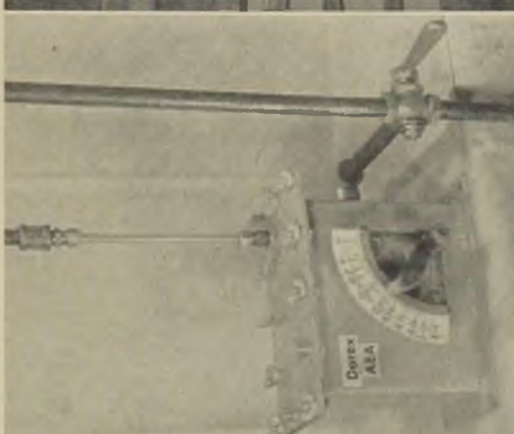
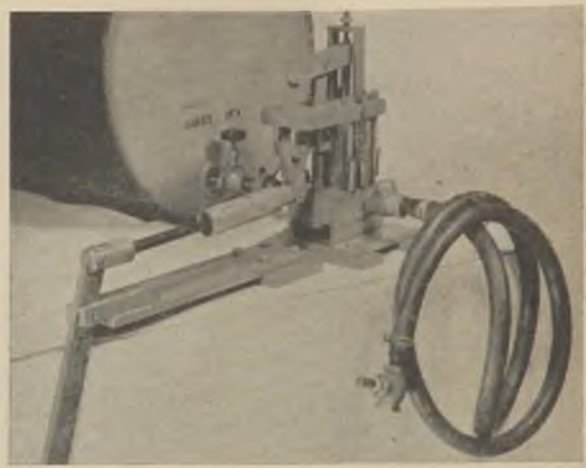


Fig. 2.—Brickett proportioner for ready-mix concrete plants.

Fig. 3.—Proportioner installed in ready-mix plant.

Fig. 4.—Watts meter.



between the water tank and the mixer drum. As the skip is again lowered the pump plunger returns to the adjustable stop, drawing in a new charge of agent in preparation for the next batch. The position of this stop, which is merely a set screw with a lock nut, determines the length of stroke and, consequently, the quantity of agent pumped to the batch. To adjust for any desired stroke the mixer skip is raised and the pump plunger is forced to the bottom of its stroke. While it is held there the set screw is adjusted with the aid of the proper one of three spacers shown attached to the pump by chains.

At one end of the tension link there is a spring over-run mechanism. The pump plunger reaches the end of its stroke while the skip still has a foot or more to go. The remaining travel of the skip is taken up in the over-run mechanism. With this arrangement the pump attains its full stroke even though the skip may not be completely raised. Also the skip may be rapped or vibrated without pumping more agent into the batch.

This meter is entirely automatic. It operates on the pressure principle and its performance is constant, even though the paver may be on a grade.

To facilitate selective distribution, **separate prints** of this title (42-35) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946

Title 42-35 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

A Simple Accurate Method for Determining Entrained Air in Fresh Concrete*

By S. W. BENHAM†

INTRODUCTION

This method, which is used by State Highway Commission of Indiana both in the laboratory and in the field, is based on the equation

$$\text{percentage of air} = \frac{(T\text{-}A)\ 100}{T} \text{ in which}$$

T = unit weight of the air-free concrete, and
A = unit weight of the concrete containing air.

The above equation is, of course, applicable regardless of the method employed for determining T and A. The principle involved in the experimental method described here, i.e., measuring volume by displaced water, is so old that no originality is claimed.

The determination of A is made by the same procedure as that employed in the yield test (cement content) using a 0.5 cu. ft. cast aluminum yield bucket.

T is determined by measuring the volume, by displacement in water, of a sample of fresh concrete of known weight. The apparatus used consists of the yield bucket and a hook gage which converts the device into a pycnometer the volume of which is calibrated with water of known temperature (0.452 cu. ft. in Table 1). The necessary data and computations for the complete air determination test consists of four weighings, five subtractions, three divisions, and one multiplication, all of which are extremely simple. See example in column 1 of Table 1.

PROCEDURE

After A is determined the operator removes concrete from the yield bucket until approximately 30 pounds remain (line E Table 1). After

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Indiana State Highway Commission, Indianapolis, Ind.

TABLE 1—FORM USED BY INSPECTOR IN THE FIELD

I.T. 595B

STATE HIGHWAY COMMISSION OF INDIANA

AIR CONTENT REPORT

CONTRACT NO.__________PROJECT NO._____SECTION_________194____

Test Number	1	2	3	
Weight of Container and ½ cu. ft. Concrete Weight of Container (empty, clean, and dry) Weight of ½ cu. ft. Concrete Weight of 1 cu. ft. Concrete	85.00 10.00 75.00 150.00			 A
Weight of Container and Concrete Sample Weight of Container (empty, clean, and dry) Weight of Concrete Sample	40.00 10.00 30.00			D E
Weight of Container, Concrete Sample and Water to Gauge Point Weight of Container and Concrete Sample Weight of Water to fill Container to Gauge Point	56.10 40.00 16.10			 D F
Volume of Water in cu. ft. $\frac{F}{62.30}$	0.258			G
Calibrated Volume of Container to the Hook Gauge Point Less Volume of Water—cu. ft. Absolute Volume of Concrete Sample in cu. ft.	0.452 0.258 0.194			V G H
Weight of Solid Concrete on Air Free Basis $\frac{E}{H}$ Lbs. per cu. ft. Air Content $= \frac{T - A}{T} \times 100$	154.64 3.00			T P

CALCULATIONS:—

____

____

Title

weighing (line D), sufficient water is added to inundate the sample completely. The hook gage is set in position and water added until the "dimple" in the water surface breaks. The hook gage is then removed and the gross weight obtained. From this line D is subtracted to obtain line F, the weight of water to fill the pycnometer to the hook gage point. This weight of water is converted into cubic feet, recorded in line G, and subtracted from the calibrated volume of the pycnometer (line V) to obtain line H, the absolute volume of the concrete sample in cubic feet. T is then obtained by dividing the weight of the sample (line E) by its absolute volume (line H) obtaining, as shown in the attached example, 154.64 lbs. per cu. ft. The formula, $\frac{(T\text{-}A)\ 100}{T}$, is then used to compute percentage of air which is recorded in line P.

ADVANTAGES OF THE METHOD

This method for determining T experimentally is considered to be much more satisfactory than the method in which T is arrived at by computation, by dividing the weight of the batch by the sum of the absolute volumes of all the constituent materials. In the latter method the batch weight of each of the five constituents (two sizes of coarse aggregate, sand, cement and water) is divided by its specific gravity and by the unit weight of water to obtain its absolute volume. All five absolute volumes are then added to obtain the absolute volume of the batch. The weight of the batch is then divided by the volume to obtain the theoretical air-free unit weight of the fresh concrete. Such computations are laborious and the computed value of T is of no reliability whatever unless corrections are made for the moisture content of each of the three aggregates. These corrections require additional testing at the batching plant and make the computations still further complicated. An additional criticism is that specific gravity of aggregate from a given source is sometimes variable. A still additional criticism is that a change in the batch weights requires a new computation.

In the experimental method the operator is not concerned with batch weights, specific gravities, nor moisture contents of the aggregates. If, unknown to the inspector, steel punchings were substituted for one of the sizes of coarse aggregate the validity of the method would be entirely unaffected. Such a substitution would, of course, be apparent at once because of the high values of both A and T. What the inspector is actually doing is determining the specific gravity of a sample of fresh concrete although he does not compute it because he wants to know unit weight only.

Both A and T are determined at the mixer thereby avoiding all travel back and forth between the batching plant and mixer.

Three independent tests, as provided for in Table 1, can be conducted and all computations performed within approximately a half hour. In addition to being quick the method and computations are so simple that the average inspector can learn to conduct the test satisfactorily with very little instruction. The fact that he is not concerned with the quantities nor properties of any of the materials in the batch makes his work extremely simple in comparison with the "sum-of-the-absolute-volumes" method.

CRITICISMS OF THE METHOD

It has been found that with excessive quantities of some foaming agents, sufficient to cause more than 7 per cent air, the removal of *all* the air by stirring becomes quite difficult. For that reason the method might be improved by the use of a water tight lid on the yield bucket so the apparatus can be rolled. However, this change in procedure, which would require additional equipment and more time, has not been considered necessary because our present specifications permit not more than 5 per cent air. The foam on the surface of the water, of course, obscures the hook gage point. Such foam can be dispersed instantly by one or two drops of a normal solution of amyl alcohol.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Effect of Use of Blended Cements and Vinsol Resin-Treated Cements on Durability of Concrete*

By W. F. KELLERMANN†
Member American Concrete Institute

SYNOPSIS

Presents a part of the results obtained from an investigation of the durability of concrete, by the Public Roads Administration using blends of portland cements with natural cements (86 per cent and 14 per cent by weight, respectively) and Vinsol resin-treated cements. Results presented in this contribution have a bearing on resistance to freezing and thawing tests, especially because of unusual results of a prolonged interruption of the daily freezing and thawing cycle.

The three methods now commonly employed for securing air entrainment in concrete are: (1) the use of air-entraining portland cements, (2) air-entraining admixtures added at the mixer and (3) the use of plain portland cements blended with blending cements containing an air-entraining agent. In the latter case the general practice has been to use a blend consisting of 1 bag of natural cement and 5 or 6 bags of plain portland cement, the air-entraining agent being carried by the natural cement.

This discussion concerns results of freezing and thawing tests obtained in connection with an investigation to determine the effect of the use of blended cements and Vinsol resin-treated cements on various properties of concrete. A paper giving complete results of this investigation is now in preparation and will be available at a later date. The results presented herein were selected because they bear directly on the subject of durability from the standpoint of resistance to freezing and thawing and because of the rather unusual results obtained in the freezing and thawing test resulting from a prolonged interruption in the daily freezing and thawing cycle.

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Senior Materials Engineer, Division of Physical Research, Public Roads Adm., Washington, D. C.

Included in the investigation were two plain portland cements; the same two cements interground with unneutralized Vinsol resin; and the two plain cements blended with six different blending cements in the proportion of 86 per cent portland cement and 14 per cent blending cement by weight. Five of the six blending cements were natural cements and of the five, two were from the same mill. One of these contained an air-entraining agent while the other did not. Aggregates were a river sand and a river gravel of 1-in. maximum size. The mix used with the plain cements contained approximately 5¾ bags of cement per cu. yd. and 6 gal. of water per bag. For those mixes containing the treated cements or the blends, the same weight proportions were used as were used with the plain cements. However, the water was varied so as to keep the slump within the range of 3 to 4 in. for all mixes.

Test specimens were 3- by 4- by 16-in. beams which were alternately frozen and thawed in water, the temperature range being from 70 to -20 F, one cycle being made per working day. The natural frequency of each specimen was determined dynamically just before freezing and thawing started and after the completion of each five cycles of freezing and thawing. Readings were taken on the 3-in. and also on the 4-in. faces of each specimen, there being two specimens for each variable investigated.

Two separate series of freezing and thawing tests will be discussed. In the first series the specimens were given an initial curing of 230 days in the moist room at 70 F. The results of this series are given in Fig. 1, which shows the relation between freezing and thawing cycles and the reduction in the natural frequency squared (N^2), which is a measure of the reduction in the dynamic modulus of elasticity. In the left panel results are shown for cement A plain, cement A treated, and cement A plain blended with the six blending cements. Reference to the panel indicates a marked difference in behavior for the different combinations tested. For instance, specimens representing the plain cement A (1) and the blends A (1) + II (a) and A (1) + VI broke down very rapidly. The air contents of these three combinations were all of the order of 1½ per cent. Somewhat better resistance was shown by the blend A (1) + I, which had an air content of the order of 2 per cent. The remaining combinations, treated cement A (2) and the blends A (1) + II (b), A (1) + III and A (1) + V, all showed superior resistance. The range in air content for this group was of the order of 3 to 5 per cent. It is of interest to note that blending cement II (b), included in one of the combinations in the latter group, was the same product as blending cement II (a) except that it carried air-entraining material. This was reflected in the air contents of the two different combinations and serves as an ex-

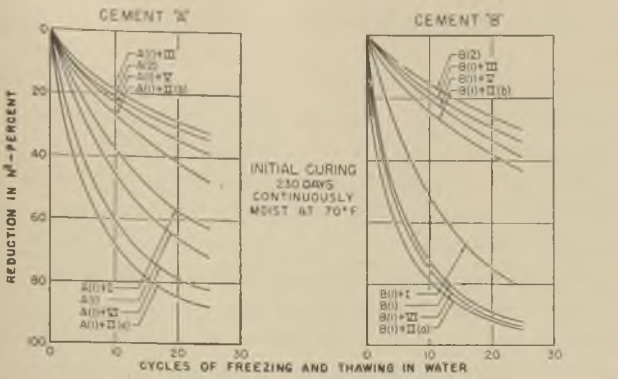

Fig. 1—Effect of use of blended cements and Vinsol resin-treated cements on resistance to freezing and thawing (First series).

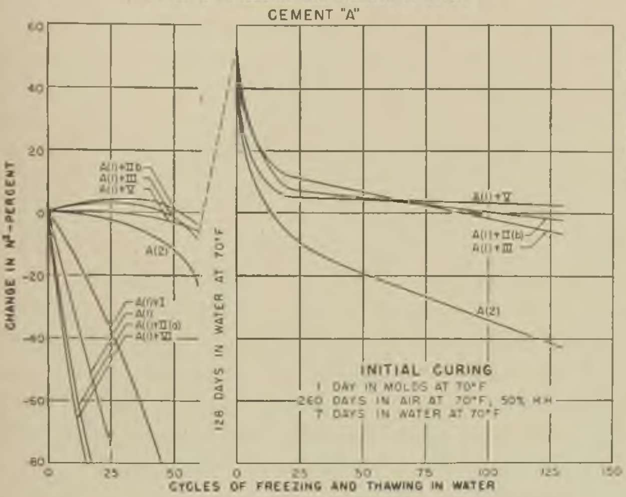

Fig. 2—Effect of use of blended cements and Vinsol resin-treated cements on resistance to freezing and thawing (Second series, Cement A).

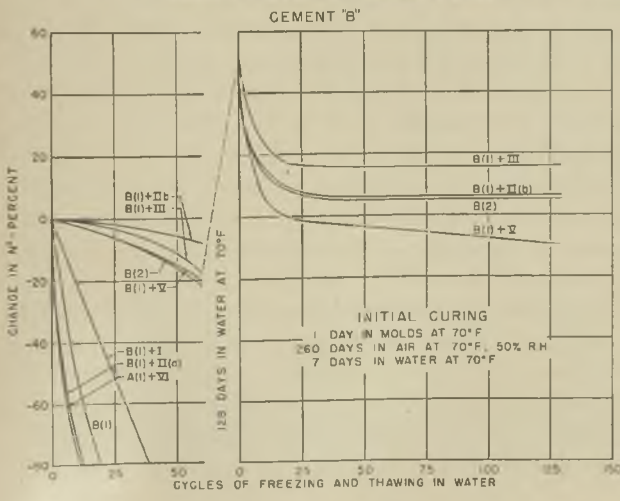

Fig. 3—Effect of use of blended cements and Vinsol resin-treated cements on resistance to freezing and thawing (Second series, Cement B).

planation for the difference in behavior between blends A (1) + II (a) and A (1) + II (b).

Results obtained with cement B are shown in the right panel of Fig. 1. Although somewhat poorer resistance was shown for the plain cement B (1) and the blends B (1) + I, B (1) + II (a) and B (1) + VI than in the case of cement A, the order of resistance for the eight combinations was, with one minor exception, exactly the same as shown for cement A. The general trends were also the same.

Because of the great difference in results shown between the different combinations after 25 cycles of freezing and thawing, it was decided to test all specimens in flexure and compare the results with tests on companion control specimens which had been stored continuously in moist air at 70 F. This was done with the results shown in Table 1. The table indicates good correlation between the reduction in modulus of rupture and the reduction in the natural frequency squared.

The second series of freezing and thawing tests was conducted with companion specimens which had been originally used for volume change measurements. Specimens were cured in the molds under wet burlap for 24 hours and then stored in air at 70 F. and at a relative humidity of 50 per cent for 260 days. They were then immersed in water for 7 days before freezing and thawing started. After 60 cycles of freezing and thawing it was necessary to interrupt the daily cycle because of the urgent need of the freezing equipment for work related to the war effort. These first 60 cycles of freezing and thawing will be referred to as the first phase. Accordingly, the specimens were stored in water for 128 days after which the daily cycle was again started and continued for 130 additional cycles. This will be referred to as the second phase.

Results of these interrupted tests are shown in Fig. 2 and 3. Fig. 2 gives results for cement A and from it the same trends during the first 60 cycles of freezing and thawing (first phase) are exhibited as were shown in Fig. 1. A very rapid breakdown is indicated for plain cement A (1) and the blends A (1) + II (a) and A (1) + VI. The group showing good resistance in the first series, A (2), A (1) + II (b), A (1) + III and A (1) + V, also showed the best resistance in this series. However, combination A (2) gave poorer resistance than any of the blends in the good group. Of interest is the shape of the curves for the good group, which show an opposite trend from their companions in Fig. 1. This is due to the fact that there was no loss in frequency during the first few cycles. In fact some combinations showed a gain during the first 40 or 50 cycles. All specimens gained slightly in weight during the first few cycles, indicating additional curing after freezing and thawing started. Apparently the 7-day immersion period following the 260-day air curing treatment was insufficient to completely saturate the specimens.

At the completion of 60 cycles of freezing and thawing, the shape of the curves for three combinations in the good group indicated a rapid rate of deterioration, particularly combination A (2).

As previously explained, the test was interrupted at this time and the specimens from the good group were stored in water at 70 F for 128 days, after which the freezing and thawing cycle was resumed. Frequency readings taken at the end of the water storage period indicated an increase of 40-50 per cent in N^2 over the zero readings taken at the time freezing and thawing was first started (beginning of the first phase). For the first 15 cycles of freezing and thawing immediately following, the percentage reduction in N^2 was at a very rapid rate. However, at 20 cycles the curves, with the exception of the one representing combination A (2), flattened out to the extent that at the end of 130 cycles of freezing and thawing there was little change in N^2 from the original zero taken at the beginning of the first phase. In the case of combina-

TABLE 1—EFFECT OF THE USE OF BLENDING CEMENTS AND VINSOL RESIN TREATED CEMENTS ON PROPERTIES OF CONCRETE

Results of freezing and thawing tests
First Series

Reduction in modulus of rupture and frequency squared after 25 cycles of freezing and thawing in water.

Portland cement	Blending cement	Reduction in	
		Modulus of rupture	N^2
		Percent	Percent
A (1)	None	69	72
A (2)	None	32	35
A (1)	I	61	66
A (1)	II (a)	82	88
A (1)	II (b)	57	48
A (1)	III	40	33
A (1)	V	45	40
A (1)	VI	77	82
B (1)	None	86	92
B (2)	None	37	32
B (1)	I	77	81
B (1)	II (a)	88	94
B (1)	II (b)	57	44
B (1)	III	48	35
B (1)	V	47*	39
B (1)	VI	89	93

*One control test only.

Specimens 3 x 4 x 16-in. beams stored continuously in moist air 230 days prior to start of freezing test. Each result average of two tests unless otherwise noted.

Reduction in modulus of rupture based on strength of control specimens stored continuously in moist air.

tion A (2) the decrement was about 40 per cent. Based on results obtained in the first phase, it would be expected that combination A (2) would not stand up as well as the other combinations tested in the second phase. Apparently due to the greater decrement at the end of the first phase (24 per cent), the rest period of 128 days in water was not as effective in producing the same degree of resistance for this combination as was obtained with the other three combinations.

It is worthy of note that the shape of the curves in the second phase is not the same as those representing the same combinations in the first phase but are similar in shape to those shown in Fig. 1. The data shown in Fig. 1 and for the second phase in Fig. 2 were obtained with specimens which had been subjected to prolonged moist curing prior to the start of freezing and thawing. On the other hand, the data shown for the first phase in Fig. 2 were obtained with specimens which had been subjected to prolonged air curing followed by but 7 days of moist curing prior to freezing and thawing. The above discussion is presented as an explanation of the difference in the general shapes of the curves between the first and second phases and not as an explanation of the general nature of the results shown for the second phase.

The fact that specimens which have been subjected to cycles of freezing and thawing will, when given a rest period in water, show a recovery in dynamic modulus of elasticity, has been reported by other investigators.* However, in the tests referred to, the specimens were tested for strength after a water storage period of 30 days, so that data similar to that shown in the right panel of Fig. 2 were not obtained. The data herein reported indicate, in three cases out of four, that there was practically no decrement in dynamic modulus due to the combined 190 cycles of freezing and thawing to which the specimens were subjected. Compared with the data shown in the left panel of Fig. 1 for the same combination, a much greater decrement would be expected.

It is well known that the moisture condition of the specimens at the start of the test has an influence on freezing and thawing results, thoroughly saturated specimens showing poorer resistance than partially dried specimens. This would explain the superior resistance shown in the first phase of the second series (left panel of Fig. 2), particularly for those combinations of relatively high air contents. On the other hand the excellent resistance shown in the second phase (right panel of Fig. 2) was obtained with specimens that had 128 days of water curing in addition to the water curing available during the first phase of 60 cycles of freezing and thawing. It would appear, therefore, that the results obtained in the second phase were influenced by the 128-day rest period,

*_Proc._ of the Highway Research Board, v. 24, 1944, p. 196, fig. 17.

during which time the specimens had an opportunity for recovery because of favorable curing conditions.

Results obtained with cement B are shown in Fig. 3. The trends follow those shown for cement A in Fig. 2 and for that reason no detailed discussion of the results will be presented.

It is believed that the results shown in Fig. 2 and 3 for the second phase of freezing and thawing (right panels) explain some of the erratic results that have been reported in the past. Interrupting the daily cycle for short periods and allowing the test specimens to remain unfrozen, may result in a recovery in dynamic modulus. For that reason, it is recommended that where interruptions are necessary because of non-work days, the test specimens be kept in a frozen rather than in a thawed condition.

The results discussed in this paper are presented at this time because of the trends shown in Fig. 2 and 3 resulting from the interruption of the daily freezing and thawing cycle. The completed report will include strength and volume change data in addition to data from a third series of freezing and thawing tests involving tests on discs cut from cores which were frozen and thawed in a calcium chloride solution.

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT, MICHIGAN **June 1946**

Air-Entraining Concrete—Pennsylvania Department of Highways*

By W. H. HERMAN†

SYNOPSIS

The experiences of the Pennsylvania Department of Highways with air-entraining concrete in which 331,555 bbl. of normal strength portland cement containing Vinsol resin were used since 1940, are reported. The Pennsylvania department's attitude on the subject of air entrainment is characterized by more concern with the particular admixture used than with the percentage of air entrainment and such use was inspired by difficulties with finer ground cements which prompted seeking an additive to improve pavement durability.

The experience of the Pennsylvania Department of Highways with air-entraining concrete dates from 1940. Since that time we have used 331,555 bbl. of normal strength portland cement containing Vinsol resin. We do not have voluminous laboratory or field data covering the development or effects of specific quantities of entrained air.

Pennsylvania's concrete suffered considerably with the cement industry's development of finer ground cements which were to produce greater plasticity or workability accompanied by higher and earlier strengths. We, therefore, sought an additive to improve the durability of our concrete.

Two experimental projects were constructed in 1940 using normal strength portland cements, of fine and coarse grinds, and the same cements to which pulverized Vinsol resin and tallow were interground with the clinkers at the rate of 0.18 lb. per bbl. of cement. The purpose of those additives at that time however, was to determine their effectiveness as deterrents of concrete pavement surface deterioration. The weight reduction or air entrainment of the cement containing the additives was

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†Chief Research Engineer, Pennsylvania Department of Highways, Harrisburg, Penn.

determined as a matter of record and for future reference, and was based on unit weights of the various concretes.

Our views concerning the exploitation of the air entrainment feature of cement may provoke considerable criticism when we say that we are more interested in the additive causing the air entrainment than we are in obtaining a specific percentage of air entrainment or in recording the air entrainment on a percentage basis as is now generally promoted. We have accepted Vinsol resin as an additive to normal strength portland cement as a deterrent. On the basis of the first two experimental projects, we dismissed tallow as an alternate additive because of the difficulty of handling the cement itself; because of its tendency to cause surface foaming of the concrete and because of the inconsistent workability of the concrete. We have never used Darex because none of the 34 plants furnishing cement to our Department used or promoted the product.

Our first Vinsol resin experimental project (Route 84, Section 10, Crawford County, constructed 1940) is in an area having many freeze-thaw cycles. The concrete mix was not designed on an absolute solid volume basis. The results to date are very gratifying. Our second Vinsol resin experimental project (Route 271, Section 4, Crawford County, constructed in 1942) is in the same area. The concrete mix was designed on the absolute solid volume basis and controlled by means of the slump test. No provision was made on the first project for a reduction of fine aggregate but in the designed concrete of the second project it was found necessary to vary the fine aggregate volume for Vinsol resin concrete.

A third Vinsol resin project (Route 67054, Sections 1A and 2A, City of Philadelphia, constructed in 1942-43) started in the fall and completed the following spring was authorized not as an experimental project. No decrease was made in the fine aggregate of the concrete mix as no thought was given at that time to the air-entrainment control feature. The district engineer took exception to the slow initial set and the low strength of the concrete. We observed the lower flexural and compressive strengths obtained on our experimental projects and saw no cause for alarm. When paving operations were continued in the spring of 1943, we recognized the Portland Cement Association design adjustment of the fine aggregate to compensate for the entrained air. Being reluctant to reduce the mortar content, we cut approximately 3 per cent of the total fine and coarse aggregate. The necessary workability and weight loss were uniformly obtained.

During 1943 and 1944, three other Vinsol resin sections of the same route were constructed. Three different brands of cement were used but the same design and the same fine and coarse aggregates were used with favorable results.

In 1944 a 45-ft. single span reinforced concrete bridge (Route 103, Section 6, Tioga County) was constructed with Vinsol resin cement. The sand was extremely coarse and the crushed gravel coarse aggregate was generally flat and elongated. Here we were unable to develop air entrainment with a reduced mortar content and again found it necessary to reduce the total fine and coarse aggregate as we did in Philadelphia. Satisfactory weight losses were obtained. The flexural strength, although lower than regular concrete, was higher than anticipated. This project demonstrated the cohesive factor of the Vinsol resin because normally, flat and elongated aggregate requires excessive mortar to hold the particles. We also observed the lack of mobility in placing the concrete in structures. Chutes were replaced with elephant trunks so that the concrete was placed at the required location instead of flowing the mass concrete. There was no segregation in any part of the structure. Internal vibration was tried and found to be of no practical value, because the cohesive properties of the Vinsol resin reduced its effectiveness. External vibration was not tried.

A third experimental pavement project (Route 238, Section 14 and Route 320, Section 2, Mercer County, constructed in 1944) contained alternate sections of Vinsol resin cement, slag puzzolan cement and normal strength portland cement manufactured by the same company, but the normal cement was from different plants. The coarse aggregate was slag and, despite sand reduction, we were unable to obtain a weight loss in the Vinsol resin concrete.

In 1945 we used 194,956 bbl. of Vinsol resin cement in various pavement and structure projects scattered throughout the state. Our concrete designs are formulated at the sight of the project by laboratory-trained, field materials engineers. Up to this time, we could not develop air entrainment in hand mixes. Then the Hercules Powder Co. produced Vinsol resin in a neutralized form (sodium resinate), first added to the cement in liquid form and now in powdered form (Vinsol NVX). The improved product provided increased lubrication. The water content was reduced slightly and entrained air is injected into our hand mixes.

We learned the reason we could not obtain the weight loss on slag concrete was because of the differences of specific gravities of the minus 1½-in. and plus 1½-in. slag coarse aggregate which are proportioned separately, and also due to variations of the specific gravities during production of the slag. The same condition occurs to a slight extent with gravel. This means we may have to make specific gravity checks at the sight of the work.

We have developed a method for determining pavement deterioration, progressively classified with each type having a successively higher numerical value. These values are applied to each pavement slab during a condition survey. The values are then factored to produce numerically a type rating for the entire pavement with zero being a perfect surface.

With reference to our Vinsol resin experimental projects we quote deterioration ratings (Table 1) based on annual condition surveys for 1943, 1944 and 1945.

TABLE 1—DETERIORATION CLASSIFICATION

	1943	1944	1945
Rt. 85, Section 10, Crawford County constructed 1940			
A-1 S.S.A. 1750-2000	16.20	18.82	22.99
A-2 S.S.A 1750-2000 +Vinsol resin	.59	1.78	2.16
B-1 S.S.A 1275-1450	11.74	17.58	19.83
B-2 S.S.A 1275-1450 +Vinsol resin	.15	1.58	2.45
Rt. 271, Section 4, Crawford County constructed 1942			
A-1 S.S.A. 1750-2000	7.77	10.09	12.01
A-2 S.S.A. 1750-2000 +Vinsol resin	0.00	0.02	0.25
B-1 S.S.A 1275-1450	0.11	0.26	0.94
B-2 S.S.A 1275-1450 +Vinsol resin	0.00	0.01	0.38
Route 238, Section 14 and Rt. 320, Section 2 Mercer County constructed 1944		1946	
Normal Strength, Plant A		1.76	
Normal Strength, Plant B		2.50	
Slag—Puzzolan, Plant B		0.82	
Normal Strength + Vinsol resin, Plant B		0.39	

The results in every project where Vinsol resin was used show remarkable improvement in resistance to deterioration.

Sometime ago we decided to learn more about our first Vinsol resin experiment so we determined the air content of the cements with the following results:

Cement	Per cent Air*
A-1 Normal Strength S.S.A.—1750-2000	5.7
A-2 Normal Strength S.S.A—1750-2000 + Vinsol resin	14.8
B-1 Normal Strength S.S.A—1275-1450	9.7
B-2 Normal Strength S.S.A.—1275-1450 + Vinsol resin	16.0

Plotting the turbidimeter analyses from 60 micron to 7.5 micron, we found that although A-1 cement had 23.82 per cent of 7.5 micron against 13.80 per cent of 7.5 micron for B-1, the percentages of the successive coarse sizes of the gradations paralleled each other. From the pavement results and the entrained air results of B-2 cement the question arises whether a slightly higher air content of B-1 would eliminate the necessity of an additive.

Cores were cut from the third experimental project approximately three months after the project was completed. The compressive strength

*A.S.T.M. C 185.

of the Vinsol resin cores was higher than with other cements. One core of each set was sawed in three approximately equal parts and permanently designated as top, center and bottom. From these we learned that the absorption of the top part of the cut cores for all types of cement was much lower than the center and bottom parts. This appears to indicate a lack of compaction. However, the comparative specific gravity of the different parts of the cut cores show fluctuations but not to the same marked degree.

Results of the freezing and thawing tests in a ten per cent solution of calcium chloride at -10 F. and thawed at room temperature developed the following results.

The normal strength portland cement concrete (Plant A) started to disintegrate at 30 cycles and was completely disintegrated at 55 cycles.

From Plant B, the concrete started to disintegrate at 40 cycles and was completely disintegrated at 90 cycles.

The slag puzzolan concrete started to disintegrate at 40 cycles and was completely disintegrated at 90 cycles.

The Vinsol resin concrete started to disintegrate at 85 cycles and at the completion of 100 cycles, 64 per cent of the concrete was still intact.

We have not investigated the efficiency of minus 50 and minus 100 mesh fine aggregate particles in the development of entrained air. Our specification for concrete design is based on the maximum solid volume of the aggregate with a maximum water content per bag of cement. The specification is broad and of necessity, permits variables to handle problems involved in using 34 brands of cement, 91 sources of sand, 222 sources of stone, 16 sources of slag and 63 sources of gravel. The design specification has been in effect for 5 years and most of the problems concerning variation in character and compatibility of material have been solved. From 1940 to 1944 inclusive, our field engineers made a total of 3446 concrete designs of which 691 had to be redesigned while in use. Vinsol resin, as an air-entraining additive, helped considerably to overcome the flat and elongated coarse aggregate problem as well as some of the fine aggregate problems, and added considerably to the workability of poor aggregate combinations. Our field men have been given directions to be followed in developing entrained air to obtain a weight loss of from 3 to 6 lb. per cu. ft., assuming normal strength portland cement concrete to have one per cent of entrained air recognized to be nonuniformly distributed. They are privileged, however, to work emperically, based on their experience with the aggregates in their sections, to obtain a practical design with the lowest water content applicable to placing of the concrete. In doing this, they will reduce the fine aggregate only or both the fine and coarse aggregate. In computing the yield, a nominal figure of 0.25 gal. of water is included as the additional amount required for non air-entrained cement.

We have tabulated the average weight loss and average per cent of air entrainment covering all designs made during 1945. (Table 2).

TABLE 2

	No. of Designs	1:2:3.50	No. of Designs	1:2:3.75	No. of Designs	1:2.5:4.5
Average Wt.—Loss Pounds						
Stone	13	4.1	14	4.3	5	4.2
Gravel	12	3.7	17	3.6	11	4.0
Slag	3	4.5	4	4.5	1	5.5
Average Weight—Loss Percent						
Stone	13	2.6	14	2.8	5	2.8
Gravel	12	2.2	19	2.3	11	2.6
Slag	3	3.2	4	3.2	1	4.1
Water Gallons —Per Bag						
Stone	16	5.08	18	5.36	8	6.41
Gravel	16	4.71	20	5.06	15	5.98
Slag	3	5.18	7	5.50	1	6.55

Personnel not being available for several years we have not studied the issue of optimum or maximum percentage of air entrainment of the concrete or of the ratio of the air entrainment of the concrete to that of the cement. At present we see no advantage in designing for a specific per cent of air entrainment of the concrete. We are, therefore, requiring a 3 to 6 pound weight loss in our revised specifications. We are in accord with specifying 16 per cent ± 4 per cent for the air entrainment of the cement rather than a percentage of Vinsol resin but, as yet, see no advantage.

To facilitate selective distribution, **separate prints** of this title (42-38) are currently available from ACI at 25 cents each—later in a book of collected papers on air entrainment to be announced. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Sept. 1, 1946

Title 42-38 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

Portland-Rosendale Cement Blends Give High Frost Resistance*

By B. H. WAIT†
Member American Concrete Institute

SYNOPSIS

Results are reported on numerous paving jobs in northeastern states, in the support of the use of portland cement blends as a means of reducing disintegration from frost action where the air entrained averaged about 1 per cent only. Results were satisfactory and the weight of the concrete was higher than for straight portland-Vinsol resin mixes.

This discussion is from the viewpoint of the manufacturers and consumers of cementing materials which, when blended with portland cement, produce the desired results uniformly in resistance to freezing and thawing and at the same time assist in the production of a more desirable concrete from all other angles.

In the northeastern section of the United States the portland-Rosendale blend over a period of ten years has produced very satisfactory results. These results in their relation to freezing and thawing have been aided by a very small amount of grinding compound or waterproofing. The amount of entrained air in the millions of square yards of roads constructed, has shown such a small loss in weight that this work should not be classified as "air-entrained concrete." The extra air entrainment in excess of that in the standard untreated portland cement concrete has averaged about 1 per cent.

On these roads which have been built under ordinary construction conditions with ordinary materials, the amount of air entrainment and the resistance to freezing and thawing has not been affected by the gradation or quantity of fine aggregate used or by any other conditions of standard construction with the exception that very high water content used in transit mix trucks has caused extra loss of weight in some cases in structural work.

*Presented at Air Entrainment Session, 42nd ACI Convention, February 19, 1946.
†The Wait Associates, Inc., New York, N. Y.

I present data from a few jobs which are well known and have been well publicized:

1) The Wellsville-Bolivar Road in Alleghany County was built partly with standard portland, the Rosendale-portland blend and the treated portland, about fifteen or twenty different cements being shipped in from a wide variety of mills throughout the U. S.

On that job the average weight per cu. ft. of the standard portland cement concrete was 149.4 lb.

The average weight of the portland Vinsol resin concrete was 145.5 lb.

The average weight of the Rosendale standard portland blend with the same basic materials was 148.3 lb.

The average loss on the Rosendale blend was 1.1 lb. as against 3.9 lb. for the Vinsol resin concrete.

The average length of life in freezing and thawing was 156.1 cycles for the blend, as against 119.3 cycles for the air-entrained portland-Vinsol.

These results, from a New York State Dept. Public Works report, indicate the greater resistance to freezing and thawing with only about 25 per cent of the air voids in the Vinsol treated cement. Four portland cements were used blended with the Rosendale. The concrete was laid at the ordinary 2-in. slump generally used in highway construction with the portland blend.

2) On the Storm King Highway in Orange County the following weights per cu. ft. are reported by the New York State Highway Dept. on samples of concrete two years old, with 6 or more portland cements used:

Average portland with 80 lb. Rosendale replacement . . . 150.24 lb.
Average portland with 94 lbs. Rosendale replacement . . 151.56 lb.
Average Vinsol resin portland . . . . 146.25 lb.

The resistance to freezing and thawing was substantially the same with the different combinations, the loss on all of them being so low that very excellent results were to be expected.

However, in this connection results in freezing and thawing were obtained with 4-5 lb. less air entrainment and air voids in the blend as against the Vinsol resin.

3) On the Idlewild Airport in New York City additional data will shortly be available, but we are able to advise at this time that the amount of air entrainment in the portland-Rosendale blend is about 1 per cent and averages no greater than the air entrainment found in the standard portland mix. On this work the strengths—both compressive and flexural—have been extremely satisfactory. Freezing and thawing tests have not been carried to destruction but have been in the freezer

sufficiently long to indicate that the concrete will have extreme durability. The average loss at 100 cycles was about 4 per cent. The results are typical of millions of cubic yards of concrete laid under standard conditions and with the low water cement ratio applicable when the Rosendale blends are used.

A summation seems to indicate that with an improved cementing material little if any grinding aids or air voids are necessary for the protection of concrete under the worst practical conditions.

During the last two years we have experimented in the field and in the laboratory with cementing materials other than the Rosendale and the results with at least one other material available in large quantities and over a wide area give full protection against freezing and thawing, with a very small amount of grinding aid if that aid is of the right quality.

On one job with this new material freezing and thawing resistance has been practically perfect with high strength and other desirable qualities with as little as one third of .01 per cent of a special grinding compound or approximately one-eighth of the average amount used with portlands.

In view of past experience is it not desirable that attempts be made to produce a better and more uniform cementing material either by blending the portland with other cementitious materials or by manufacturing a better portland cement instead of attempting to protect unsatisfactory cements with waterproofing or grinding aids with all of the attendent non-uniformity in the concrete in the field? This has been done by the New York State Dept. of Public Works for 7 or 8 years with uniformly excellent results from all angles by combining the best portland cement obtainable with a satisfactory cementitious blend. It appears that while a small percentage of air-entraining agents of the right kind is desirable, it is not necessary to rely on them entirely for freezing and thawing resistance as is proposed in the air-entraining concrete.

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Title 42-39 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **June 1946**

The Repair of Concrete: An Introduction*

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Member American Concrete Institute

SYNOPSIS

The repair of concrete structures is an engineering problem, each job containing the elements of diagnosis, treatment and execution. Diagnosis is essential to devising successful repair. Treatment may mean the correction of faults of design, materials, workmanship; protection against destructive agents and exposure; restoration of decay; or a combination of these. The execution of repair may sometimes use methods of expediency rather than logic—a compromise between what one would like and what one can do. The paper considers the more common agents destructive to concrete and is a brief introduction to an important subject.

The repair of concrete structures is an engineering problem and should be dealt with as such. In some cases the problem may be simple, in others complex, but whichever it may be, it can be resolved into a few basic elements each of which needs to be considered in any repair. This paper is an attempt to outline briefly some of these. In general, all repair jobs, whatever their nature or complexity, contain these elements:

(a) Diagnosis—the determination of the cause for the deterioration.

(b) Treatment—the choice of a method which will stop further deterioration and restore the structure.

(c) Execution—the application of the treatment adopted.

Diagnosis

It is not uncommon to find repair jobs being undertaken with little or no thought given as to why the concrete in question came to need repair. It hardly seems necessary to point out that any repair stands a better chance of being successful if it is predicated on an understanding of what brought about the deterioration, for without this it often happens that while the remedial measures adopted may restore the concrete for

*Presented 42nd ACI Convention, Feb. 20, 1946.
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a time, deterioration is likely to reoccur for the same reasons that caused the original trouble. Therefore, diagnosis is an important element in devising a successful repair.

Diagnosis presupposes the ability to determine from various phenomena, by inspection or experiment, the underlying causes for the conditions observed. Applied to concrete, this requires a knowledge of the agents that attack it and the manner in which they act. The destructive elements may be from natural causes such as weathering, from conditions of service such as abrasion, from stresses developed within the structure such as settlement or from a combination of all three. To determine which may have damaged a given concrete requires a study of the form and occurrence of the damage and the manner in which it came about.

Because the deterioration of concrete occurs in many forms and for a variety of reasons and because the same outward form may result at different times from different reasons, it is difficult to develop a wholly satisfactory classification under which the causes of deterioration can be systematically studied. Therefore, no attempt will be made here to present a classification and the groupings that follow are offered simply to facilitate discussion.

One approach to the problem would be to consider all deterioration as due to some deficiency in either design, materials or workmanship. This grouping does not take into account the deterioration that sometimes occurs solely by reason of exposure or misuse nor does it differentiate cause from effect. Both the reason of the failure and the mechanism that brought it about need to be considered before deciding on the remedial measures to be taken in any given case.

Faults of design

Consider for a moment the defects that may originate with design. Most of these cause cracking in one way or another and a study of each particular case will usually reveal which of several possible causes is to blame. And it should be kept in mind that with outdoor concrete, the structural origin of the defect may be obscured by the effects of weathering. A few of the more common sources of structural cracking are:

(a) Foundation troubles such as settlement or other movements.

(b) Insufficient reinforcement at openings as over doors, around windows, at the corners of pits, stair wells, gate openings in dams, etc.

(c) Concentration of embedded pipes and conduits in floor slabs and walls.

(d) Poor details where embedded parts such as rails, steel beams, pipes or gate checks are incorporated in the structure.

(e) The omission, improper placing or poor design of contraction and other joints.

(f) Extreme changes in section.
(g) Overloading.

Under troubles due to design may also be classed the many that result from a lack of proper drainage, either within or around the structure. It is usually accompanied by one or more of the following—efflorescence, calcium carbonate deposits or ravelling—and lack of drainage should be suspected where any of these phenomena are observed. It is well to check a structure to see if there are any pockets where water is being trapped, if drains called for in the drawings were omitted or if they have become inoperative due to plugging or other cause.

Spalling may be due either to design, workmanship or materials but the designer has the first responsibility, except where materials alone are at fault. Some of the more common causes of spalling are:

(a) *Pinching of joints.* The exposed face of a slab such as a deck or wall will often expand more than does the slab as a whole, and as a result pinching and spalling may occur along the edge of a joint.

(b) *Drag at joints.* Slabs intended to slide on the supporting abutment may fail to do so, and if the binding occurs near the outer edge of the bearing surface it may spall the edge of the abutment below. This can also occur if proper care has not been taken in the construction of the joint.

(c) *Rusting reinforcement.* There may be spalling due to rusting of the embedded steel if it has not been provided with a sufficient cover of concrete for the exposure encountered. Similar spalling can be due, also, to a poor grade of concrete, the result of careless workmanship.

(d) *Popouts.* When unsound aggregate particles lie close to a concrete surface they will, with certain exposures, cause unsightly "popouts" or spalls. These will be bigger and deeper the poorer the concrete or the larger the size of the aggregate involved.

Faults of materials

The quality of concrete is dependent on the quality of its constituents which are cement, aggregate and water. Defective cement or aggregate usually cause volume changes that are evidenced by pattern cracking, more or less conspicuous. Although not strictly true, in general the quality of the mixing water seems to have little influence on the durability of the hardened concrete in which it is used but the quantity of mixing water is most important: the latter is a matter of workmanship and will be discussed under that heading.

Defective cement is a relatively rare cause of deterioration and when it occurs, usually results in complete failure or is accompanied by symptoms so unusual as to suggest the need of a thorough investigation. Some authorities blame nearly all defective concrete on deficiencies in the cement used; others claim that only a few failures can be traced to this cause. Undoubtedly the truth lies somewhere between. Cement is probably a secondary or contributing factor in many cases for which other causes are assigned. But it acts only as other factors, such as poor workmanship, make a concrete liable to attack by the normal agencies of destruction. Insofar as the diagnosis of concrete ills are concerned, unless gross volume changes are encountered cement as a cause of deterioration can be largely ignored and its effects considered under other headings.

Stanton has pointed out a type of failure in which cement does play an important role. Cements high in sodium and potassium, when combined with certain types of silicious aggregates, have been found to cause concrete to expand, resulting in conspicuous cracking. This expansion is accompanied by other well-established phenomena such as the formation of certain reaction products of which sodium silicate is one. The reaction has been given much publicity in the technical press and the reader is referred to the many excellent articles in the JOURNAL and elsewhere for details of its occurrence and methods for its identification. In localities where the cements are high in the alkali components and where potentially dangerous types of aggregate might occur, any unusual cracking or other evidence of volume change should be investigated with this in mind.

Expansion similar in appearance to that just discussed, may be caused by the aggregate alone. It is usually due either to unsoundness of some of the aggregate particles or to major differences in the coefficient of expansion of the aggregate and the surrounding concrete. Some spalling may accompany unsound aggregate and give a clue to the cause but differences between the coefficient of expansion of the aggregate and concrete are harder to detect since the aggregate itself is sound.

Poor grading might also be considered a fault of the aggregate and strictly speaking it is, but it affects the concrete not through any lack of quality of the particles themselves but because of an increase in the porosity of the concrete in which it is used. The defects that arise from this cause are similar to those discussed under workmanship and will be considered there.

Faults of workmanship

Defects caused by workmanship are many and varied; they are also by far the most numerous and account for or contribute to probably ninety percent of existing deterioration. Any omission, inaccuracy or carelessness in any of the steps that go into the preparation of the materials, the proportioning, mixing, placing or finishing of concrete or its protection afterward has a deleterious effect on its quality and makes it less able to resist weathering or other destructive agent. It is without the scope of this paper to discuss in detail the many faults of workmanship; they have been described many times in the pages of the JOURNAL, but for the sake of completeness they will be briefly listed.

(a) *Poorly graded aggregates.* Either inherent in the aggregate itself or brought about through segregation in handling or in stockpiles and bins.

(b) *Improperly proportioned mixes.* Either due to a poor mix design or inaccurate measurement of the materials.

(c) *Excess water.* This could be classed under (b) but because it is a major cause of poor concrete it has been listed separately. No single defect of workmanship has caused so much inferior concrete as the use of an unnecessary amount of water in mixing.

(d) *Insufficient mixing.*

(e) *Segregation or separation of the constituents of the concrete.* It takes place during handling and placing of the concrete and may be due to carelessness in these operations. It may be made almost a certainty, even if carelessness in placing does not occur through any of the faults of workmanship listed from (a) to (d).

(f) *Over-manipulation.* Either through excessive puddling, vibration or trowelling or from finishing at the wrong time.

(g) *Lack of curing and protection.* Such as allowing the concrete to dry out too soon, permitting it to freeze, etc.

In general, it may be said that faults of workmanship do one of two things: either by increasing the porosity of the concrete make it more susceptible to deterioration, or by interfering with the hydration of the cement weaken it and make the concrete less able to resist attack. Thus, with this class of faults it is more important to know the forces acting to destroy the concrete and the way they act than to determine which of several faults may have given rise to the particular defect to be corrected.

Destructive agents

An entire paper could be devoted to a description of these agents and how they act, but for our present purposes it will be sufficient to indicate the more important. For simplicity they will be divided roughly into three groups: those that act on the surface of the concrete, those due to moisture movements within the concrete and those that cause volume changes.

Under the first group, those that attack concrete at its surface are erosion, abrasion and corrosion. Each of these brings about a wearing away of the surface; the first by wind and water, the second by abrasive or mechanical means and the third by chemical action. To these could be added scaling caused by frost acting on a surface that has been overworked.

Moisture movements require that concrete contain pores or channels through which water or vapor can pass. Movements may be brought about by absorption or evaporation, they may be due to pressure forcing water from a wet to a dry surface as in a wall with water on one side, or moisture may be drawn from a wet to a dry face by the forces of evaporation and capillarity. The moving water dissolves some of the constituents of the hydrated cement, thus weakening the concrete, and it may deposit the products of this solvent action beneath the surface so as to cause scaling. If also the moving water contains corrosive chemicals, the rate of attack is greatly accelerated and if the pores become saturated at a time when a falling temperature causes the water in them to freeze, the concrete may be destroyed through frost action.

The forces causing volume changes are many. Temperature is one, loss of moisture is another, defective materials a third, and another, chemical changes such as that brought about through the action of the sulfates of sodium and potassium on concrete or the alkali-aggregate reaction already mentioned.

Exposure

One other factor needs to be considered before passing on to the question of repairs, namely that of exposure. Structures, in contact with moisture on one side only, such as dams or retaining walls, are subject to a much more severe exposure than those normally dry, such as the superstructure of a building. The south and west faces of dams and other water-impounding structures are, on this continent, more liable to deterioration than those facing north or east; and in the same structure, horizontal surfaces more than vertical surfaces. The severity of an exposure can be roughly gauged by the opportunity offered to the concrete to become saturated with moisture and the number of annual cycles of freezing and thawing to which it is subjected. Time is an important element of exposure. For instance, surface scaling in a structure only a couple of years old would indicate a concrete of poor quality, a very severe exposure or both; the same degree of scaling in a concrete twenty-five years old would not be very significant.

Methods of repair

Let us consider in a general way, the problem of repair. It seems to the author that any repair should

(a) Correct structural defects, if any, and strengthen the structure, if weakened.

(b) Eliminate, if possible, the sources of deterioration.

(c) Restore the decayed parts of the structure to its original form.

(d) Protect the concrete, if necessary, to prevent further damage.

It hardly seems necessary to argue the desirability of correcting structural defects, if serious, prior to reconditioning the concrete. The problems likely to be encountered are many but they are structural rather than a matter of the quality of the concrete, and will not be discussed here. In general, it may be said that they require the highest engineering skill for their solution.

Elimination of the sources of deterioration is not always possible. It is obvious that a structure cannot avoid the normal hazards of service such as natural weathering. But in many cases it is possible to remove the sources of deterioration. A typical example of this would be faulty drainage, one of the more common faults encountered. It is often possible to intercept water reaching a structure by rearranging the drainage in the surrounding area so that the concrete will not come in contact with it. Water trapped in the structure can be piped away and not allowed to seep through cracks, to run down surfaces or to remain on floors, platforms, or decks. But if the cause of deterioration cannot be eliminated, then the structure must be otherwise protected.

Restoring the decayed parts of a structure may be either simple or difficult. If all that is involved is the removal of the defective concrete and its replacement, the problem is simple and its execution only requires the application of well-established methods of construction. But if it requires the removal of load-bearing sections or of keeping structures such as a dam or bridge in operation, it may become a major engineering problem. In the latter case, the methods of repair have often to be those of expediency rather than logic—a compromise between what one would like and what one can do.

Concrete can be protected from damage in various ways. One is to eliminate the cause of deterioration but, as has been pointed out, this is not always possible. In such cases, re-surfacing of the affected concrete offers a solution, using pressure-applied mortar, plastering or concrete cast in place. Usually the thicker surfacings are the more permanent. Such surfacings should be reinforced and dowelled to the structure they cover. Care must be taken to prevent water from being trapped behind the surfacing else the results may be disappointing. Re-surfacing can be carried a step further until the original structure is totally encased; where this protects the original concrete against further attack, it is a satisfactory method of handling many jobs, especially with massive structures. Leaky dams and other water-impounding structures require that some means of cutting off the seepage be provided. A cut-off wall or blanket of concrete on the upstream side is a very satisfactory solution to such a problem, although not always the cheapest. Where the structure is part of a storage system or hydro-electric development, this is not always possible in which case grouting of all joints and fill planes will usually dry up the structure to a reasonable degree.

In closing, the author would repeat that this paper is not intended to be either complete or exhaustive but only an introduction to an important subject which to an increasing extent is engaging the attention of engineers throughout the continent. Its purpose will be served if it helps engineers to a better understanding of the problems of deterioration and repair and causes them to study the many admirable examples of successful work that have been reported in the literature wherein the principles outlined have been applied.

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Title 42-40 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE Vol. 42

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN June 1946

Behavior of Concrete Structures Under Atomic Bombing*

By E. H. PRAEGER†
Member American Concrete Institute

SYNOPSIS

The destruction wrought by atomic bombing of the Japanese cities, Hiroshima and Nagasaki, August 1945 is outlined, with an analysis of typical damage within areas with respect to "zero point." The survival of certain modern buildings of reinforced concrete and composite construction is noted with interest. The paper discusses principles and procedures of design necessary to resist attacks by these special new weapons.

INTRODUCTION

Shortly after the end of the war in Japan the Chief of the Bureau of Yards and Docks recommended to the Secretary of the Navy that a group of qualified officers and technical experts be assigned to special duty to survey damage wrought by the atomic bombs and other weapons on targets in Japan. It was anticipated that criteria would be developed from the observed facts for the design of military structures that would withstand present and future special weapons.

Surveys were made of targets that had been attacked by atomic bombs, high explosive bombs of various sizes and incendiary bombs. Targets of bombardment by shells of naval guns were not visited because another Navy group had been assigned to these duties. However, information concerning the physical damage inflicted by this weapon was received from that group.

In addition to the data from inspections of damaged structures, considerable information of interest was obtained through interviews with Japanese who were in the target areas at the time of the bursts and with city officials, architects, engineers, contractors and owners of

*Presented 42nd ACI Convention, Feb. 20, 1946.
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damaged buildings. Original plans of many of these buildings were obtained from the designers and several objects and samples of representative materials were shipped to the United States for tests.

ATOMIC BOMBS

Two atomic bombs were released over Japan, one over the city of Hiroshima and the other over Nagasaki. Both bombs were detonated in the air.

Hiroshima

The Hiroshima bomb was released at 8:15 a.m. on August 6, 1945 over the center of the city.

Hiroshima was a commercial center situated in a delta at the mouth of the Ota River. It comprises five islands extending in a north-south direction and the adjacent mainlands. The city proper is approximately four miles wide and five miles long, the various islands and mainland being interconnected by many bridges. The topography is level within the city limits but hills rise to an elevation of about 800 feet to the northwest and northeast.

While there is a shipyard and several industrial and military facilities on the outskirts of the city, the center contained many parks, shrines, residences, schools, military reservations and a considerable number of multi-story commercial buildings. The population in July 1945 was approximately 350,000.

It has been stated that the explosive peak pressure of this bomb was equivalent to that of 10,000 tons of TNT. Observation of the damage indicates, however, that this figure may be high. The location of the burst was determined by projecting edges of flash burns, defined by shadows of intervening objects, and found on numerous timber and other objects, to a point of intersection. The point on the ground directly below the burst has been designated as the "zero point".

The explosion has been described as a red flaming meteor-like ball of fire; thereafter a great sudden pressure and heat; then noise. A cloud of dust filled the area, and rain fell in torrents in some sections of the city, accompanied by a wind.

Fires of secondary origin sprang up in all directions and burned for a considerable period as no attempt was made to extinguish them. The momentary heat at the ground surface has been estimated, by the Japanese, to be in the range of 2000 C. Tests of sample roof tiles exposed to the blast heat are now in progress at the National Bureau of Standards.

Approximately one quarter of the population were killed immediately or died within the next several weeks, one quarter suffered injuries but survived and, of the remaining half, many lost their homes and became ill because of exposure and other hardships.

A medical team reported that no radioactivity was found in the ground two weeks after the explosion.

In general appearance the center of the city resembles the completely burned out areas of Tokyo and other Japanese cities which were targets of large-scale incendiary raids. Destruction of small, lightly constructed dwellings and business structures was complete, and fires consumed practically all such buildings within an area having a diameter of two miles with the zero point as a center. Light steel and timber structures collapsed or suffered various degrees of failure in an area of approximately ten square miles.

Brick, concrete block and stone masonry structures were completely destroyed within an area extending some 5,000 feet from the zero point.

The majority of the modern earthquake proof buildings of reinforced concrete, fireproofed structural steel, or composite construction (of structural steel and reinforced concrete) at distances greater than 2000 feet from zero point suffered little or no damage to the main structural parts. Closer to zero point a number of these buildings suffered some structural damage. Few such buildings collapsed but many were gutted by fire.

Bridges suffered varying degrees of damage, some by forces lifting the deck. Piers and other water front structures were outside the sphere of influence of the bomb.

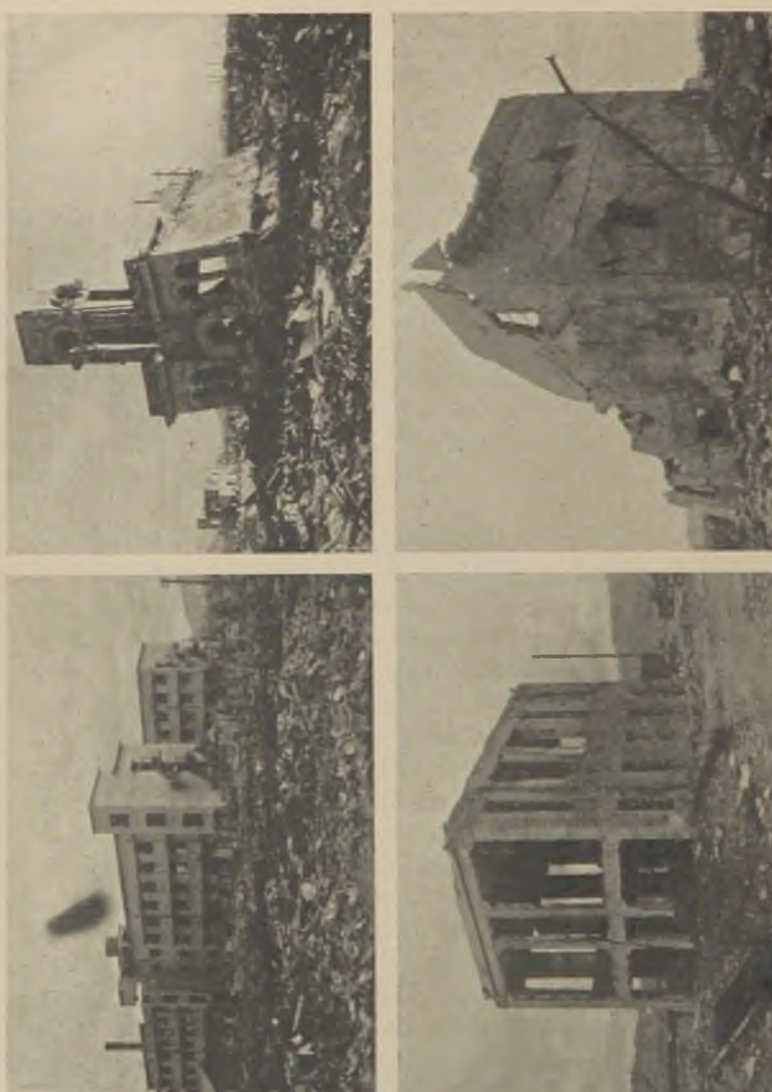
Reinforced concrete chimneys suffered relatively little damage. Underground utilities generally were not damaged but above ground wiring systems were put out of service because of failures of supports. Steel tanks containing gas and liquid fuels were damaged beyond use at distances of 6000 feet from zero point.

Few trees were blown over but those near the zero point had been burned and in many instances heavy branches were broken.

The effects usually connected with high wind pressures were not evident except at the fringes of the devastated areas. While doors and windows on surfaces exposed to the blasts were generally blown in, many cases of inward, rather than outward pressure were noted also on the leeward side of structures. This condition has also been observed with high explosives; the velocity of the blast wave is so great that the pressures occur almost simultaneously on all sides of an obstruction. Evidence of downward pressure on roofs near the zero point was common.

The survival of the many modern buildings of reinforced concrete and composite construction of from 3 to 10 stories, is of extreme interest. While roofs of buildings close to zero point were deflected downward

Fig. 1—Hiroshima after the atomic bomb: a (upper (left), Red Cross Hospital; b (upper right), Shimomura Watch Co.; c (lower left), south view of Bussan Shohin Buretsu Kan; d (lower right), Hiroshima Ice Manufacturing Co.



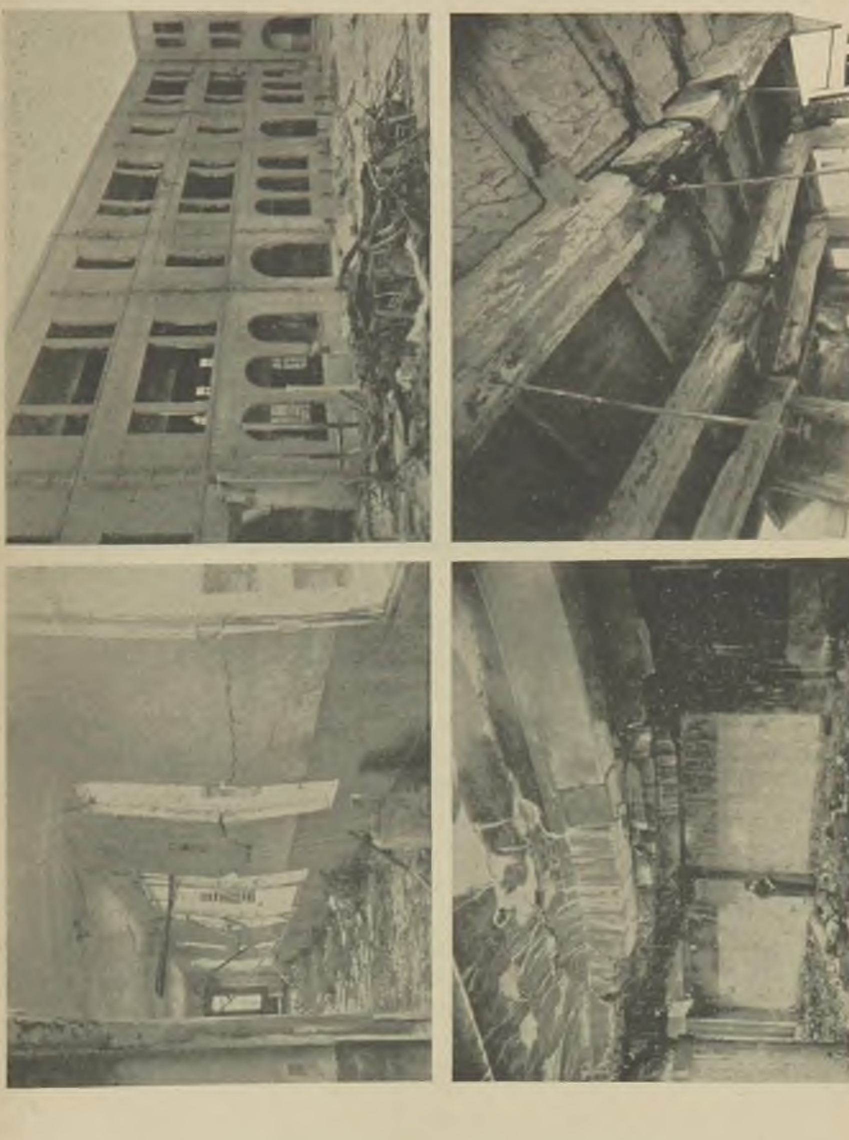


Fig. 2—Hiroshima structures after the blast: a (upper left), Honkawa National School; b (upper right), Honkawa National School; c (lower left), Fuel Distribution Building; d (lower right), roof of Bussan Shohin Bureto Kan.

and local failures were experienced in many members, the majority of these buildings can be restored to their original condition. In these buildings fires, probably most often started by burning embers from adjacent buildings, caused considerable damage. It has been reported that most fires in fireproof buildings did not start until about twenty minutes after the burst.

Fig. 3—Hiroshima Teikoku Bank, after the blast.

Nagasaki

Nagasaki is an industrial city with a reported population of approximately 250,000 in July 1945. The city lies on both sides of Urakami River and Nagasaki Bay into which the Urakami flows. The center of the city is approximately at the mouth of the river and the city extends from this point about two miles in a northerly and two miles in a southerly direction. Hills, approximately 1200 feet at their highest peaks, parallel the river on both sides. The valley varies in width from one half mile at the north to two miles at the south. Industrial plants, business sections, residential areas, schools, hospitals and other facilities were located in the valley on both sides of the river.

The bomb burst at 11:00 a.m. on August 9, 1945. This bomb was of a type different from that released over Hiroshima and was equivalent in characteristics to a theoretical bomb with a 20,000 ton charge of TNT. The appearance of the burst, insofar as could be learned from discussions with Japanese, was similar to that for the bomb released over Hiroshima.

The general appearance of the section of the city, for a distance of one mile north and one mile south of zero point, is ghost-like. Burned ruins predominate. Yet many permanent type buildings survived with comparatively little structural damage. Destruction extended in a northerly and southerly direction for a distance of approximately two miles from the zero point but was confined in an east-west direction to the area between the hills paralleling the river and bay.

In an area within 7,000 feet of zero point all small frame buildings, dwellings, stores, etc. were completely destroyed leaving only foundations and broken roof tile. The remains of these buildings were destroyed by fire. Trees were tilted in the direction away from the bomb and many were uprooted. Doors and windows, including frames and glass were blown inward on the side facing the bomb, but as a rule outward on the opposite side. Contents of buildings, except heavy machine tools, showed evidence of having been blown about. Burning of inflammable materials in fireproof buildings was widespread but not complete. Wood floors, interior trim, furniture, etc. were found to have been entirely consumed in one room and unburned in the next which leads to the conclusion, confirmed by eye witnesses, that many fires, particularly those in fire-proof buildings, were secondary rather than primary.

As would be expected, heavily built steel structures suffered damage to a lesser degree than the comparatively light factory type steel buildings. Light steel and timber structures, were severely damaged by blast pressure and primary or secondary fires. Many buildings of this type were deflected bodily and lean with an inclination of as much as 30 deg.

There was no crater nor evidence of any great disturbance in the ground under the point of detonation.

Deaths and injuries to personnel in the exposed area were in about the same proportions as those noted for Hiroshima. However, a smaller percentage of the total population of the city was in the exposed area.

EFFECTIVENESS OF ATOMIC BOMB

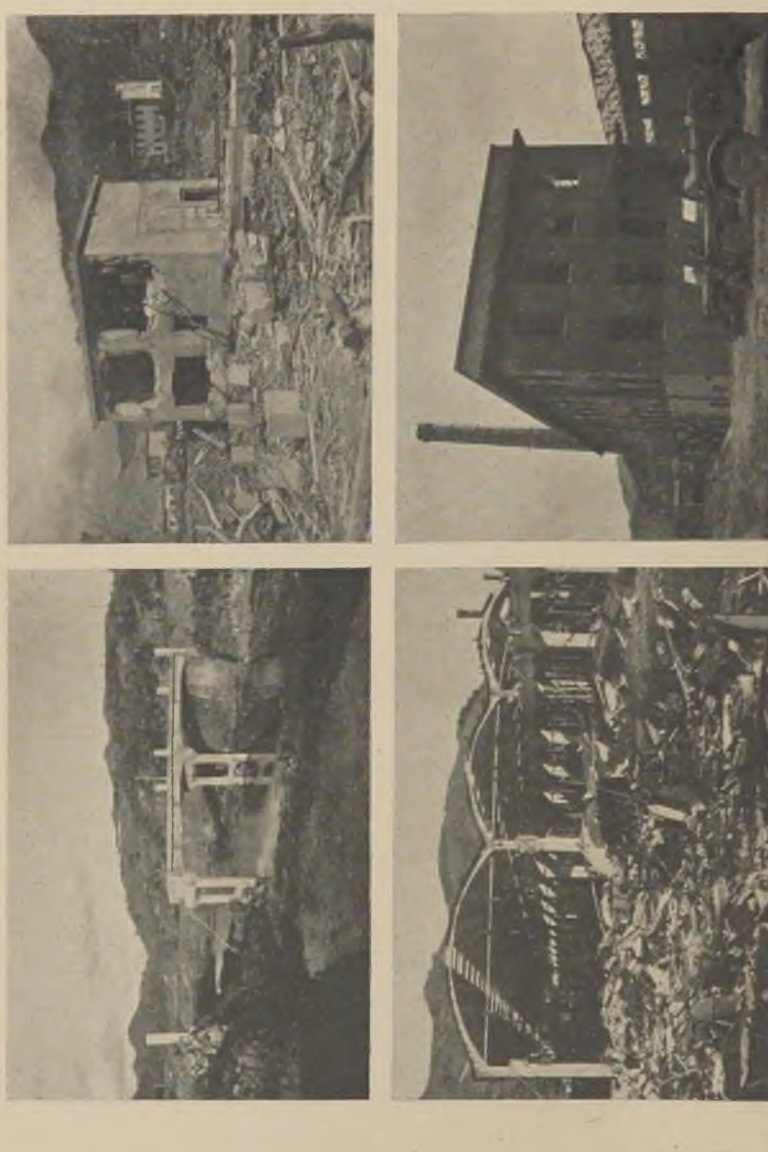
The atomic bombs damaged structures by reason of:

(a) Great pressure, causing collapse or structural damage.
(b) Extreme heat, causing fires.

The atomic bombs produced casualties to personnel by any of the following causes or by any combination of them:

(a) Injuries from collapsing structures.
(b) Flying debris.
(c) Fires.
(d) Great pressure.

Fig. 4 — Atomic bomb aftermath at Nagasaki: a, (upper left), Bridge about 2500 feet north of zero point; b (upper right), Building at Nagasaki Medical School; c (lower left), Building at Mitsubishi Steel and Arms Works; d (lower right), Mitsubishi Shipbuilding Co. office building.



(e) Burns by direct heat rays.
(f) Internal injuries by penetrating "gamma" rays.

The atomic bomb disrupted public utilities over a considerable area. Overhead telephone, light, power and trolley wires were damaged beyond use. Automobiles, trucks, trolley cars and railroad equipment were damaged in varying degrees. Machine tools were dislocated from their bases and damaged by falling debris and subsequent exposure, but generally not beyond repair.

ANALYSIS OF TYPICAL DAMAGE FROM ATOMIC BOMBS

As many of the well built structures in both Hiroshima and Nagasaki, particularly those of composite construction, withstood the blast pressure with comparatively little damage, the question naturally arises: why did these buildings not fail? After a study of many of those structures, one finds some plausible answers to this question.

The spectacular case of the reinforced concrete chimneys, the majority of which survived at all distances from the blast, will be considered first. These structures are of simple standardized design, which probably included allowance for earthquake forces. They are usually built by specialists and are therefore of relatively good construction. Being circular in plan, they offer minimum resistance to the blast wave. Their periods of vibration are relatively long. A common size chimney has an average diameter of 5 ft. The blast wave travels at a rate in excess of 1000 feet per second and therefore the wave traveled from the directly exposed surface to the back in about 1/250 of a second, at which time the entire structure is subject to approximately equal pressures in opposite directions except that there is a secondary effect in the form of a gust of high wind. While the period of the blast is relatively long, it is probable that the impulse passes before the chimney which has been set in motion or has had time to reach the amplitude of its swing. These conditions explain, in general terms, why the concrete stacks withstood a much greater transient load than would be possible were the force an equal static load.

It has been noted that well built reinforced concrete buildings, and buildings of composite construction (there were no multi-story steel frame buildings) not far from the zero point, survived except for damage to some structural parts. In most cases the buildings are well designed, earthquake forces have been included and the materials and construction are adequate. An average size for this type of building might be 100 ft. square and 40 ft. high. The period of vibration of the structure as a whole is longer than are those of most of the component units (slabs, walls, etc.). The structure has a much greater relative depth than that of the chimney, yet it requires approximately 1/12 of a second for the blast wave to travel from the windward to the leeward face. If the exterior surfaces had no openings and if they did not fail, the external pressures on the structure as a whole would be approximately balanced in 1/12 of a second. The resultant pressure on an outside wall or roof would be the difference between the interior pressure and the blast pressure. If this difference is great enough, the units of insufficient strength would fail but the building as a whole would not necessarily suffer. Under this condition the floor slabs, beams and girders act as compression members, stressed by the external horizontal load and must be of sufficient strength to withstand these forces. Since these parts of the structures are designed for the generally more severe condition of bending, it is entirely possible that they would be strong enough to resist this horizontal thrust.

As a matter of fact, there were relatively few totally enclosed structures, and most of the buildings have a large percentage of window surface. The windows and frames, being relatively weak, are blown in instantly on the windward side. They may be blown out or in on the leeward side depending on whether the blast from the front (through the openings in that wall) or from the back reaches the rear surface first, and both conditions were noted in Japan. The loads on the roof members and on the floors above grade are then equalized by approximately equal pressures above and below except in the case of the floor directly over an enclosed basement, which is subject to unbalanced blast pressure from above. Cases of failure from downward pressure were noted at first floor levels. Cases of both downward and upward failures were noted in roofs although most roof failures, particularly in buildings directly under the blast, were downward. Few failures were noted in floors between the roof and first floor, but this would be possible if the wall openings in one floor were materially different from those in adjacent stories.

PRINCIPLES OF DESIGN

The design of structures to withstand atomic bombing presents many problems. Permanent buildings would necessarily be of fireproof construction and, therefore, the hazard of fire damage to the structure proper would be eliminated. Extremely vital structures that must be constructed in the open should be designed to withstand direct hits of high explosive bombs and shells and the blast pressure of atomic bombs.

The strength and thickness of structures which will be required to resist heavy HE weapons would probably be sufficient to withstand the blast pressures from atomic bombs which detonate at relatively high altitudes.

It is not expected that an atomic bomb would be used to destroy an isolated target by fusing it to detonate at close range. It is recognized, however, that this is a possibility, either by intent or by error.

The characteristics of atomic bombs must be considered in designing "atomic bomb-proof" structures. Unlike the loads used in the design of conventional structures, blast pressure is a transient load, and the mass and vibrational characteristics of the proposed structure must be considered in the design.

If the duration of the impulse is less than one quarter of the period of vibration, the impulse will merely start the swaying of the structure, and the structure will not reach its amplitude of movement until after the cessation of the impulse; the strains occuring when the amplitude is finally reached will be only a fraction of the strains resulting from equal static loads. On the other hand, if the duration of the impulse is several times greater than the period of vibration, the structure will reach its greatest strains or nearly its greatest strains before the impulse is over, and these greatest strains are comparable in magnitude to those resulting from equal static loads. In general, the square of the period of vibration is proportional to the mass and inversely proportional to the stiffness, (so that if the mass is doubled without changing the stiffness, the square of the period will be doubled, and if the stiffness is doubled without changing the mass, the square of the period will be cut in half).

While these premises might lead one to strive for flexibility and mass, it must be remembered that the requirement of thickness for strength works against flexibility. Thickness is also required to shield persons inside.

The individual component units may all have different periods of vibration; all are probably different from the period of the structure as a whole.

The structural engineer's problem is to meet these conditions. One phase of the problem lies in structural dynamics rather than structural statics.

Allowable working stresses may be increased for blast pressure loads.

PROCEDURE OF DESIGN

The planners must decide what structures are to resist attacks by these new special weapons, what forces should be used in designing the structures and what degree of protection is to be afforded. It will obviously not be feasible to design any structure to resist an atomic bomb which detonates at extremely close range.

The structural designer has many problems to solve which do not occur in buildings subject only to static or "equivalent static" loads. It will probably be necessary first to design the structure to meet the usual requirements plus special conditions imposed by the bomb, and then to analyze the resulting structure to determine whether or not it meets the dynamic conditions. The first design will probably require revisions to satisfy the latter conditions. While this is a relatively laborious task, it is not an impossible one when all the requirements are known.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 42

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN June 1946

Job Problems and Practice

Five cash awards—$50.00, $25.00, and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1945 to June 1946.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved —in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authcrity; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

What Kind of Cement Stucco? (42-174)

A member of the Institute commenting on the fact that very little has been done recently on the subject of "stucco", makes a suggestion that seems highly discussable. Discussion might indicate what lines might be followed up in new study of stucco. Here is his suggestion: "It has been our experience that a masonry cement of the type now marketed by a considerable number of cement manufacturers offers a better stucco material than straight portland cement. These masonry cements contain roughly 50 per cent of blending material, usually ground limestone, together with air-entraining agents. Both experience and theory indicate that such cements will give more satisfactory results than the excessively strong, rigid, and brittle, straight cement mortars".

Setting Heavy Machinery on Concrete Bases* (42-175)

by R. R. KAUFMAN†

The discussions of the subject of grouting machinery bases in the February 1945 and June 1945 issues of the Journal have been read with great interest. The need for improved grouting methods has long been recognized. That there are wide differences in grouting requirements which vary depending on loads, vibration, impact and whether the grout

*See also contributions on same subject (41-164) in February and June 1945 JPP sections.
†Chief Engr., The Master Builders Co., Cleveland, O.

will be subject to oil, water, heat, or other conditions which can affect its stability, is also known.

Different grouting operations have employed materials such as: Sulfur, lead and sulfur, lead and antimony, cement, sand and aluminum powder, and magnesium oxychloride cements.

Sulfur, lead and sulfur combined, and lead and antimony combined, have an advantage in that they may be melted, poured in place, and will harden immediately upon cooling. The lead-sulfur and lead-antimony combinations produce "shrinkproof" grouts. Sulfur alone or in combination with lead fractures rather easily under impact. Lead-antimony will withstand considerable impact. However, because it is ductile it tends to flatten out and creep under continued vibration and impact, and does not recover or expand again to take up the space caused by such impact.

The above materials are usually employed only in minor operations because of (1) difficulty in the case of the sulfur compounds to heat them without burning, (2) danger to the workman in handling the hot molten material and danger of spattering if it comes in contact with water, (3) surrounding concrete must be dry and (4) only small amounts may be poured at a time, otherwise there is danger of disrupting the adjacent concrete by driving off water of hydration.

Aluminum powder is sometimes employed in portland cement grouts to produce expansion by its reaction with the alkali content of the cement which produces hydrogen gas.

This type of grout may be properly designed and controlled by a technician and is satisfactory for grouting equipment with relatively low loadings not subjected to vibration or impact.

Magnesium oxychloride cements have been employed on a minor scale. They have the advantages of acquiring high strengths and of expanding, and the disadvantages of being brittle and soluble in water.

Rust joint iron

Many railroads have found that under bridge seats, turntables, etc. they cannot procure sufficient strength and durability in the time available from grouts made with cement and sand, either poured or packed dry. Therefore, they employ what is termed "rust joint iron" for emergency use in these areas. Rust joint iron was originally prepared by some railroads by mixing cast iron borings from their machine shops with small amounts of ammonium chloride and sulfur. A small amount of water was mixed with this material which was then caulked in place. This produced an expanding, highly ductile grout which could carry loads immediately, and gave satisfactory service under loads, vibration and impact. The difficulties in preparing this type of grout are:

(1) Clean, graded iron borings with low or zero oil content are hard to procure. If too much oil is present the rate of oxidation is slowed to such an extent that failure is likely.

(2) This type of material requires the grout to be under compression at all times. Since in bridge seats and similar grouting work the edges of the grout are unrestrained, the continued oxidation of the unrestrained surface permits ravelling and ultimately causes failure.

(3) Rust joint iron must be caulked in place to be effective as a non-shrink, ductile grout capable of withstanding high loads. This requires competent, semi-skilled labor, and increases supervision, and initial cost as compared with grouting with flowable mixes.

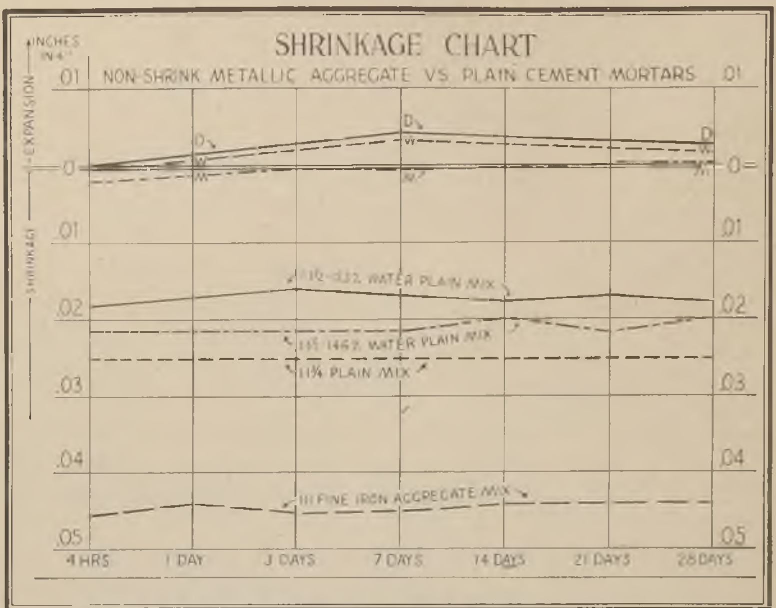
Home-made preparations have about disappeared since most railroads have found it more economical and convenient to employ carefully prepared rust joint iron manufactured by companies specializing in this field.

About 1925 the engineer of a large steel mill employed some metallic floor hardner instead of sand in a cement grout, under some narrow bed-plates, and found that it produced in the grout a ductility which greatly increased its resistance to the impact of strip mill operations. This led to the development of an iron aggregate of proper size and grading for grout, combined with materials to promote oxidation and expansion of the metallic particles, and later with water-reducing agents to allow placement with lowered water cement ratio.

In practice, the metallic aggregate is mixed with portland cement and aggregates to a flowable consistency. The grout is then flowed under or around the equipment being grouted, without caulking. Properly used it produces a non-shrink, ductile grout, of high strength to resist vibration and impact. Successful elimination of shrinkage depends in part on the proper oxidation and expansion of the metal particles, and in part on the water reducing agent, which cuts the amount of mixing water required.

This grouting procedure has proved to be practical and economical in large and difficult operations. The service record over 15 years shows no work being replaced due to grout failure.

Non-shrink metallic aggregate should be distinguished from metallic waterproofing or finely divided iron when used for grouting purposes. Finely divided iron, having considerable more surface area, requires an increased quantity of water to maintain workability, which in turn causes additional settlement or shrinkage. This is illustrated in the curves on shrinkage made by Professors Voss and Adams of Massachusetts Institute of Technology, shown in Fig. 1.



D —— 1:1 NON-SHRINK METALLIC AGGREGATE DRY MIX
M—·—1:1 " " " " MEDIUM MIX
W---1:1 " " " " WET MIX

Fig. 1—Shrinkage chart

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Vol. 17 No. 6 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN June 1946

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Design of dowel bars in road and runaway joints

J. N. McFeeters and E. T. Hanrahan, *Concrete and Constructional Engineering*, V. 40, No. 2 (Feb. 1946) pp. 27-39
Reviewed by Glenn Murphy

The analysis presented in this paper is based on Professor H. M. Westengaard's treatment of groups of rigid dowels, but is extended to include Mr. B. F. Friberg's analysis of single flexible dowels. Examples are worked out and practical considerations discussed.

Allowance for inclined compression in reinforced concrete beams

A. J. Ashdown, *Concrete and Constructional Engineering*, V. 41, No. 3, (Mar. 1946) pp. 75-81
Reviewed by Glenn Murphy

The author presents formulas for the design of shear reinforcement in beams. The "treatment is based on Dr. Oscar Faber's work on the theory of inclined compression." The results contain simplifying assumptions, such as the neutral axis being located at 0.39 of the depth from the top.

Soil movements as affecting paved surfaces

W. H. Elgar: Surveyor, London, 1944, 103 (2746), 427-8; Publ. Wks., N. Y., 1944, 75 (12), 17-8, 46-7. *Road Abstracts*, Vol. XIII, No. 2, February 5, 1946
Highway Research Abstracts

Soil movements causing cracking of road surfacings parallel to the edge of the road are discussed. Fluctuations in moisture content under the haunches cause expansion and contraction of the subgrade. The moisture content under the center of the road remains fairly constant and hence volume changes are small. Frost heaving, in the author's experience, is more severe in gravel than in other subsoils. The drying and shrinkage of the subgrade under the edges of a road caused by moisture being drawn out of the surrounding soil by vegetation is stated to be the cause of longitudinal cracking of a road during a drought. Methods of preventing cracking due to the above causes are discussed.

The reconstruction of the Longeray Viaduct

Leslie Turner, *Concrete and Constructional Engineering*, Vol. 41, No. 3, (Mar. 46) pp. 63-68
Reviewed by Glenn Murphy

The largest work undertaken in France since 1940 is the reconstruction of the Longeray single track railway viaduct across the Rhone about 15 miles from Geneva. The new reinforced concrete structure, designed by Société des Entreprises Limousin in conjunction with the railway, consists of three parabolic arches with an approach of three semi-circular arches to match the portion of the bridge which was not destroyed in 1940. The outer arch has a span of 208 ft. and a rise of 188 ft., is hollow, and has a rectangular cross section. Maxium design stress in the concrete was 1060 psi. Forms for each half arch were constructed vertically at the piers and rotated at the springing until they met at the crown. A total of 13,000 cu. yd. of concrete and 550 tons of steel were used in the construction.

Responsibility for defective construction

G. Magnel, *Concrete and Constructional Engineering*, Vol. 41, No. 3, (Mar. 46) pp. 69-73
Reviewed by Glenn Murphy

Professor Magnel of the University of Ghent describes a building insurance plan which has been in operation in Belgium for 10 years. The program covers construction defects and risks of damage to adjacent structures for a 10-year period. The organization, Bureau S. E. C. O., is a cooperative body of organizations of contractors, consulting engineers, and architects, and functions under a technical council of ten. The director is a civil engineer. The total coverage of the Bureau is about $50,000,000 and premiums are about 1.2 per cent. While the organization does not function as a consulting group, it has in effect become an advisory group to most of the civil engineering construction in Belgium. The author states that the scheme has proved successful in Belgium.

Discussion of the plan by Sir Alexander Gibb, Dr. Oscar Faber, and A. E. Butland with references to adoption in Great Britain are included.

The influence of aggregates type and of age on the thermal-expansion of concrete

F. Keil and H. G. Tempel: *Zement*, 1940, 29 (20) 248-53, Road Abstracts, Vol. XIII, No. 3, Mar. 5, 1946
Highway Research Abstracts

In the laboratory experiments described, the thermal expansion was measured of concretes made with different types of aggregates, at temperatures of about 5 to 45C. All concretes had a consistency suitable for road surfacing, and a cement content of 560 to 590 lb. per cu. yd. (330 to 350 kg/m^3). The principal results were as follows: (1) The expansion was greatest for Rhine gravel, lump slag and Piesberg sandstone, and least for limestone. (2) The expansion of concrete made with gravelly aggregate was very close in value to figures quoted for the expansion of gravel alone; as the expansion of the aggregate itself decreased, the concrete expansion also decreased, but not in proportion. (3) The expansions at 7 and 28 days were appreciably less than at 90 days. (4) When curing takes place under water, the expansion of the aggregate appears to determine the expansion of the concrete. When curing takes place in air the percentage of air voids and the setting process are also important.

Prestressed concrete

G. Magnel, *Concrete and Constructional Engineering*, V. 41, No. 1, (Jan. 1946) pp. 10-20
Reviewed by Glenn Murphy

In this article of the series Prof. Magnel describes some of the prestressed concrete structures which have been built in Belgium under the auspices of the Laboratory of Reinforced Concrete at the University of Ghent.

A six-track railway bridge of deck construction was built with a different system for each track. Each deck had a span of about 66 ft. and a width of 12 ft. Two of the decks which were reinforced with prestressed wire were 3 ft. 7½ in. thick and each required 119 cu. yd. of concrete as compared with 190 cu. yd. of concrete for a conventional reinforced concrete span with a thickness of 6 ft. 1 in. The weights of steel were 5.7 and 26 tons, respectively.

Other structures built using prestressed concrete include a road bridge, footbridges, pipeline bridge, silos, and an arch roof for a factory. Plans are also being developed for replacing the four supporting piers for the main tower of the St. Nicholas Church in Ghent.

Railway roadbeds stabilized with portland cement grout

ARTHUR J. BOASE, *Civil Engineering* Vol. 15, No. 10 (Oct., 1945) pp. 447-449 Reviewed by J. R. SHANK

Passing trains develop pumping action under track when the ballast and sub-grade are wet and where impervious layers exist on the sub-grade and are developed in the lower strata of the sub-ballast by reason of imperviousness in the original ballast or by penetration of clay from earlier pumping action. Cost of maintenance has been very high in regions where these conditions exist.

Pressure placing of portland cement grout has served to fill these pump chambers with stable material and to seal off inlets and outlets of the same, resulting in greatly reduced costs of maintenance and operation of trains.

Pneumatic pressures and occasionally hydraulic pressures, rarely over 100 psi, serve to inject grouts carrying up to 200 lbs. of sand to the bag of cement through pipe injection points driven to the pocket previously called a pump chamber. Thin neat cement grout is used at the start which is followed by more and more sand until the pocket is filled. The mix and condition of the grout is largely determined by facility for placing as the cost of the cement is less than 20 per cent of the cost of the work.

This work started in 1941 and is now being used by 28 major railroads. On the New York Central lines west of Pittsburgh in 1942, 31 spots were treated. Before treatment these spots were costing $1,187.10 per month in maintenance. Since treatment the cost has been $73.95. The total cost of treatment was $4,334.11.

Aqueduct for Salt Lake City

L. R. DUNKLEY, *Civil Engineering*, Vol. 16, No. 4, pp. 160-162, (April, 1946) Reviewed by J. R. SHANK

Twenty-two and one-half miles of reinforced concrete pipe is part of the aqueduct which connects Deer Creek Reservoir in Provo Canyon, Utah to Salt Lake City, a total distance of 41 mi.

Bell and spigot sections 69 in. diameter by 20 ft. long having 7½ in. walls were constructed and laid. The steel was welded prefabricated. Three sizes of aggregate, sand, $\frac{3}{16}$ to ¾, and ¾ to 1½ in. were used. The basic mix was 1:2.25; 3.35 of equal parts of the two coarse aggregate sizes. The concrete was pumped to the forms and external vibrators were used in placing. On completion of the pour the pipe was covered with wooden covers and steam cured at 110 to 130 F. A strength of 750 psi was attained before the forms were removed. This took from 8 to 12 hrs. Steam curing was continued until the strength was 3500 psi. The 28-day strength was 4500 to 5000 psi. Pipe units for bends in alignment were cast with the required bevels in the bells.

A circular rubber gasket was used at the joints. Each successive length was pulled into place by a portable hand winch, installed in the previously laid pipe. Green soap with a vegetable oil base was used as a lubricant. Outside grouting and inside calking completed the joint.

Tests for leakage indicated losses over one 10-mi. length of 235 gal. per mi. in 24 hrs. One 1.7 mi. section of this showed 500 gal. The maximum permitted was 3500 gal. per mi. per 24 hrs.

Measuring the consistency of concrete mixtures

From "*Investigations as to the Consistency of Concrete Mixtures*", N. V. MATERN and N. ODEMAKK. Bull. 68, *Statens Vaginstitut*, Stockholm, Sweden, 1944 HIGHWAY RESEARCH ABSTRACTS

The Road Institute of Sweden has carried out a series of investigations with different concrete mixes, trying several methods of measuring the consistency, namely: (1) the flow table, (2) the slump test, (3) the Swedish Vebe-apparatus and (4) the same apparatus equipped with a sinker.

The Vebe-apparatus, Fig. 1, consists of a plane table (A) vibrating 3000 vibrations per minute. To the table is attached a cylindrical vessel (B) 24 cm. in. diameter and 20 cm. high. The pivoting rod (D) carries a circular glass plate (E) which can be turned over the vessel, and made free to move vertically. A form, shaped like a cone (the same as that which is used in the slump test) is placed in the center of vessel (B) and filled with concrete. After removal of the form, the glass plate is turned over the concrete and carefully lowered to contact. The vibrating device is then started. The time is measured from this moment until the concrete is deformed to full contact with the whole underside of the glass plate. The consistency is defined by this time in seconds and is designated °VB.

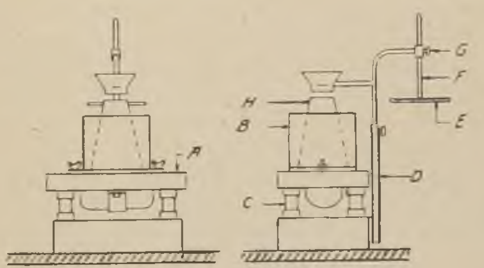

Fig. 1—Vebe-apparatus for measuring the consistency of concrete mixes

The Road Institute has also tried a modification of the measuring device in the Vebe-apparatus. The glass plate was replaced by a sinker (a steel plate fastened to a rod). The consistency was defined as time in seconds for 5 cm. sinking. The consistency thus obtained is called °VB_m.

The flow table gives exact and, with repeated tests, equal values. The method however, can only be used for rather loose consistencies, less than 4 to 5 °VB. The slump test covers about the same range of consistency as the flow table. This test is not very accurate for lean mixes and mixes with a deficiency of the finest particles. The values of repeated tests often have great dispersion. The Vebe apparatus generally gives good values between 2—3 °VB to 25—30 °VB and the dispersion is low. Thus the apparatus is especially useful for rather stiff mixtures which are well compressed at the placing, for instance vibrated concrete for pavements. The method based on using a sinker in the Vebe-apparatus is suitable for mixtures with maximum size of coarse aggregates less than 0.79 to 1.18 in.

Concreting at very low temperatures

The Surveyor and Municipal and County Engineer, Vol. CIV, No. 2812, Dec. 14, 1945, (London).

HIGHWAY RESEARCH ABSTRACTS

The fact that, under extreme conditions of cold, electric current is employed for keeping concrete warm while construction is proceeding is bound to interest engineers, and they will find an account of such and other practices in the U.S.S.R. in a paper entitled "Electro-Concrete," by Mr. K. Billig, published in abridged form in the October, 1945, number of the Journal of the Institution of Civil Engineers (England).

Mr. Billig describes methods normally followed in that region, for keeping concrete from freezing. Temporary hutting is erected, the air inside being heated in various ways. Sand and gravel, frozen hard in transit, are thawed under canvas in large heaps on floors on which a number of lines of perforated steam pipes are laid. The mixing water is heated to about 100-120 F. Concrete mixed at remote plants is conveyed in well insulated containers so that the material cast in the moulds is never at a temperature less than 41 F.

The principle on which heating by electricity is based is that wet green concrete is, to a certain extent, electro-conductive, the resistance being sufficient to raise the temperature considerably, which may be useful in accelerating setting and hardening of the concrete or in preventing it from freezing at too early a stage. Current is passed through the green concrete by introducing two or more poles. Only alternating current can be used, because direct current decomposes the water by electrolysis. It has been found that the leaner the mix and the smaller the proportion of sand to large aggregate, the higher is the electro-resistivity of the concrete; that, for equal mix and consistency, gravel concrete has a higher resistivity than broken-stone concrete and that for the same nominal mix and for similar aggregates a moist concrete has a higher resistivity than a wet concrete. The cost of treatment for a work of about 1,300 cu. yd. of concrete, at a rate of 26 cu. yd. per day, averages 25 to 30 per cent of the contract price. In the case of the construction of a large factory, in the course of which the author supervised the electric warming of 3,270 cu. yd. of concrete, the costs, per cu. yd. in English equivalents, were: current 6s, 5d., ($1.28), other items, 9s, 7d., ($1.92), including labour 3s. 11d. ($0.79) and organization 2s. 5d. ($0.48). At temperatures above 15 F costs would probably be considerably less.

Soil cement in aerodrome construction

W. C. Andrews, *Journal, Engineers Australia* Vol. 17, No. 9 (Sept., 1945) pp. 165-172

Reviewed by J. R. Shank

The construction of two aerodome projects in western New South Wales involved 350,000 sq. yd. of soil cement. The total thickness was 6 in. resting on a compacted sub-base 6 in. thick.

A soil map was prepared in accordance with A.S.T.M. and U. S. Public Road Administration test procedures. The cement content was 10 per cent by volume with a surface enrichment of 4 per cent additional cement. The soils were of a brown, desert type containing lime marl. They ranged from lighty sandy loams of A-2 P.R.A. classification, having a maximum density up to about 130 lb. per cu. ft. at optimum moisture contents ranging from 8 to 12 per cent, and with less than 15 per cent passing the 200 screen, down to light clay loams of A4-A6 type, with maximum densities of about 110 lb. per cu. ft.

Eighty per cent of the soil was specified to pass ¼-in. screen. The moisture content, below which the cement did not deleteriously prehydrate in the dry mix, appeared to be about 9 per cent. It was found that pulverization procedures were most efficiently carried out if this moisture content was brought up to 9 per cent.

On one of the projects all mixing was carried out by windrowing and on the other by rotary mixing. Cement in amounts up to 300 tons per day were spotted in bags over the area a half day in advance. The bags were emptied and spread over the surface to uniform depth with rakes. Dry mixing was most efficiently carried out by rotary hoes making two passes traveling at from 1 to 1½ mph mixing a depth of 4 to 5 ft. Water was added by power and gravity sprayers. Spring-tooth cultivators at maximum tractor speeds or disc harrows followed to disperse the water through the mixtures. Continual traversing of the mix by six-furrow plows assisted in turning over the lower dry material. This wet mixing procedure had to take place within one hour. Rolling operations, every third pass of which was followed by disc harrows to break up any clods formed by the rolling, to within 1½ in. of finished grade took place during the next 1¼ hr. The additional cement used for the top enrichment was spread from trucks during the final stages of sheepsfoot rolling, while water to hydrate this additional cement was added. Finishing then proceeded by grading and rolling with pneumatic rollers, followed first by nail drags to insure against formation of compaction planes near the surface, and then as compaction approximated to the surface, by final grading, pneumatic rolling, and broom dragging.

Curing because of the drying winds prevalent was very difficult. Sisalkraft blankets about 26 ft. wide by 50 ft. long were applied generally within one hour of finishing. Considerable amounts of soil were placed on the edges and joints as well as here and there on the blankets to hold them down against the wind. At the end of seven days condensation on the under sides of the blankets usually appeared.

The results were a uniform gray close-textured material devoid of streaks and soil nodules without apparent cracking during the curing operations. Cracks appeared after exposure at intervals up to 20 ft. Subsequently, cracks developed at closer intervals but were confined to the surface layers. The surface enrichment tended to form separate laminations of material.

The average cost per sq. yd. was 5 s, 4 d.

The strengths attained were 440 and 660 psi for windrowing and rotary mixing procedures respectively.

Rapid determination of the bitumen content in pavements

HARRY ARNFELT. *Statens Vaginstitut*, Stockholm, Sweden, 1942

HIGHWAY RESEARCH ABSTRACTS

A method for the rapid determination of the tar content of tar mineral masses, such as tar-concrete, has been devised. Rapidity in the process of extraction has been effected by means of a mechanical device, consisting essentially of a cylindrical container, which is rotated about its axis. The direction of rotation is reversed at regular intervals, the extraction liquid thus being made to flow through the bituminous mass. Benzene was used as extracting liquid. The chief advantage of this method of extraction is that the mineral particles are gently rubbed against each other. When tar is dissolved from the surface of the aggregates, it leaves a layer of insoluble organic matter, which prevents the tar underneath this layer from being dissolved. The rubbing of the particles against each other removes the insoluble organic matter. The tar is thus rapidly dissolved, the insoluble organic matter and fine mineral particles being dispersed in the solution. This solution is filtered through a paper filter in a metal funnel. After drying, the material on the filter is removed from the paper and ignited in air, under specified conditions ensuring the non-decomposition of the mineral matter.

The procedure described may be expected to give the percentage of tar in the specimens, within a comparatively short time, i.e., less than 3 hr.

Some sources of error in the determination of the tar content are discussed. In order to test the correctness of the results obtained by this method, some determinations of the tar content of known specimens have been made. The result proved to be satisfactory.

It was considered of some interest to know if this procedure also could be applied to pavements containing asphaltic bitumen. Experiments showed, that extraction was easily performed in the aforesaid extraction apparatus, but the filtration of the solutions of asphaltic bitumen presented some difficulties.

An investigation on the filtration properties of such solutions was therefore carried out.

Even in the case of particle-free solutions of asphaltic bitumen, clogging of the filter often occurs when a hard filter is used. This is due to small amounts of a substance associated with the asphaltic bitumen, the nature of which is still obscure, but which very likely consists of almost insoluble high-polymeric hydrocarbons, related to, but not identical with the carbenes. Apparently this substance gradually renders the filler cake forming on the filter impenetrable to the remaining parts of the asphaltic solutions.

Different solvents were examined regarding their capacity for dissolving asphaltic bitumen and yielding solutions easy to filter. Among the substances tried, a B-dichloro-ethylene was found to be best suited to the purpose.

Filtration became easier when care was taken to suppress the formation of a filler cake, and by use of filters, in which the filler particles were gradually caught and distributed over a comparatively large filter volume. Such filters, consisting of combinations of paper and cloth filters, commencing with a weak filtering cloth and ending with a hard filter paper, proved to be useful for the filtration of solutions of asphaltic bitumen containing fine mineral particles.

It was thus possible to filter solutions of asphaltic bitumen if an adequate filtering medium was employed.

In order to attain greater rapidity, however, it was found better to dispense with the finest filter paper and instead to use a centrifuge to recover the finest mineral particles.

Finally, a chapter on the decrease in weight by heating mineral matter is added. The accuracy of the determination of tar by the above method is largely dependent on the constancy of the weight of the mineral matter by the ignition for the removal of the organic matter.

The mineral matter consists often to a large extent of calcium carbonate, which is likely to decompose during the ignition. The amount of carbon dioxide given off by the decomposition is dependent on the rate of decomposition at the temperature used. It was found by experiments that less than one per cent by weight of carbon dioxide is given off at temperatures below 550 C. during one hour. In order to obtain additional data a study of the literature on this subject has been made.

A few experiments have also been made to ascertain the amount of water given off from powdered rocks, not containing carbonates, during the ignition. These experiments showed that it generally can be expected that the loss in weight caused by the ignition is less than one per cent.

Damage on concrete pavements by wintertime salt treatment

HARRY ARNFELT. *Statens Vaginstitut*, Stockholm, Sweden, 1943 HIGHWAY RESEARCH ABSTRACTS

In the spring following the exceptionally cold winter of 1940-1941 some damage was observed on the surface of a concrete road near the town of Linkoping in central Sweden. On some stretches a great part of the surface had scaled off in comparatively large flakes.

A request was made to the State road institute to investigate the cause of the damage.

It was found that the damages could possibly have been caused by the winter-time treatment of the road with sand mixed with calcium chloride to remove ice.

In order to test this possibility specimens from the road were compared with specimens taken from another road, in the vicinity of the town of Orebro in central Sweden, which had not been treated with calcium chloride and which was free from damage.

These specimens were then subjected to the influence of aqueous calcium chloride solutions under conditions of alternate freezing and thawing. Preceding the description of the freezing and thawing tests is a chapter discussing the physiochemical equilibrium between a salt and water on the basis of the equilibrium diagrams of sodium chloride-water and calcium chloride-water.

Prior to the tests which the specimens mentioned, some freezing and thawing tests were made on small pieces of concrete chiseled from a concrete pavement. The purpose of these tests was to find the strength of the calcium chloride solution which acted most quickly on concrete.

When submitted to the freezing and thawing tests the specimens were completely immersed in solutions with different percentages of calcium chloride. The vessels containing the solutions were placed in a refrigerator for 12 hours at a temperature of –25 C. (–13 F.). After that time they were taken out and allowed to stand in the laboratory at room temperature for 12 hours. The freezing and the subsequent thawing were repeated in the same manner until the specimens had disintegrated to a required degree.

It was found that a solution of calcium chloride having a concentration of approximately 3.5 per cent had the strongest detrimental effect on the concrete specimens. Weaker as well as stronger salt solutions were far less detrimental.

So far as the author knows it has earlier been assumed that the strongest solution had the strongest action. In fact, those solutions which were too concentrated to freeze even at the lowest temperature used had no observable action at all.

Some freezing and thawing tests were made on specimens from the same source by the use of aqueous solutions of other substances.

Rock salt was found to act most strongly in 5 per cent solution. Sodium chloride, barium chloride, potassium ferrocyanide, potassium ferricyanide, ethyl alcohol, and urea in solutions having the same depression of the freezing point as a 3.5 per cent calcium chloride solution were found to disintegrate concrete strongly. (Except in case of potassium ferrocyanide, owing to the formation of a protecting layer of potassium calcium ferrocyanide).

Strong solutions of ethyl alcohol and urea had a comparatively minor power of disintegration.

For comparison some freezing and thawing tests were carried out on specimens of brick and of sand-stone. These specimens were, in contrast to those of concrete, disintegrated by pure water to the greatest degree. Dilute calcium chloride also affected the specimens but to a lesser degree. Solutions stronger than 5 per cent showed very little action either on brick or on sand-stone.

The most aggressive aqueous solution of calcium chloride, 3.5 per cent, was used for the freezing and thawing tests on the concrete specimens from the Linkoping and Orebro roads. After seven cycles of freezing and thawing, heavy damages were observed. It was thus shown that the damage on the road was probably caused by the detrimental action of dilute calcium chloride solutions, which were formed by calcium chloride and melting ice.

Furthermore, it was shown that a mechanical action on the surface of the concrete, such as blows from a hammer, which in itself had no visible effect, accelerated the disintegration.

In addition, some conclusions of more general interest can be drawn from the freezing and thawing experiments.

Dilute, hypoeutectic, aqueous solutions of different substances have, when freezing, an ability to disintegrate concrete. It has been shown that the power of destruction of solutions for some substances varies with the concentration and reaches a maximum at a certain, not very high, concentration.

Most probably this conclusion can be generalized to include all substances which are sufficiently soluble in water. The fact that the most active solutions always seem to be hypoeutectic, i.e., those solutions, in which at lower temperatures ice is primarily crystallizing, strongly suggests that the disintegrating agent is the ice crystallizing from the solution when the temperature is decreasing in the fine capillary system of the concrete.

ACI NEWS LETTER

Vol. 17 No. 6 JOURNAL of the AMERICAN CONCRETE INSTITUTE June 1946

SYMPOSIUM ON ENTRAINED AIR IN CONCRETE

The session on Air Entrainment at the 42nd Annual Convention in Buffalo contained a group of short contributions, each, in most cases, covering only the high-lights of a more comprehensive paper on the subject.

In this JOURNAL (pp.601 to 700) these fourteen contributions have been "filled out" and, with a short added introduction, are presented as a symposium on this timely subject.

Since the last symposium on Air Entrainment in the June 1944 JOURNAL much has been learned about air entrained in concrete and these papers add much to the published knowledge. Many questions have been answered. Many more are raised. Meters, dispensing devices for introducing admixtures, methods and apparatus for determining air content have been developed and are described. The problems of the designers, producers and users are discussed and the solutions to some of them are described.

The symposium, with discussion, will soon be combined with related papers by C. E. Wuerpel, "Field Use of Cement Containing Vinsol Resin" (Sept. 1945) and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures" (Feb. 1946) into a second publication by the ACI on "Air Entrainment in Concrete."

MEMBERSHIP APPLICATIONS INCREASE

Records of long standing were broken in May when the Secretary's office received 112 applications for membership during the month. This number has not been equaled since the late 1920's.

The names of applicants approved by the Board of Direction will be published in the September JOURNAL.

WHO'S WHO in this ACI JOURNAL

Bryant W. Pocock

whose paper, "Asphaltic Oil-Latex Joint-Sealing Compound," appears on p. 565, attended Wayne University, Detroit; John Hopkins University, Baltimore, Md. (A. B. degree 1927); Ludwig-Maximilians Universität, Munich, Germany; Privatlaboratorium Benders und Hobeins, Munich, Germany; and the University of Michigan, Ann Arbor (M. A. degree 1934).

With the exception of two years during the late war, when he was chief chemist for Nash-Kelvinator's Propeller Division, Mr. Pocock has been employed by the Michigan State Highway Department since 1937, his present position being that of chemical research engineer. For the past four years he has been consulting editor on the staff of Products Finishing magazine and is the author of numerous technical papers published in this and other journals.

Mr. Pocock is a member of the American Chemical Society, the Electrochemical Society, the American Society for Metals, the American Electroplaters' Society, and the Photographers' Society of Michigan.

Roger Rhoades

a member of the ACI since 1942, and co-author of the paper "Petrography of Concrete Aggregate" (p. 581), attended the University of California at Berkeley for both undergraduate and postgraduate work in geology. Following work in Mexico and Venezuela he was for a time Assistant Professor of Geology at Antioch College, Ohio, then in 1934 joined the Tennessee Valley Authority as an engineering geologist. In 1941 he went to the Bureau of Reclamation to organize their Petrographic Laboratory and to participate in research relative to the deleterious reaction of some aggregates with high-alkali cements, where he currently supervises the Bureau's Geology Section including the Petrographic and Earth Materials Laboratories and, in addition to other functions, engages in the search for construction materials, including concrete aggregate.

Richard C. Mielenz

co-author (with Roger Rhoades) of the paper on "Petrography of Concrete Aggregate" (p. 581) attended the University of California (Berkeley) receiving his Ph.D. in 1940. After graduation he was employed as geologist by the Standard Oil Co. of California in the San Joaquin Valley from 1939 to 1941. He then joined the Petrographic Laboratory of the Bureau of Reclamation where he has been engaged in application of petrography and geology to engineering problems, particularly selection of concrete aggregates and riprap materials for use on Bureau projects. In addition, he is conducting research into basic problems concerning construction materials, especially the problem of cement-aggregate reaction.

In April 1946 he published a paper entitled "Petrographic Examination of Concrete Aggregates" in the Bulletin of the Geological Society of America. An article written in collaboration with C. J. Okeson entitled "Foundation Displacements Along the Malheur River Siphon as Effected by Swelling Shales" appeared in Economic Geology for May 1946. Mr. Mielenz joined the Institute in 1945.

W. A. Cordon

whose paper entitled "Entrained Air—A Factor in the Design of Concrete Mixes" appears on p. 605 received a B.S.C.E. degree from Utah State College in 1935, and since graduation he has been continuously in the employ of the U. S. Bureau of Reclamation. For 3 years he was Concrete and Earth Control Engineer on the Upper Snake River Project, Idaho, during construction of Island Park Dam and Cross Cut Canal and Diversion Dam.

He also spent 2½ years as Construction Inspector on the Grand Coulee Dam, Washington. Since 1941 he has been stationed at Denver, Colorado, and at the present time is Head of the Concrete, Aggregate and Mix Investigations Unit of the Materials Laboratories.

L. E. Andrews

regional highway engineer, Portland Cement Association, New York City, is author of "Recent Experiences with Air-Entraining Portland Cement Concrete in the Northeastern States," p. 621. For many years he has taken an active part in the design of concrete mixtures and concrete performance studies and has followed closely all phases in the development of air-entrained concrete since its introduction about 1938.

Prior to going with PCA, Mr. Andrews was district engineer, New Jersey State Highway Dept.; assistant district engineer, Pennsylvania Dept. of Highways and resident engineer, Illinois Division of Highways.

During World War I he served with the Corps of Engineers as Private, Corporal, Sergeant, Second Lieutenant and Captain. He was engaged on military road construction and maintenance work in this country and in France. Mr. Andrews is a graduate in civil engineering from Pennsylvania State College.

Alexander Foster, Jr.

a member of the Institute since 1912 and Director from the Third District is Vice-President of the Warner Company, Philadelphia, Pa. His paper, "Experiences with Air-Entraining Cement in Central-Mixed Concrete" (p. 625) is his second contribution to the JOURNAL.

Stanton Walker and Delmar L. Bloom

are co-authors of "Studies of Concrete Containing Entrained Air" (p. 629).

Mr. Walker, Vice-President of the Institute and a member since 1921 is a frequent contributior and needs no introduction to JOURNAL readers. He is also co-author with W. H. Klein on "A Method for Direct Measurement of Entrained Air in Concrete" (p. 657).

Mr. Bloom is in immediate charge of the laboratory of the National Ready Mixed Concrete Association and of the National Sand and Gravel Association Research Foundation at the University of Maryland where he has been employed since January 1, 1945. While attending Iowa State College, where he graduated in Civil Engineering in 1943, he was employed by the Iowa Engineering Experiment Station. After his graduation he worked for a year for the Chicago Bridge and Iron Company before entering the concrete research field.

Henry L. Kennedy

author of "Homogeneity of Air-Entraining Concrete" (p. 641) is not new to these pages, having been the author of several contributions in recent years. Mr. Kennedy has been a member since 1934 and is a Director-at-Large.

Edward W. Scripture, Jr.

author of "Methods of Entraining Air in Concrete" (p. 645), a member of the Institute since 1931, is Director of Research for the Master Builders Co. Author of "Metallic Aggregate in Concrete Floors," *Proceedings* Vol. 33, and of several discussions, Dr. Scripture needs no further introduction to JOURNAL readers.

A. T. Goldbeck

a member of the Institute since 1926, member of the Publications Committee and author of many papers is well known to the ACI. His paper "Effect of Air Entrainment on Stone Sand Concrete," appears on p. 649.

William H. Klein

co-author with Stanton Walker of a paper entitled "A Method for Direct Measurement of Entrained Air in Concrete" (p. 657), a member of the Institute since 1924 and of the Publications Committee, is Vice-President and General Operating Manager of the Pennsylvania-Dixie Cement Corp. Mr. Klein graduated from the University of Michigan with the degree Bachelor of Science in Chemical Engineering in 1906.

From 1906-11 he was with the United Kansas Portland Cement Co. In 1926 he was with the Dixie Portland Cement Co., when he became General Manager of the Pennsylvania-Dixie Cement Corp. He has held his present position since 1933. Mr. Klein is also a member of American Society of Testing Materials and the American Society for the Advancement of Science.

R. R. Kaufman

member of the Institute since 1937, has spent the past 30 years in the concrete field in both routine and research laboratory work, as well as field experience throughout United States and Canada.

He was with the Sandusky Cement Company from 1916 to 1923, as Chief Chemist of the Cleveland Laboratory.

In 1923 he became affiliated with the Master Builders Company as Chief Chemist, and in 1927 became Chief Engineer, the position he holds at the present time.

During these years he has been engaged in the development of techniques, products and equipment related to concrete work. His paper, "Automatic Dispensing Equipment for Air-Entraining Agents" appears on p. 669.

E. M. Brickett

author of a paper entitled, "Mechanical Dispensing Devices for Air-Entraining Agents" (p. 673), although not a member of the Institute, is not new to ACI readers, having been co-author with J. C. Pearson of a paper "Studies of High Pressure Steam Curing." Mr. Brickett is with the Cement Division of the Dewey and Almy Chemical Co.

S. W. Benham

whose paper "A Simple Accurate Method for Determining Entrained Air in Fresh Concrete" (p. 677) is his second contribution to ACI literature, is Research Engineer for the State Highway Commission of Indiana, Bureau of Materials and Tests.

W. F. Kellermann

an ACI member since 1938 and a frequent contributor to professional journals, needs no introduction on the occasion of his paper "Effect of Use of Blended Cements and Vinsol Resin-Treated Cements on Durability of Concrete" (p. 681).

W. H. Herman

author of a paper on "Air-Entraining Concrete" (p. 689) has been employed by the Pennsylvania Department of Highways for the past 21 years. In 1944 he was appointed to his present position as Chief Research Engineer, supervising the activities of the State Testing Laboratory and the Planning, Traffic, Architectural and Forestry Units.

Bertrand H. Wait

member of the Institute since 1939, graduated from Cornell University, C. E. degree in 1902. He spent several years on New York Subways; five years with Board of Water Supply on Catskill Aqueduct work and as Division Engineer on city tunnels when he resigned in 1913. From 1913-1918 as Division Engineer in charge of eleven counties in New York State he was in charge of Design and Construction of a large percentage of the early concrete roads built by the New York State Highway Dept. From 1918-1929 he was Eastern Manager for the Portland Cement Association in charge of the eleven northeastern states. Since that time he has been actively interested in the development and promotion of construction materials principally in connection with concrete and asphalt pavements in the northeastern states. He is President of The Wait Associates, Inc., and consulting engineer for various construction material manufacturers. He is a member of the A.S.T.M. and A.S.C.E. His paper, "Portland-Rosendale Cement Blends Give High Frost Resistance," appears on p. 697.

Roderick B. Young

member of the Institute since 1917; author of many papers; member of the Board of Direction as Director, Vice-President, President and Past-President for 16 years; is one of our best known authors. His paper "The Repair of Concrete: An Introduction" appears on p. 701.

E. H. Praeger

an ACI member since 1927 contributes his first paper to the JOURNAL in many years, entitled "Behavior of Concrete Structures Under Atomic Bombing" (p. 709). Capt. Praeger's contribution is based upon observations made as a member of a group of qualified officers and technical experts sent to Japan by the Secretary of the Navy after the war to survey damage wrought by the atomic bombs and other weapons on targets in Japan.

Navy Seeks Former Seabees

Former Seabee Warrant Officers, as well as other qualified men and officers with a background in construction work, are being sought to fill approximately 50 vacancies in the regular U. S. Navy Warrant Officer grades.

Needed particularly for work at overseas naval bases and other projects administered by the Bureau of Yards and Docks, the Warrant Officers, including Chief Warrant Officers, would be assigned to varied duties involving all types of naval shore maintenance and construction.

Former Warrant and Commissioned officers are eligible if they had not passed their 37th birthday on the date their first appointment above Chief Petty Officer was made by the President. Other eligibility requirements are set forth in Bureau of Naval Personnel Circular Letter 288-45 (revised).

Honor Roll

February 1 to May 31, 1946

Anton Rydland heads the list with 6½ new members since February 1, 1946.

Anton Rydland 6½
Charles E. Wuerpel 5½
Jacob Fruchtbaum 5
C. C. Oleson 4
Walter H. Price 4
Birger Arneberg 3
D. R. Cervin 3
C. A. Hughes 3
James J. Pollard 3
F. E. Richart 3
K. E. Whitman 3
H. F. Gonnerman 2½
Karl W. Lemcke 2½
Wm. R. Waugh 2½
A. Amirikian 2
A. J. Boase 2
Aloysius E. Cooke 2
E. F. Harder 2
Dean Peabody 2
Henry Pfisterer 2
Flory J. Tamanini 2
I. L. Tyler 2
H. M. Hadley 1½
John J. Murray 1½
H. C. Shields 1½
Lewis H. Tuthill 1½
Stanton Walker 1½
Julius Adler 1
J. F. Barton 1
F. J. Beardmore 1
Rene L. Bertin 1
R. F. Blanks 1
M. E. Capouch 1
Frank W. Chappell 1
Anthony D. Ciresi 1
Emil W. Colli 1
F. K. Deinboll 1
C. T. Douglass 1
H. B. Emerson 1
Ray C. Giddings 1
Elliot A. Haller 1
Issac Hausman 1
M. J. Hawkins 1
Denis O. Hebold 1
Elmo Higginson 1

W. M. Honour....................1
O. W. Irwin....................1
W. W. Johnson....................1
Wm. R. Kahl....................1
R. R. Kaufman....................1
Henry L. Kennedy....................1
H. Walter Leavitt....................1
James E. McClelland....................1
F. R. McMillan....................1
M. J. McMillan....................1
Edward P. McMullin....................1
Robert L. Mauchel....................1
D. G. Marler....................1
Hugh Montgomery....................1
Ben Moreell....................1
Philip Paolella....................1
D. E. Parsons....................1
J. C. Pearson....................1
A. F. Penny....................1
C. J. Posey....................1
C. C. Pugh....................1
E. M. Rawls....................1
John A. Ruhling....................1
Raymond C. Reese....................1
Erik Rettig....................1
George P. Rice....................1
J. L. Savage....................1
G. R. Schneider....................1
John C. Seelig....................1
Thomas C. Shedd....................1
LeRoy A. Staples....................1
J. W. Tinkler....................1
Bailey Tremper....................1
Harold C. Trester....................1
Oscar J. Vago....................1
David Watstein....................1
George C. Wilsnack....................1
Ralph E. Winslow....................1

The following credits are, in each instance, "50-50" with another Member:

A. Arnstein
Kasim Atlas
Michel Bakhoum
Paul L. Battey
E. Ben-Zvi
R. H. Bogue
J. M. Breen
Ernest L. Brodbeck
Fred Burggraf
Fred Caiola
T. F. Collier
Sam Comess
R. W. Crum
Clifford Dunnells
G. J. Durant
A. C. Eichenlaub
Axel Erikeson
Cevdet Erzen
E. E. Evans
H. F. Faulkner
K. P. Ferrell
G. V. Gezelius
H. J. Gilkey
E. A. Glennen
D. E. Gondolfi
Howard A. Gray
Per O. Hallstrom
Shortridge Hardesty
H. L. Henson
H. D. Humphries
Frank H. Jackson
W. D. Kimmel
Wm. Lerch
Douglas McHenry
Eugene Mirabelli
C. C. More
A. B. L. Moser
Fernando Munilla
Wm. T. Neelands
Wm. D. Painter
George P. Palo
R. S. Phillips
Herman G. Protze, Jr.
R. D. Rader
Frank A. Randall
N. L. Shamroy
T. E. Shelburne
C. E. Shevling
Charles M. Spofford
D. J. Steele
John Tucker, Jr.
Jose Vila
Ray V. Warren
W. W. Warzyn
Eugene P. H. Willett
R. B. Young

Clayton N. Ward

member of the Institute since 1945, and John A. Strand announce that they have formed a new firm of consulting engineers under the name of Ward and Strand. The firm will engage in engineering projects involving water, diesel and steam power; water works; sewerage works; flood control; irrigation and drainage; hydraulic research; and hydrological investigation.

S. C. Hollister

Dean of the College of Engineering at Cornell, an Institute member since 1917, former Director and President (1932-33) has been named to the new vice-presidency recently created by the Board of Trustees at Cornell University. Dean Hollister will continue as head of the College of Engineering while assuming his new administrative duties. As vice-president, he will be in charge of University development.

F. E. Richart

member of the Institute since 1917, former Director and Past President (1939), has been elected a member of the Board of Directors of the American Society for Testing Materials for a term of three years.

George R. Wernisch

recently discharged from the Navy as a lieutenant commander has just been appointed manager of Ceco Steel Products Corporation's steel joist and roof deck division. He will make his headquarters at Ceco's general offices and plant No. 1 in Chicago.

New Members

The Board of Direction approved 52 applications for Membership (37 Indidvidual, 3 Corporation, 8 Junior, 4 Student) received in March and April as follows:

Antrim, Hugh B., Box 35, Juneau, Alaska.

Altobellis, Julian A., 872 West Peachtree St., Atlanta, Ga.

Banerjee, H. S., 5 Maharsi Devendra Rd., Burra Bazar, Calcutta, India.

Barrada, Baha'Uddin, 201 Engineering Hall, Urbana, Ill.

Bayer, Joseph, 4059 Kendall, Detroit 4, Mich.

Bechtold, Ivan, c/o U. S. E. D. Testing Laboratory, Government Moorings, 8010 N. W. St. Helens Rd., Portland, Ore.

Bjuggren, Ulf, A. B. Betongindustri, Mejerivagen 4, Stockholm 9, Sweden

Columbia Concrete Products Co., 2401 Consaul St., Toledo, Ohio.

Cornell, Holly A., Cornell, Howland, Hayes & Merryfield, Room 5, Smith Bldg., Corvallis, Ore.

Cortland, Abram, 750 Grand Concourse, Bronx 51, N. Y.

Crenshaw, Allen E., 1724 Hyde St., San Francisco, Calif.

Dienst, Frank L., 119 S. Summit St., Bowling Green, Ohio.

Dumaresq, J. Phillip, 73 Pemberton St., Cambridge, Mass.

Earle, M. O., Pittsburgh Testing Laboratory, 421 Canal St., New York 13, N. Y.

Eleta, Fernando, 428 Memorial Drive, Cambridge, Mass.

Elkow, Milton O., 315 Linden Rd., Southgate, Ky.

Ghosn, Raymond S., M. I. T., Cambridge, 39, Mass.

Gonzalez-Rubio, Jr., Elberto, Dept. of Architecture, Georgia School of Technology, Atlanta, Ga.

Hayes, John M., Public Road Administration, 303 Old Post Office Bldg., Little Rock, Ark.

Kelsall, K. J., 47 Waugh St., North Perth, Western Australia.

King, John, 1579 Metropolitan Ave., Bronx 62, N. Y.

Kochanski, Anthony J., Ordnance Works, Weldon Springs, Mo.

Lancashire County Council, County Surveyer & Bridgemaster, County Offices, Preston, Lancashire, England.

Lebelle, Jean, Les Procedes Techniques de Construction, 9 Place des Ternes, Paris 17° France.

Lin, Yuan-ti, U. S. Bureau of Reclamation, Denver 2, Colo.

Lindh, R. T., Twin City Readymix Concrete Co., 2840 Aldrich Ave., S., Minneapolis 8, Minn.

Lowey, Leslie L., 69-61 184th St., Flushing, N. Y.

Lum, Walter, 2021 Cornell Rd., Cleveland 6, Ohio.

Manning, Rear Admiral J. J., Chief, Bureau of Yards & Docks, Navy Dept., Washington 25, D. C.

Mattimore, H. S., 4226 E. Amerton Ave., Colonial Park, Pa.

Mazas, Ing. Enrique, Calle A No. 15, ent. la. y3a, Vedado, Havana, Cuba.

Myott, E. B., 6 Virginia Rd., Reading, Mass.

Naulty, R., c/o Hume Pipe Company (Aust.) Ltd., Upham St., Subiaco, Western Australia.

Nordahl, A., Room 56, N. Y. C. R. R. Bldg., W. Third & St. Clair Ave., Cleveland 4, Ohio.

Pinnell, Richard L., 2375 La Jolla Ave., San Diego 10, Calif.

Powell, W. S., c/o Yonge & Hart, Architects, P. O. Box 928, Pensacola, Fla.

Rahn, George A., Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.

Rivett, J. W., 401 Collins St., Melbourne C. 1, Australia

Schlyter, Ragnar, c/o Statens Hantverksinstitut, Stockholm 4, Sweden.

Schultz, Philo Howard, 118 N. Yankton Ave., Pierre, S. Dakota.

Seeck, Roy H., 441 Manor Pl., N. W., Washington 10, D. C.

Seifried, John F., Ceco Steel Products Corp., 1926 S. 52nd Ave., Chicago 50, Ill.

Solel Boneh Ltd., P. O. Box 563, Haifa, Palestine.

Thadaney, K. V., The Concrete Association of India, P. O. Box 312, Mount Rd., Madras, India

Torregrosa, Miguel, 76 Ponce de Leon Ave., Santurce, Puerto Rico.

Vaughan, J. Philip, 294 North St., Halifax, N. S., Canada.

Whitsett, James J., c/o W. F. Schenck, Trenton, Ohio.

Williams, L. G., 3611 Watson Ave., Toledo 12, Ohio.

Wood, Earle H., 505 Delaware Ave., Buffalo 2, N. Y.

Woodroffe, V. M., 453 Oxford St., Halifax, N. S., Canada.

Woolley, W. R., Federal Works Agency, Public Roads Adm., 2938 E. 92nd St., Chicago 17, Ill.

Yasko, Karel, Box 42, Wausau, Wis.

Joseph L. Duffy

Word has recently been received of the death of Mr. Duffy, April 5, 1946 at the age of 53. He had been a member of the Institute since 1936.

Graduating from Armour Institute in 1915 as a structural engineer, he spent most of his career in Chicago and vicinity engaged in engineering design and construction of highways, bridges and public and private buildings. His most recent work was the construction of a large sanitarium at Kalamazoo, Michigan.

Conde B. McCullough

member of the American Concrete Institute since 1939 died suddenly at his home in Salem, Oregon, on May 6, 1946. Dr. McCullough had been connected with the Oregon State Highway Commission since 1919, first as Bridge Engineer and, since 1937, as Assistant State Highway Engineer. He was fifty nine years old at the time of his death.

He was born in South Dakota and received his engineering training at Iowa State College where he received the degree of Bachelor of Science in 1910 and the professional degree of Civil Engineer in 1916. Prior to his college days, with engineering even then in mind, he served as chainman, rodman and transitman on the Illinois Central Railroad. Following his graduation and after a few months with the Marsh Engineering Company of Des Moines, he joined the Iowa State Highway Commission as designing engineer and later became assistant state highway engineer. Oregon State College brought Dr. McCullough to the Pacific Coast in 1916 as professor of civil engineering, where he remained until 1919 when he became Bridge Engineer for the Oregon State Highway Commission. During his long service in this capacity he supervised the design and construction of many bridges, the most noted being the major structures on the Oregon Coast Highway, well known throughout the nation for their beauty.

In 1936 Dr. McCullough was granted leave by the state and for eighteen months was in charge of design and construction of bridges on the Inter-American Highway with headquarters at San Jose, Costa Rica. On his return to Oregon, he became Assistant State Highway Engineer.

In recognition of his accomplishments and of his service to the state, the Oregon State College conferred on him the degree of Doctor of Engineering in 1928. He also found time to study law and was graduated from Willamette University with the degree of Bachelor of Law in 1928.

Dr. McCullough was the author of numerous papers published by the Institute and several technical books. His latest work, published a few weeks before his death, is a treatise on engineering law entitled "The Engineer at Law," and is an outstanding contribution in this field. Besides membership in the American Concrete Institute, he was a member of the American Society of Civil Engineers, the Northwest Society of Highway Engineers, Tau Beta Pi, Sigma Tau, and Delta Theta Pi, honorary law fraternity.

William Johnson Henderson

a member of the Institute since 1942, died May 10, 1946. He was born at Glen Gardner, N. J., October 31, 1899.

He received the degree of Bachelor of Science in Civil Engineering from Lafayette College at Easton, Pa., in 1922 and the degree of Civil Engineer in 1926. After his graduation, Mr. Henderson spent three years with the Bethlehem Steel Company. In 1925, he became an instructor in structural engineering at Purdue University and later was promoted, successively, to assistant professor and associate professor.

He was a member of Alpha Chi Rho fraternity, Chi Epsilon, and an Associate Member of the American Society of Civil Engineers.

Professor Henderson was married to Ruth Elizabeth Banghart of Glen Gardner on October 4, 1924. He is survived by his widow, his mother and a brother, Abraham B. Henderson.

Portland Cement Association Creates New Division

The Portland Cement Association, through its President, Frank T. Sheets, has announced the election, effective July 1, of Dr. A. Allan Bates as Vice President to direct the newly created Division of Research and Development.

Dr. Bates is widely known for his accomplishments in the fields of chemistry, chemical engineering, ceramics and metallurgy. Since 1938 he has been Manager of the Chemical, Metallurgical and Ceramic Research Division of the Westinghouse Electric Corporation.

He holds a Bachelor of Arts degree from Ohio Wesleyan University, where he majored in chemistry; Bachelor of Science in metallurgical engineering from Case School of Applied Science; Doctor of Science, "Magna cum laude," from the University of Nancy, France; and an honorary degree of Doctor of Engineering from Stevens Institute of Technology. He is a member of Tau Beta Pi and Sigma Xi.

In commenting on Dr. Bates' election as Vice President for Research and Development and on the Association's expanded program, President Sheets said, "It should be understood that scientific research and product development are not new ventures for the Portland Cement Association because these have been important functions of the Association for over 30 years. Now, with augmented funds and facilities, the present and expanded staff of able scientists and engineers will, under Dr. Bates' leadership, go on to new heights of accomplishment, and thus enhance materially the research and technical service which the Association has rendered in the past to the users of portland cement and concrete."

Central Concrete Laboratory Moved

The Central Concrete Laboratory of the North Atlantic Division, Corps of Engineers, U. S. Army at Mount Vernon, New York is to be deactivated during the month of June 1946. The purpose of the deactivation is to permit the transfer of the key personnel and equipment to Clinton, Mississippi where a new laboratory is to be created as a division of the U. S. Waterways Experiment Station, Vicksburg, Miss. The new laboratory will be known as the Concrete Research Division of the U. S. Waterways Experiment Station and will be headed by Mr. Charles E. Wuerpel, who is now engineer in charge of the Central Concrete Laboratory.

Sources of Equipment, Materials, and Services

A reference list of advertisers who participated in the Fifth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1946 issue and (when it is completed) in V. 42, ACI Proceedings. **Watch for the 6th Annual Technical Progress Section in the February 1947 JOURNAL.**

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