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# JOURNAL

*of the*

# AMERICAN CONCRETE INSTITUTE

(ACI PROCEEDINGS Vol. 43)

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**Studies of the Physical Properties of Hardened Portland Cement Paste**—T. C. Powers and T. L. Brownyard

**Resuming, with this volume year, the former JOURNAL publication schedule of 10 issues instead of 6 for the year, the Supplement, issued in recent years with the November issue, will be mailed with the December JOURNAL. It will contain Title Page, Table of Contents, Closing Discussion and Indexes, concluding the volume otherwise completed in the issue of the previous June.**

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# JOURNAL

*of the*

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**Vol. 18 No. 2** 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **October 1946**

# Studies of the Physical Properties of Hardened Portland Cement Paste*

By T. C. POWERS†
Member American Concrete Institute

and T. L. BROWNYARD‡

## IN NINE PARTS

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## SYNOPSIS

This paper deals mainly with data on water fixation in hardened portland cement paste, the properties of evaporable water, the density of the solid substance, and the porosity of the paste as a whole. The studies of the evaporable water include water-vapor-adsorption characteristics and the thermodynamics of adsorption. The discussions include the following topics:

1. Theoretical interpretation of adsorption data
2. The specific surface of hardened portland cement paste
3. Minimum porosity of hardened paste
4. Relative amounts of gel-water and capillary water
5. The thermodynamics of adsorption
6. The energy of binding of water in hardened paste
7. Swelling pressure

*Received by the Institute July 8, 1946—scheduled for publication in seven installments; October 1946 to April, 1947.
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## FOREWORD

This paper deals with the properties of hardened portland cement paste. The purpose of the experimental work on which it is based was to bring to light as much information as is possible by the methods of colloid chemistry and physics. Owing to the war, the original program, which included only a part of the field to be explored, was not completed. Moreover, the interpretation of the data is incomplete, partly because of the inability of the authors to comprehend their meaning, and partly because of the need of data from experiments yet to be made.

Although the work is incomplete, it represents a considerable amount of time and effort. Experimental work began in a small way in 1934 and continued until January 1943. Some additional work was done in 1945 during the preparation of this paper. The first three years was a period of intermittent work in which little of permanent value was accomplished beyond the development of apparatus and procedures. This phase of the work presented many problems, some of which have never been solved to our complete satisfaction.

The interpretation of the results of experiments also presented many difficulties. During the course of our experiments, important new developments in colloid science were coming to light through a series of papers from other laboratories. It was necessary to study these papers as they appeared and to seek their applications to our problems. The result is that the theory on which much of our present interpretation is based is one that did not exist when our work began and is one that is still in the process of development. The reader may note that many of the papers referred to in Part 3 were not published until 1940 or later.

The theory referred to is that of multimolecular adsorption by Brunauer, Emmett, and Teller as first given in 1938 and as amplified in the paper by Brunauer, Deming, Deming, and Teller in 1940. In justification for the use of such a recent and unfinished development, we may note in the first place that a remarkable number of papers by various authors have appeared since 1940 strongly supporting the kind of use that we have made of the theory, particularly the estimation of surface area. In the second place, the basic conclusions reached through the use of the theory might have been reached from a strictly empirical analysis of the data. However, it is difficult to imagine how a picture of the hardened paste as detailed as the one presented in this paper could have been drawn without adopting theoretically justified assumptions.

The paper is composed of nine parts. Part 1 contains a review of previous work done in this field and discusses various experimental methods. Part 2 elaborates on the principal method used in the present study, namely, the measurement of water-fixation. It also presents the empirical aspects of the data so that the reader may become familiar with facts to be dealt with.

Part 3 presents the theories upon which an interpretation of the data in Part 2 can be based. It gives also a partial analysis of most of the experimental data given in Part 2 in the light of the adopted basis of interpretation. Part 4 is a discussion of the thermodynamics of moisture-content changes in hardened paste and the phenomena accompanying those changes. It is thus an extension of the earlier discussion of theory.

In Part 5 data are presented pertaining to the volumes of different phases in the paste. The interpretation of these data involves the use of factors developed in the preceding parts of the paper. The final result is a group of diagrams illustrating five different phases, the relative proportions of each, and how those relative proportions change as hydration progresses.

The relationship between the physical characteristics of the hardened cement paste and compressive strength of mortars is discussed in Part 6. A similar discussion of permeability and absorptivity is given in Part 7.

A study of the freezing of water in hardened paste is presented in Part 8. The conditions under which ice can exist in the paste are described and empirical equations are given for the amounts of water that are freezable under designated conditions.

The properties of portland cement paste as they appear in the light of these studies are described in Part 9. This part amounts to a summary of the outcome of the study, at its present incomplete stage, without details of experimental procedures, or theoretical background.

As mentioned in the first paragraph, a particular point of view as to the meaning of the data has been adopted. Specifically, we have assumed, on the basis of evidence given in the paper, that the various phenomena discussed are predominantly of physical rather than chemical nature. The result of the study therefore constitutes a hypothesis, or series of hypotheses, rather than a rigorous presentation of established facts. Considering the present state of our knowledge, we believe this policy to be more fruitful than one of trying to maintain a strictly unbiased view as to the meaning of the data. As written, the paper represents the thinking of one group of workers (influenced, of course, by many others), and it implicitly invites independent investigations of the same field by any who may have good reason to adopt a different point of view. To this end, we have appended tabulations of the original data.

Though we thus concede the possibility of other interpretations, we nevertheless feel confident that a large part of the present interpretation will withstand logical criticism. However, it seems very likely that corrections and changes of emphasis will develop as experimental work continues, and as further advances are made in fundamental colloid science.

The paper is directed primarily toward all who are engaged in research on portland cement and concrete. However, it may be of considerable interest to many who seek only to understand concrete as they work with it in the field. Studied in connection with earlier papers on the characteristics of paste in the plastic state,* this paper affords a comprehensive, though incomplete, picture of the physical nature of portland cement paste. It, therefore, pertains to any phase of concrete technology that involves the physical properties of the cement paste. This means that the paper should find application to most phases of concrete technology.

For the most part, the reader will find few items of data that bear directly on specific questions or problems. Successful application of this study to research or practical technology requires some degree of comprehension of the work in its entirety. Consequently, it is not likely that a single, casual reading will reveal much that is of value to one not already familiar with the methods and background of this type of investigation.

## ACKNOWLEDGMENTS

We are deeply indebted to Mark L. Dannis and Harold Tarkow, not only for their long and painstaking labor with the various experiments, but also for their contributions to an understanding of the results. Mark Dannis made most of the adsorption and specific-volume measurements and Harold Tarkow performed the freezing experiments reported in Part 8.

We are grateful to Gerald Pickett, whose constructive criticism was of great value throughout most of the period of study.

*Bull. 2, "The Bleeding of Portland Cement Paste, Mortar and Concrete," by T. C. Powers, P.C.A. Research Laboratory (1939); Bull. 3, "Rate of Sedimentation," by Harold H. Steinour, P.C.A. Research Laboratory (1944), reprinted from *Ind. Eng. Chem. 36*, 618; 840; 901 (1944); Bull. 4, "Further Studies of the Bleeding of Portland Cement Paste," by Harold H. Steinour, P.C.A. Research Laboratory (1945).

We are especially indebted to Harold H. Steinour, who took time from his own work to carry on the final volumenometer work described in Part 5. Also, he helped prepare the discussion of thermodynamics in Part 4 and gave much valuable criticism of various other parts of the paper.

To Virginia Atherton, who made many of the hundreds of computations, typed the manuscript, and corrected printer's proof we express our kindest thanks.

## Part 1. A Review of Methods That Have Been Used for Studying the Physical Properties of Hardened Portland Cement

### CONTENTS



Starting as a suspension of cement particles in water, portland cement paste becomes a solid as the result of chemical and physical reactions between the constituents of the cement and water. A solidified paste of typical characteristics is capable of giving up or absorbing a volume of water equal to as much as 50 per cent of the apparent volume of the paste. These facts engender the idea that whatever the chemical constitution of the new material produced by chemical reaction, the new material is laid down in such a way as to enclose water-filled, interconnected spaces. That is, the hydration product appears to be not a continuous, homogeneous solid, but rather it appears to be composed of a large number of primary units bound together to form a porous structure. It seems self-evident that the manner in which the primary units are united, that is, the physical structure of the paste, is closely related to the quality of the paste and is therefore something about which we should be well informed.

Freyssinet(1)* discerned the need for knowledge of paste structure and devised a hypothesis about the setting and hardening process and about the structure of the hardened paste. Giertz-Hedström,(2) an active contributor to this subject, has given an excellent review of publications on this subject. This review, together with Bogue's(3) earlier one of a slightly different aspect of the subject, obviates the necessity of an extensive historical review at this time.

A program of studies of the properties of the hardened paste was begun in this laboratory in 1934. It has consisted mainly of studies of the fixation of water, but has also included measurements of the heat-effects accompanying the regain of water by the previously dried paste, measurements of the freezing of the water in the saturated paste, and various other related matters.

This work has yielded a considerable amount of information on the physical aspects of hardened paste. It contributes to the knowledge of the chemical constitution of the hydration products only in a negative way; that is, it shows that some of the current information on the constitution of the hydration products must be incorrect. Later parts of this paper will give an account of these studies.

## METHODS FOR STUDYING THE PHYSICAL PROPERTIES OF THE HARDENED PASTE†

The question of structure can be broken down into three parts: first, the question as to the chemical constitution of the hydration products, which includes the question of structure of the ultimate parts; second, the question of the structure of the smallest primary aggregations of the ultimate parts; third, the question of how the primary aggregations are assembled and how they are held together. A review of some of the work done by earlier investigators follows.

### Microscopic examinations

*The light-microscope.* The effectiveness of the microscope as a means of studying the structure of the hardened paste is limited because the

*See references end of Part 1.
†See also the review by Giertz-Hedström (Ref. 3).

units of the essential part of the structure are too small to be seen. The results obtained by Brown and Carlson[4] are typical. They, like others, observed that the hardened paste is predominantly "amorphous," so far as the microscope can reveal. Embedded in this amorphous mass are the remnants of unhydrated clinker grains, crystals of calcium hydroxide, and sometimes crystals of other compounds.

Kühl[5] reported that thin sections of hardened cement paste showed ". . . a residue of undecomposed cement particles, separated by a gray and only slightly differentiated mass which gives a feebly diffused luminescence in polarized light. Even under the highest magnification individual particles cannot be distinguished in this material, which is obviously almost entirely ultramicroscopic in structure." However, on examining the same specimens 20 years later he found that ". . . the passage of years had resulted in fundamental changes. No longer (was the material) uniform and only slightly differentiated under polarized light as was the case a few months after their preparation. They now showed a definitely increased (polarized-light) transmission and, most noteworthy of all, their properties were markedly different according to the percentages of water with which they had been gaged. The specimens gaged with the greatest amount of water showed the greatest changes while those mixed with the least water had undergone considerably less modification." The changes mentioned were such as to suggest that the originally colloidal material had gradually changed toward the microcrystalline state, the change being greater the higher the original water content.

Useful information has been obtained by microscopic observations of the hydration of cement in the presence of relatively large quantities of water. But it is unlikely that the structure developed under these conditions is the same as that developed in pastes; hence, conclusions about the normal structure drawn from observations of this kind are open to question.[6] For example, Le Chatelier[7] observed that when a large quantity of water was used, needlelike crystals of microscopic dimensions soon developed. He concluded from this that similar, though submicroscopic, crystals developed under all conditions.

Brownmiller[8] described the results of microscopic examinations of hardened paste by means of reflected light. The method is a modification of that described by Tavasci[9] and Insley[10] for studying the constitution of clinker. In addition to the use of etchants to bring different phases into contrast, Brownmiller treated the surface with a dye which was taken up by the so-called amorphous material and the microcrystalline phases. Although Brownmiller's primary object was to develop the experimental technique, some of his conclusions concerning the nature of hardened paste and the hydration process are of considerable interest. He found that there was ". . . no microscopic evidence of channeling of water into the interior of cement particles to selectively hydrate any single major constituent. Hydration seems to proceed by the gradual reduction in the size of the particles as a function of the surface exposed." This conclusion seemed to be based on the appearance of the coarser particles that remained unhydrated after the first day.

Brownmiller found that a Type III* cement having a specific surface of 2600 appeared to be almost completely hydrated after one day in a sealed vial and six days in water. A Type I cement having a specific surface of 1800 showed an unhydrated residue of about 15 per cent after one day in a sealed vial and 28 days in water. The water-cement ratio of the original paste was 0.4 by weight in both cases.

The only microcrystalline hydrate mentioned by Brownmiller was calcium hydroxide. This was found as clusters of fine crystals embedded in what Brownmiller called the hydrogel.

Brownmiller found that the etched surface of the 7-day-old paste made of Type I cement showed "an extremely complicated but interesting structure. A close examination . . . shows that the cement hydrogel is not a formless mass but has an intricate structure."

*The electron microscope.* Eitel[11] used the electron microscope for photographing the hydration products of some of the constituents of portland cement. The article consulted gives almost no details concerning the method of preparing the samples that were photographed. It appears that the samples were taken from dilute suspensions of the hydrated material. Presumably, samples of $Ca(OH)_2$ were taken from saturated or supersaturated "milk of lime" and hydration products of $C_3S$ and $C_3A$ from saturated solutions of water in isobutyl alcohol. The isobutyl alcohol was used to dilute the water and thus make possible a relatively high concentration of the solid with respect to water and at the same time a dilute suspension. Since the isobutyl alcohol was saturated with water, the chemical activity of the water was unaffected by the presence of the isobutyl alcohol.

Several photographs of these preparations (magnifications ranging from 7200 to 36000) were published. The $Ca(OH)_2$ taken from milk of lime as well as that formed from the hydrolysis of $C_3S$ appeared as hemispheres ranging in size from about 0.1 to 0.5 micron. The calcium silicate hydrate appeared as thin crystalline needles about one-half

*See A.S.T.M. Designation C150-44, where five types of portland cement are defined as follows:
Type I —For use in general concrete construction when the special properties specified for Types II, III, IV, and V are not required.
Type II —For use in general concrete construction exposed to moderate sulfate action, or where moderate heat of hydration is required.
Type III—For use when high early strength is required.
Type IV —For use when a low heat of hydration is required.
Type V —For use when high sulfate resistance is required.

micron long. The $C_3A$ appeared mainly as rounded particles ("roses") about 0.1 micron in diameter. Referring to these and apparently to other observations, Eitel concluded that although the hydration products of portland cement are predominantly colloidal, they appear crystalline—not amorphous—to the electron microscope.

Sliepcevich, Gildart, and Katz,[12] reported the results of attempts to photograph the hydration products of portland cement and the major constituents hydrated separately. Most of the photographs published were of samples prepared as follows: 0.5 to 0.75 g of portland cement or a cement constituent was mixed with about 10 cc of purified water and allowed to stand. At the desired age the specimen for photographing was obtained by taking a drop of the supernatant liquid and allowing it to evaporate on a collodion film previously prepared. The material on this film was thus whatever dissolved or suspended material the drop contained.

In some respects the results were like those found by Eitel. Photographs of calcium hydroxide appeared like those of Eitel but whereas Eitel concluded that the particles were hemispheres, Sliepcevich, Gildart, and Katz concluded that they were spheres. From each material the latter investigators usually found material of several geometric forms. Some of the material appeared amorphous and some crystalline. As did Eitel, those investigators found the majority of crystals to be in the colloidal size range.

The significance of these results is open to question until it is known definitely just what relation the samples obtained in the manner described have to the hydration products making up the mass of a hardened cement paste.

#### X-ray examinations

The results of X-ray examinations were summarized by Giertz-Hedström[13] as follows: "X-ray examinations of hardened cement have so far given little beyond a confirmation of what has been shown by the microscope. The presence of clinker remains and crystallized calcium hydroxide is thus confirmed by Brandenburger.[14] The structure of the main mass, the "cement gel," is, however, such as to give, at least for the present, no clear guidance in the X-ray diagrams. This may be due to its lacking a crystalline structure or other regular fine structure or to the crystals being so small or deformed (for example bent needles) that no definite interferences are obtained."

Bogue and Lerch[15] mention the use of X-ray analysis in connection with microscopic examination. The X-ray confirms the microscopic indication that unaltered beta or gamma dicalcium silicate remained in pastes after 2 years of curing. X-ray diffraction patterns from both hydrated tricalcium silicate and hydrated dicalcium silicate showed at the end of 2 years no evidence of the development of a new crystalline structure such as would be expected if the hydrates were to change from the theoretically unstable gel state to the stable microcrystalline state. As mentioned above, evidence of the beginning of such a change within a period of 20 years was reported by Kühl.

#### Water fixation

Because direct observation fails to answer many questions concerning the structure, properties, and behavior of hardened portland cement paste, indirect methods of study have been used. The principal one is that of studying the manner in which water is held in the hardened paste.

*Isotherms and isobars.* The fixation of water by water-containing solids is usually measured in terms of the amounts of water held at various vapor pressures with temperature constant, or in terms of the amounts held at various temperatures with pressure constant. The curves obtained by the first method are called *isotherms.* Those obtained by the second method are called *isobars.* Both of these methods have been used in the study of hardened portland cement paste. The nature of the hydration or dehydration curve depends on the manner in which the water is combined and on other factors to be discussed.

*Binding of water in hydroxides.* In metallic hydroxides, which represent one class of compounds that may be included among hydrates, the elements of water are present as $OH$-groups that are strongly bound by the metallic ions. This is usually recognized in writing the formulas of metallic hydroxides; thus, calcium hydroxide is usually given the formula $Ca\ (OH)_2$ rather than $CaO.H_2O$.

*Water bound by covalent bonds.* In many hydrates, the water molecule retains its identity to a large degree, i.e., $H_2O$ is a unit in the structure. An example is $MgCl_2.6H_2O$. In this hydrate the six molecules of water are bound to the magnesium ion by covalent bonds and are arranged around the magnesium ion in an octahedral grouping. To indicate this, the formula should be written $Mg(H_2O)_6Cl_2$, for this more nearly represents the structure.

*Water bound by hydrogen bonds.* There is another type of compound in which the water molecule remains intact and is bound to the compound by one or both of its hydrogen atoms. The molecules so held are said to be bound by hydrogen bonds. $CuSO_4.5H_2O$ and $NiSO_4.7H_2O$ are hydrates in which one of the water molecules is bound in this way.

* * * * *

Water held in a compound by any of the types of bond described above is properly regarded as being chemically bound. The removal of water from such hydrates necessarily gives rise to a new solid phase and hence the isotherms and isobars of these hydrates should show well marked steps in accordance with the phase rule. Fig. 1a gives, for example, the relationship between water content and vapor pressure for the hydrates of copper sulfate.

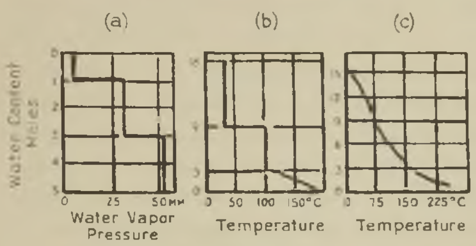



**Fig. 1 – Univariant and bivariant dehydration curves**

(a) Isothermal p-x curve for $CuSO_4.5H_2O$
(b) Isobaric T-x curve for $Cr_2(SO_4)_3.18H_2O$ The first $15H_2O$ come off along a univariant curve, the remaining $3H_2O$ being zeolitic
(c) Zeolitic dehydration of green $Cr_2(SO_4)_3.15H_2O$
(Curves and caption from Emeleus & Anderson)

*Zeolitic water.* One type of microcrystalline hydrate, which comprises the zeolites and several basic salts and hydroxides of bivalent metals, gives smooth isotherms or isobars. Fig. 1c is an example of an isobar from $Cr_2(SO_4)_3.15H_2O$. Water held in this type of compound is called zeolitic water.

According to Emeleus and Anderson[16] zeolitic water is regarded as being packed between the layers of the crystal or in the interstices of the structure. A distinguishing characteristic of zeolitic water is that it may be removed without giving rise to a new solid phase. Its removal may, however, change the spacing between successive layers of the crystal.

Water held in such a way as to exhibit the behavior described above is sometimes referred to as being in a state of zeolitic solution or solid solution.[17]

*Lattice water.* Emeleus and Anderson[16] distinguish a type of hydrate in which there is water of crystallization "that cannot be supposed to be associated chemically with the principal constituents of the crystal lattice." As an example, they cite potassium alum, $KAl(SO_4)_2.12H_2O$. There is little question that six of the twelve molecules of water are linked to the aluminum ion by covalent bonds. The remaining six molecules are known to be arranged octahedrally around the potassium ion but at such a large distance from it as to suggest to Emeleus and Anderson that the interaction is very weak and hence that the water is not chemically bound to the potassium ion. This perhaps represents a borderline case between chemically bound water and zeolitic water, which is supposed not to be chemically bound. The fact that exactly six molecules of water are associated with the potassium ion is accounted for by the geometry of the crystal lattice. The removal of these six molecules of water presumably gives rise to a new crystalline phase, however, and hence the isobars and isotherms should be stepped. When more information on this type of hydrate becomes available, perhaps many of the examples cited will have to be placed in one of the classifications listed above.

*Adsorbed water.* In addition to any water held by the chemical forces mentioned above, a small amount per unit of surface is held by surface forces. These forces, physical rather than chemical, are known collectively as van der Waal's forces.[18] If the specific surface of the solid phase is small, the amount so held is usually undetectable. But if the specific surface is very large, as it is for colloidal material, then physically adsorbed water can be a large fraction of the total held under given conditions; indeed, anhydrous solids such as quartz powder can hold relatively large amounts of water by surface adsorption if the powder is extremely fine. Zeolitic water can be regarded as adsorbed water, the "surfaces" in this case being certain planes in the crystal, as described above. The subject of adsorption will be treated much more fully in later sections of this paper.

#### Interpretation of isobars

*Influence of surface adsorption.* From what was said above it might appear that by means of isobaric or isothermal dehydration data water held in microcrystalline hydrates could readily be distinguished from that held as zeolite water, in solid solution, or by adsorption. It will be developed below that such a distinction can be drawn under some circumstances but not under others. The complication can best be illustrated by describing the work of Hagiwara.[19] Theoretically, the vapor pressure of the water in a very small crystal of a hydrate should be greater than that of a larger crystal of the same substance at the same temperature. Consequently, a mixture containing various sized crystals should exhibit a range in vapor pressures according to the range in particle size. Practically, the effect is not noticeable unless the particle size range extends into the range of colloidal dimensions. Theoretically, very small natural crystals or very small fragments of large crystals should behave similarly, though not necessarily identically. Consequently, the results of experiments with preparations made by pulverizing macrocrystals should be indicative of the general effects of changing particle size and particle-size range.

Hagiwara pulverized crystalline hydrates and obtained the isobar for each preparation. In one series of experiments, he used $Al_2O_3.3H_2O$ prepared by the method of Bonsdorff.[20] The original crystals were of microscopic size. The preparation was dried to constant weight in a desiccator over concentrated $H_2SO_4$ to establish the initial water content, which was determined by igniting a portion of the material. Samples thus dried were then heated in an electric oven at 100C for 30 minutes after which they were cooled in a desiccator and weighed. Finally, the amount of residual water was found by ignition. This was repeated on other samples at temperatures ranging from 90 to 220C as indicated in Fig. 2. Heating at 170C and at higher temperatures was continued until further heating caused no more change in the dry weight. The total heating period at these higher temperatures was not less than 5 hours and at 210C was 20 hours.

Although the corners are somewhat rounded, there is a well defined step in the isobar* at approximately 205C, where the water content decreases from 3 molecules to one molecule. This is in good agreement with the result obtained by Weiser and Milligan.[21] The rounded corner is the usual result in such experiments; very sharply defined corners are the exception.

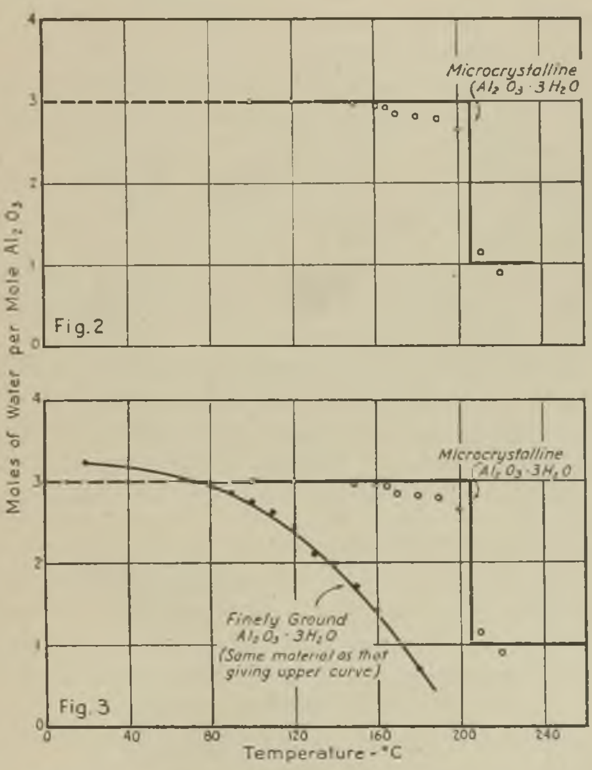



**Fig. 2 & 3 – Isobars for $Al_2O_3.3H_2O$**

Data from T. Hagiwara
Alexander: Colloid Chemistry
Vol. I, pp. 647-658
The Chemical Catalog Co.
Inc. (N.Y.) 1926

*Although the curves in Fig. 2 and 3 are called isobars and the text indicates that isobaric conditions were intended, it seems probable that isobaric conditions were not actually maintained. If the heating of the sample was done in an oven in the presence of room air, the actual vapor pressure in the oven would vary with the humidity of the room air and would be different at different temperatures. However, over the temperature range used in the experiment the variations in pressure were probably small. At any rate it is not likely that had strictly isobaric conditions been maintained the outcome would have been significantly different.

Hagiwara then ground a portion of the dried preparation in an agate mortar for 1½ hours, using fine quartz powder as a grinding aid, thus greatly reducing the particle size. The experiments were repeated with this finely ground material with the results shown in Fig. 3, where the results shown in Fig. 2 are reproduced for comparison. A comparison of the curves in Fig. 3 shows three significant effects of reducing the size of the crystals:

(1) All semblance of a step is absent in the curve for the finely ground sample.

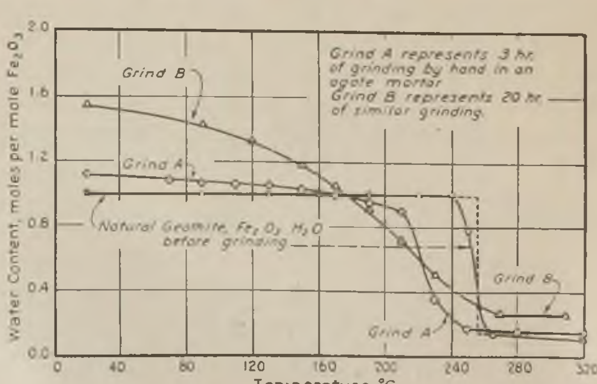
(2) The initial water content of the finely ground material is higher by 0.25 mole than that of the unground material. This additional water must have come from the atmosphere during the grinding. (The initial water content is that of the sample after it is dried to constant weight in a desiccator over conc. $H_2SO_4$.)

(3) The water is lost from the finely ground sample at a much lower temperature than from the unground material.

Hagiwara made a similar series of experiments with $Fe_2O_3.H_2O$ with the results shown in Fig. 4. The curve for the unground macrocrystalline material shows a well defined step. The curve for "Grind A," the shorter period of grinding, still shows a step though the corners are somewhat more rounded than for the unground hydrate and the step occurs at a lower temperature. When the grinding was more prolonged, "Grind B," the step disappeared and the initial water content increased to 1.54 molecules. Grinding had little effect on the temperature at which the final water content was reached.

It appears from Hagiwara's results that a hydrate in which the water is bound by chemical bonds may still yield a smooth isobar if the sample is made up of a mixture of particles of various sizes, the smallest particles being very small. This has a significant bearing on the interpretation of isobars in general. When a solid that has water for one of its con-

**Fig. 4 – Effect of prolonged grinding on the isobars of $Fe_2O_3.H_2O$**
Hagiwara, Colloid Chemistry Alexander Ed., Vol. I. pp. 647-58 (1926)



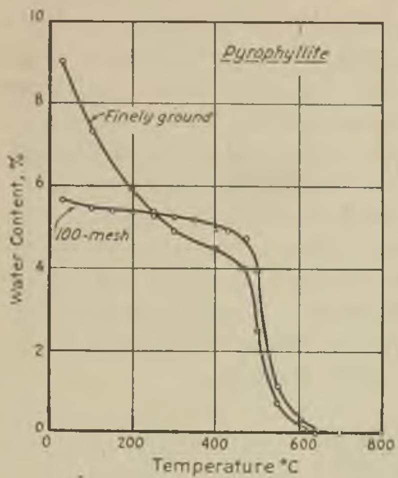



**Fig. 5 – Isobar for pyrophyllite**
Data from Kelley, Jenny and Brown Soil Sci. v. 41, p. 260 (1936)

stituents yields a stepped isobar, it can usually* be concluded that the solid is a well crystallized hydrate. When, however, the isobar is a smooth curve without steps, it may represent a sample of the hydrate comprising particles of various sizes or it may represent a material in which the water is not bound by chemical bonds in the usual sense of this term. It appears also that a lack of steps in an isobar is not sufficient evidence that the hydrate has a zeolitic structure, for neither of the hydrates with which Hagiwara experimented was of that nature.

The other effect of fine grinding, namely, the increase in the initial water content brought about by grinding, is fully as significant as the one just mentioned. It must be assumed that the water in excess of that required by the formula is held by forces of a kind different from those that hold the hydrate water. The effect is clearly a surface effect, for the initial water content is much higher after the longer period of grinding than after the shorter period. (Compare "Grind B" with "Grind A" in Fig. 4.) It seems reasonable to suppose that this excess water is held by adsorption forces, i.e., forces which reside in the surface of the crystal and which came into prominence after the specific surface of the hydrate had been greatly increased by grinding.

This aspect of the effect of fine grinding is further emphasized by the results of experiments made by Kelley, Jenny, and Brown.[22] These authors studied the effect of grinding on the isobars of clay minerals. See Fig. 5. The isobar for the 100-mesh sample shows a well marked step near 500C and thus gives unmistakable evidence that the mineral is a microcrystalline hydrate. Comparing this with the isobar for the finely ground pyrophyllite, we see that the isobars are very nearly

*But not always. See remarks below on silicia gel.

identical above 400C. Each exhibits a well marked step; the steps occur at very nearly the same temperature; however, the height of the step is slightly less for the finely ground sample. Below 250C, the finely ground sample shows a larger water content than the 100-mesh sample. That is, just as with the materials Hagiwara used, after grinding there is initially a large amount of water in the finely ground sample in excess of the hydrate water represented by the nearly vertical portion of the isobar. This additional water must have been acquired from the atmosphere during grinding and must be held by some mechanism other than that which holds the hydrate water. It seems most unlikely that the excess water is zeolitic water, since fine grinding could hardly increase the total interplanar area of zeolite crystals.

#### Interpretation of isotherms

*Isotherms of hydrates.* The usual behavior of a hydrate when the water-vapor pressure around it is varied at constant temperature is illustrated in Fig. 6. This isotherm shows well defined steps with the corners slightly rounded, as indicated by the dotted lines. What the effect of pulverizing such material on the shape of the isotherm might be is not known directly from experiment, for apparently no experiments of this kind have been published. Presumably, a sufficient amount of grinding would produce a smooth isotherm, by virtue of the change in particle size and particle-size range. It might also be presumed that in a saturated condition the minute crystals would retain more water than corresponds to the highest hydrate. These presumptions follow from considerations given above in connection with the isobars.

*Isotherms of gels.* The appearance of a well defined step in the isotherm of a solid is not always positive proof of the existence of a hydrate of definite chemical composition. Fig. 7 illustrates this point.[23] As the arrow indicates, the isotherm was obtained by progressively lowering the water vapor pressure. Just below a pressure of 4 mm Hg, the isotherm becomes very steep, a fact which might be taken to indicate the existence of hydrates having the formulas $SiO_2.1\frac{1}{2}H_2O$ and $SiO_2.H_2O$. Fig. 8 shows the same isotherm as well as that obtained by progressively increasing the vapor pressure, the "rehydration isotherm" as it is sometimes called. Note that the latter gives no evidence of a hydrate. Weiser, Milligan, and Holmes investigated this matter fully by preparing silica gel from the same materials at various temperatures ranging from 0 to 100C and from other materials at various temperatures and under various conditions. Not all preparations exhibited the vertical section in the dehydration isotherm. Gels prepared at low temperatures exhibited a step at a higher water content than gels prepared at a high temperature. The samples prepared at 100C and aged at this temperature for a few hours, and hence under conditions favoring crystal

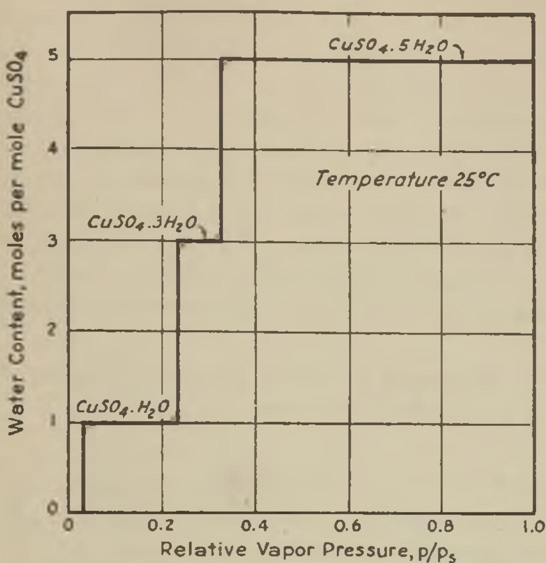
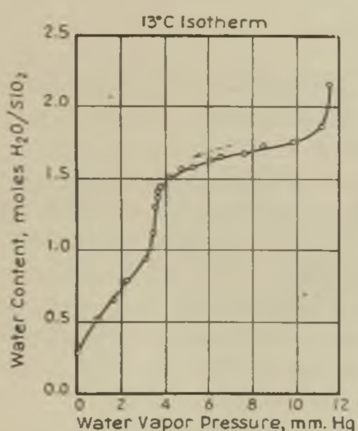



**Fig. 6 – Water content vs. vapor pressure curve of $CuSO_4$**

Collins and Menzies, J. Phys. Chem. v. 40, pp. 379-97 (1936)



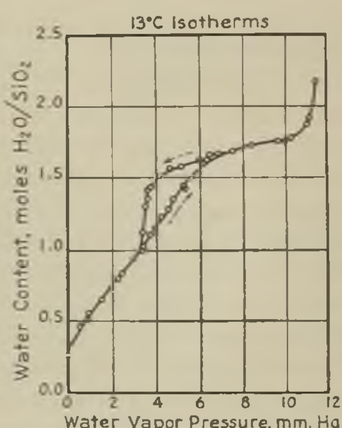




**Fig. 7—(left) Dehydration isotherm for silica gel prepared from water glass at 25C**

Weiser, Milligan and Holmes, J. Phys. Chem. v. 46, p. 586 (1942)

**Fig. 8 (right)—Dehydration and rehydration isotherms for silica gel prepared from water glass at 25C**

Weiser, Milligan and Holmes, J. Phys. Chem. v. 46, p. 586 (1942) growth, gave no evidence of a step. No evidence of a step appeared in the rehydration isotherm of any preparation. No preparation gave evidence of crystallinity when examined by the electron diffraction method. These authors concluded that the step that occurs in the dehydration isotherm of some of the preparations is not evidence of the existence of a hydrate but is rather the result of a peculiarity of the physical structure of these gels. It follows from this conclusion that the structure of the gels prepared at low temperatures differs from that of the gels prepared at high temperatures, and there is evidence that this change in structure accompanying the increase in temperature of preparation is progressive.

Fig. 8 illustrates also the phenomenon called "sorption hysteresis." This will be discussed in another part of this paper.

* * * * *

The foregoing review shows that the interpretation of isobars and isotherms is not always a simple matter. Particularly, when the isotherm is smooth throughout, alternative interpretations must be considered carefully in the light of other pertinent information.

#### Water content vs. temperature curves (isobars) from hardened portland cement paste

*Review of published data.* In this laboratory Wilson and Martin[24] obtained a group of isobars from hardened portland cement paste. The hardened paste was ground to pass the 28-mesh sieve and then was dried to constant weight at a constant temperature in a stream of air maintained at a low, constant water vapor pressure by bubbling the air through concentrated sulfuric acid.* The dry sample was then ignited at 1000C and the loss on ignition was taken as the water retained at the temperature of drying. Various drying temperatures were used, ranging from 50 to about 600C.† The resulting isobars are given in Fig. 9. Lea and Jones[25] obtained the isobars shown in Fig. 10. These authors plotted the loss in water rather than the amount retained.

In general those curves are not like those obtained from crystalline hydrates having definite amounts of water of crystallization. The nearly vertical rise in the curves between 400 and 450C, seen clearly in the curves of Lea and Jones, is attributed to the decomposition of $Ca(OH)_2$.

Krauss and Jörns[26] obtained isobars for the hardened paste at a constant vapor pressure of 7 mm $Hg$ by using the instrument shown in Fig. 11. The sample is placed in A, which can be detached from the rest of the apparatus and which serves as a weighing bottle in following the changes in weight of the sample. With A in place as shown, the

*The air was freed of $CO_2$ by suitable means to prevent carbonation.
†The procedure used does not maintain strictly isobaric conditions, for the pressure would vary with temperature. However, the pressure is so small under all conditions that variations can usually be neglected.

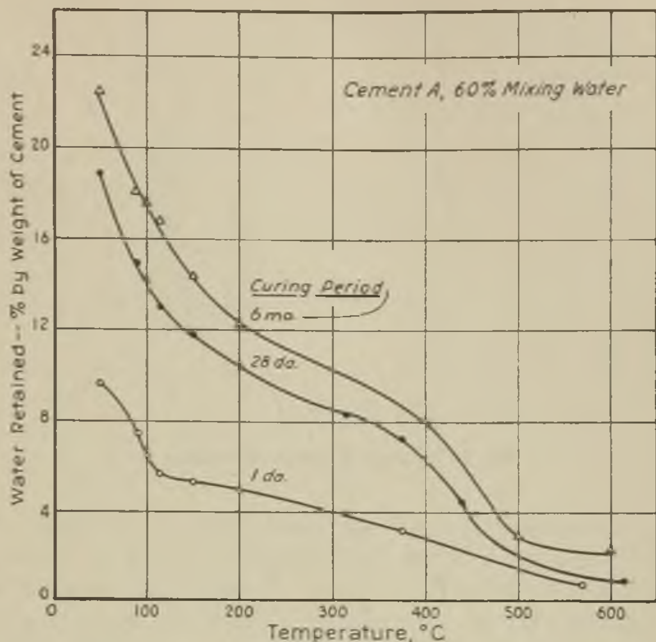



Fig. 9 – Effect of drying temperature on water retained, temperature range 50 to 600C
Wilson and Martin, J. Am. Concrete Inst. v. 31, p. 272 (1935)

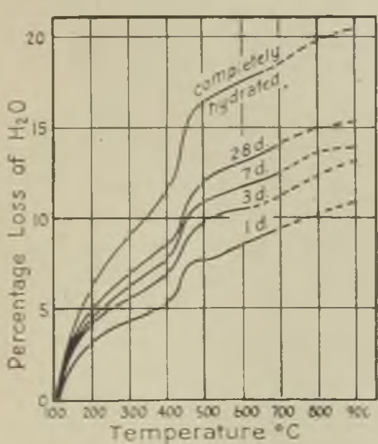



Fig. 10—Loss-on-heating curves for hardened neat cement, w/c = 0.22
Lea & Jones, J. Soc. Chem. Ind., v. 54, p. 63, 1935

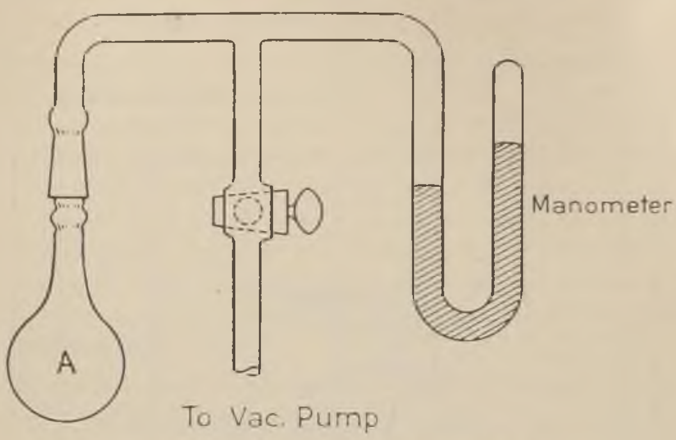



**Fig. 11 – Krauss & Jorns' Apparatus**

system is evacuated, thus removing water vapor as well as air. Periodically, the removal of water is interrupted by closing the stopcock and the water vapor pressure is observed. If the equilibrium pressure exceeds 7 mm *Hg*, the value chosen by Krauss and Jörns, the stopcock is opened and more water is removed by pumping. This process is repeated until at room temperature the equilibrium vapor pressure of the sample is less than 7 mm *Hg*. The temperature of the sample is then slowly raised until the pressure is exactly 7 mm. The loss of water to

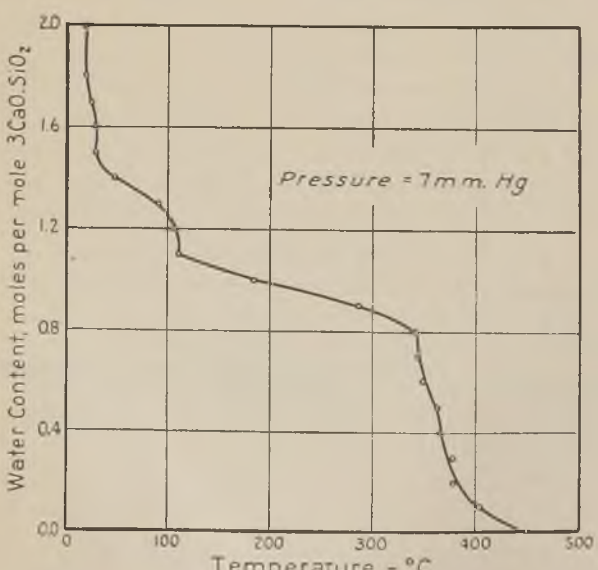



**Fig. 12 – Isobar for hardened $3CaO.SiO_2$***

*Probably a mixture of $C_2S$ and $Ca(OH)_2$ (See text)
Data from Krauss and Jorns, Zement v. 20, p. 317 (1931)

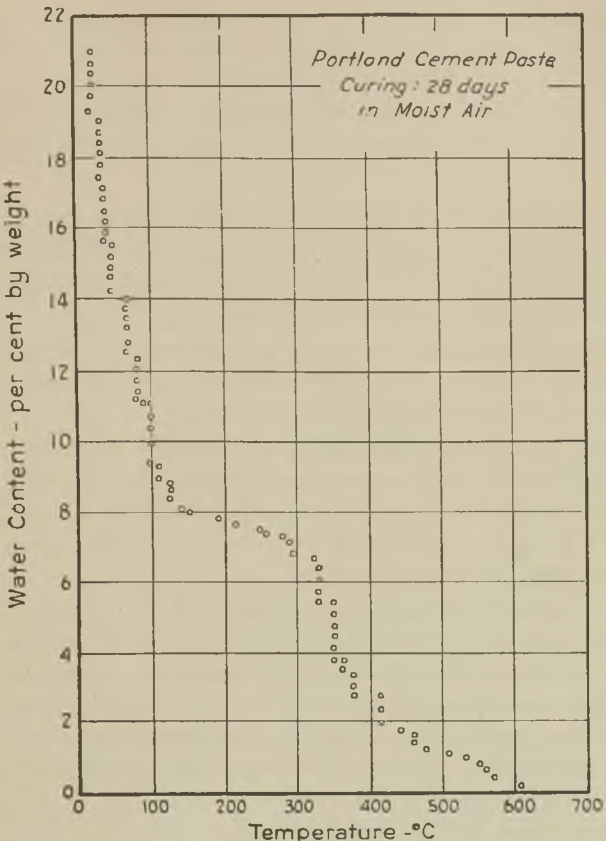



**Fig. 13 – Isobar for portland cement paste obtained by Krauss and Jorns.**
Zement v. 20, p. 343 (1931)

this point is determined by weighing A. Beyond this point, water is removed in small amounts by evacuation and after each decrement the temperature of the sample is raised until the pressure is exactly 7 mm *Hg*. Thus, the isobar is obtained. The results obtained by Krauss and Jörns are given in Fig. 12, 13, and 14.

Fig. 12 represents a sample described as tricalcium silicate ($C_3S$)* mixed with enough water to correspond with the formula $3CaO.SiO_2.$-$2H_2O$. The mixture was cured in saturated air 24 hours before the measurements were started. There is a *suggestion* of a step in the isobar

*Throughout this discussion the compositions of portland cements will be described in terms of the compound composition computed by the methods of L. A. Dahl, *Rock Products* v. 32 (23) p. 50, (1929) and R.H. Bogue, *Ind. Eng. Chem.* (Anal. Ed.) v. 1 (4) p. 192, (1929) or PCAF Paper No. 21 (1929). The following abbreviations will be used.

$$C_3S = 3CaO.SiO_2 \qquad C_3A = 3CaO.Al_2O_3$$
$$C_2S = 2CaO.SiO_2 \qquad C_4AF = 4CaO.Al_2O_3.Fe_2O_3$$

On this basis the composition is computed on the assumption that the iron and alumina compunds are $C_4AF$ and $C_3A$. Swayze, Am. J. Sci. v. 244, pp. 1-30, 65-94, (1946) has recently shown that the iron oxide occurs in a phase having the general formula $C_6A_xF_{(3-x)}$ (in which $x$ may vary from 0 to 2) which includes $C_4AF$ as a special case. After it becomes possible to make allowance for this finding, discussions in later parts of this paper may need recasting in different terms. It seems unlikely, however, that any of the arguments or deductions would be greatly altered.

near 100C and a prominent step near 350C. The latter probably represents the decomposition of calcium hydroxide. A compound that might be represented by the break at 100C is not identified. Owing to the manner of its preparation (from a melt), the material thought to be tricalcium silicate was probably a mixture of dicalcium silicate and calcium hydroxide.

Fig. 13 represents a paste cured 28 days in moist air. There appear to be numerous closely spaced steps in this isobar. Krauss and Jörns believed that these steps were sufficiently distinct to indicate the presence of several hydrates having definite chemical formulas.

Fig. 14 represents a paste that had been cured several hours in boiling water* and then stored in the air of the laboratory for several years. As can be seen, the isobar has several well defined steps.

Endell(27) determined isobars for hardened paste, but he arbitrarily limited the heating period at any one temperature to one hour. There is evidence in his results that this was not always sufficient for the attainment of equilibrium. The isobars he obtained do not exhibit any features not exhibited by those reproduced in this paper.

Meyers(28) published isobars for hardened pastes and for hydrated samples of the pure compounds $C_3S$, $C_2S$, $C_3A$, $C_4AF$, and $CaO$ hydrated separately (Fig. 15). The samples were heated in a vacuum in which the water vapor pressure was maintained at about 0.1 micron. Under these conditions the dissociation temperature of calcium hydroxide was found to be about 380-400F (193-204C). Neat hydrated cement heated under the same conditions showed a step at about 430F; presumably the step was due to the decomposition of calcium hydroxide. The conditions described for the experiments were such as to suggest that the difference between the dissociation temperature of the sample of $Ca(OH)_2$ and that of the $Ca(OH)_2$ in the hydrated cement was probably due to a difference in vapor pressure, the pressure being higher under the conditions that prevailed when the hydrated cement was tested.

The isobars for hydrated $C_3S$ and $C_2S$ show steps near 400F that may also be attributed to calcium hydroxide. The isobars for hydrate $C_3A$ and $C_4AF$ show a large step near 240F.

*Discussion of isobars.* The data of Krauss and Jörns seem to indicate that hydrated cement contains a series of hydrates besides $Ca(OH)_2$. However, the other curves, including those published recently by Meyers, indicate that the curves are smooth, except for one step in the neighborhood of 400F that is apparently due to microcrystalline $Ca(OH)_2$.

Since the structure of the hardened paste is predominantly of submicroscopic texture, a smooth isobar is to be expected whether the

*The specimen was a part used in the German Standard Test for Soundness.

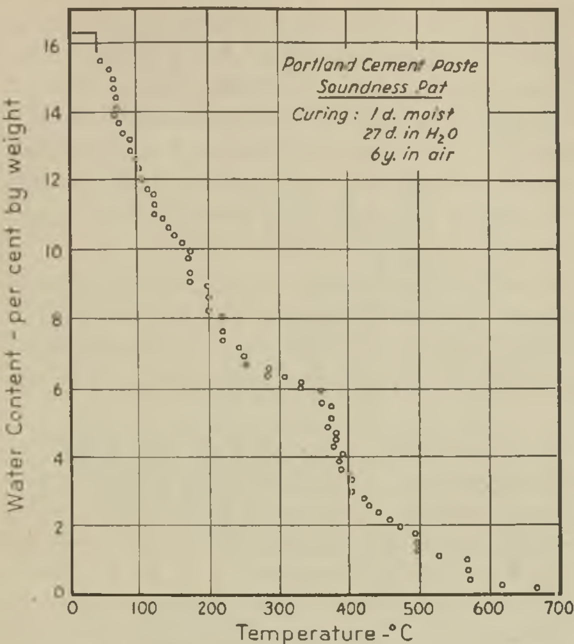



**Fig. 14 – Isobar for portland cement paste obtained by Krauss and Jorns.**
Zement v. 20, p. 343 (1931)

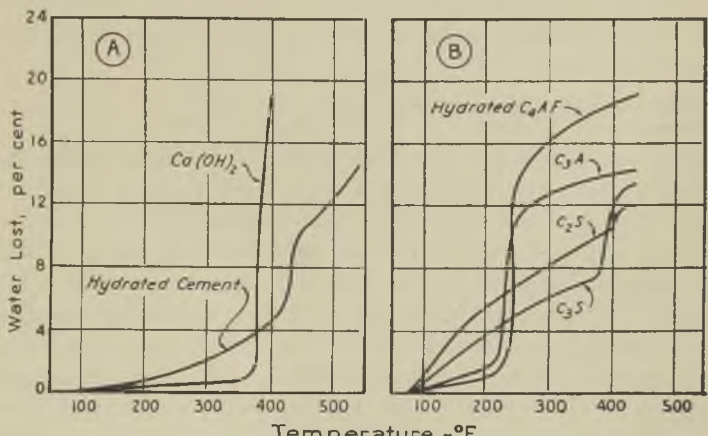



**Fig. 15 – Isobars obtained by S. L. Meyers**
Pit and Quarry (July, 1942)

hydration products are crystalline or not. As will be shown later, the specific surface of the hydrated material is so high that if the structure is of granular nature the granules must be of colloidal dimensions. If the particles were all of exactly the same size and constitution, a step in the isobar might be expected, despite the smallness of the particles. Without any a priori reason for assuming such uniformity in particle size, there is no basis for expecting anything but a smooth isobar except for the effect of calcium hydroxide already noted.

In general the isobars do not tell much about the hydration products. However, when the isobars are considered together with the information obtained with the microscope and X-ray, they may be considered to show that the hydration products are, for the most part, not in the microcrystalline state.*

Meyers' data on the four compounds $C_3S$, $C_2S$, $C_3A$, and $C_4AF$ hydrated separately are of special interest. Note that both of the alumina-bearing compounds lost large amounts of water at about 250F. Since these two compounds usually constitute 20 per cent or more of the cement, a step on the curve for portland cement, or at least a sharp increase in slope, should occur at about 240F (116C) if those compounds are present in the hydrated cement. The fact that no such indication has been found can be taken as evidence that the hydration products of cement are not a simple mixture of the same hydration products that form when the compounds are hydrated separately; at least, they differ radically with respect to physical state, whether they do with respect to constitution or not.

**Water content vs. vapor pressure curves at constant temperature (isotherms)**

Jesser[30] was apparently the first to study the relationship between water content and vapor pressure for hardened paste at constant temperature. He used the method of van Bemmelen,[31] i.e., he left the specimens in a closed container over a solution of a salt or of $H_2SO_4$ at constant temperature, until they reached equilibrium with the vapor pressure of the solution. Starting with the saturated condition, he progressively subjected the samples to lower and lower humidities, finally drying them over concentrated $H_2SO_4$. He then subjected them to progressivly higher humidities, finally storing them over a solution having a relative vapor pressure of 0.98. After this he determined a second drying curve. His results are given in Fig. 16.

*Throughout this discussion a distinction will be made between the colloidal and microcrystalline states. The term "colloid" refers to any material, crystalline or not, having a specific surface of a higher order than particles visible with the light microscope. A colloidal material may exist as discrete particles or as gels, the latter being regarded as aggregations of once discrete colloidal particles. The term "microcrystalline" refers to ordered aggregations of molecules, atoms, or ions, at least large enough to be seen with a light microscope. The adjective "amorphous" is not used as the synonym of "colloidal" for such usage tends to obscure the fact that colloidal particles may themselves be crystalline, that is, of ordered structure. The possibility of crystalline colloids has long been recognized by colloid chemists. The actual existence of such colloids has been demonstrated by means of the electron microscope.[29]

Jesser used prisms about 1½x1½x4 in. made of neat paste having a water-cement ratio of 0.25 by weight. On the average, he left each prism at a given humidity about six weeks and assumed that this was sufficient time for equilibrium to be attained. Experiments in this laboratory, in which prisms of smaller cross section (1x1 in.) having a higher water-cement ratio (and consequently, a higher drying rate) were dried over dilute $H_2SO_4$ by much the same procedure, showed that 6 weeks was insufficient for the attainment of constant weight. Moreover, in specimens as large as those used by Jesser, hydration of the cement continues at an appreciable rate at the higher relative vapor pressures, especially in specimens cured only three days. These facts make the interpretation of his results difficult. The results are of interest none the less because they were the first to indicate the similarity between the behavior of the hardened paste and that of typically colloidal substances such as silica gel.

One striking fact in Jesser's results is that the first drying curve is not reversible. Work done in this laboratory confirms this.

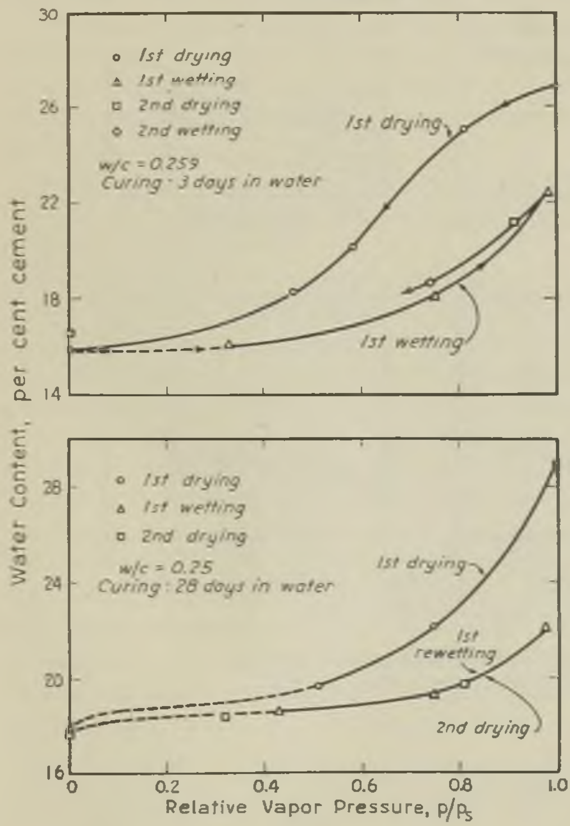



**Fig. 16 – Water content versus relative vapor pressure.**
Leopold Jesser, Zement v.16, p.741 (1927)

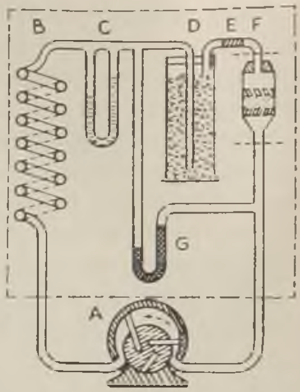



**Fig. 17 — Diagram for apparatus used by Giertz-Hedstrom for determining isotherms for hardened cement paste**

Zement v. 20, p. 672 (1931)

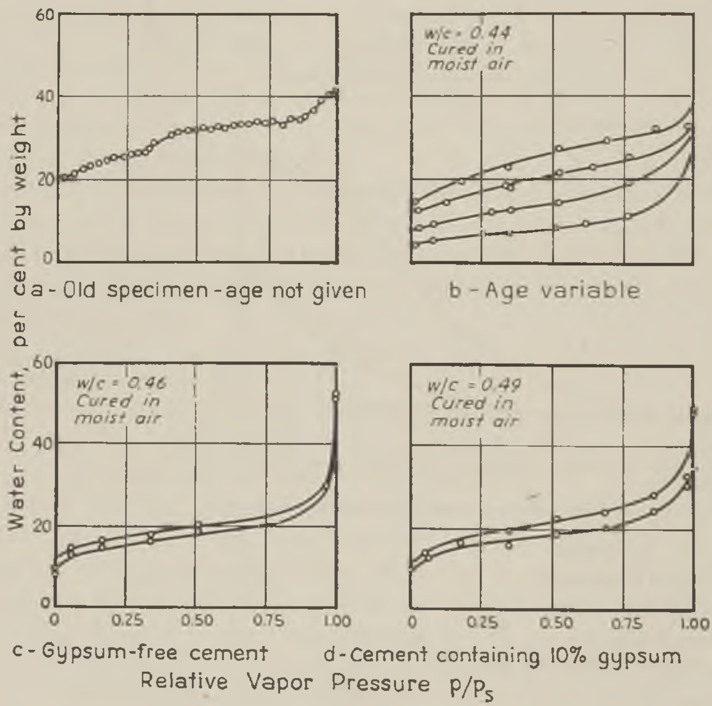



**Fig. 18 — Isotherms for hydrated cement**

Giertz-Hedstrom, Zement v. 20, p. 734 (1941)

Giertz-Hedström[32] published a number of isotherms for hardened paste. His procedure can be explained with the aid of Fig. 17. A pump, A, circulated air through a copper coil, B, a flow-meter, C, a wash bottle, D, and the sample-holder, F. The wash bottle, D, contained dilute $H_2SO_4$ and was partly filled with glass beads. The manometer, G, measured the pressure drop across D and F. The apparatus, except for the pump, was kept at a constant temperature in an air thermostat. A 1-g sample of pulverized, saturated paste was placed in F, which was detachable and served as a weighing bottle. Air was passed through the sample until periodic weighing showed that it had reached constant weight. When this point was reached, the $H_2SO_4$-solution first used in D, which was made very dilute to give a high relative vapor pressure, was replaced by a more concentrated solution and the sample was dried to constant weight at the lower relative vapor pressure. This procedure was repeated with progressively more concentrated $H_2SO_4$-solutions, and ended with concentrated $H_2SO_4$, so that several points were obtained along the isotherm. Giertz-Hedström's results are given in Fig. 18.

Berchem[33] has published isotherms for hardened cement and for hardened specimens of three of the principal compounds of portland cement. His method was essentially the method of Giertz-Hedström. As shown in Fig. 19 he obtained no experimental points at vapor pressures above $p = 0.63\ p_s$.

So far as the authors know, Jesser, Giertz-Hedström, and Berchem are the only investigators who have published isotherms for hardened portland cement pastes.*

*Relationship between isotherms and isobars.* In general, isotherms and isobars give information about the fixation of different portions of the water in the hardened paste. The isotherms give information about the fixation of that part of the water that is evaporable at a constant temperature, usually near room temperature; the isobars, on the other hand, give information about the fixation of the water that is not evaporable at room temperature. An exception is the work of Krauss and Jörns who, instead of determining isobars at a very low water vapor pressure as is commonly done, determined isobars at a vapor pressure of 7 mm *Hg*. This corresponds to a relative vapor pressure of approximately 0.3 at 25C. Thus, the range of their isobars overlaps the lower part of the isotherms given above as well as those determined in this laboratory.

*Significance of isotherms.* The isotherms, like the isobars, indicate that the hydration products are predominantly colloidal. They can be interpreted so as to give information about the volume and surface

*Gessner[34] has also studied the relationship between water content and relative vapor pressure of cement pastes. However, instead of determining a complete isotherm for a single sample, a different sample was used at each relative vapor pressure. Moreover, the procedure was such that the extent of chemical reaction and the water-cement ratio were different for each. Thus, many variables are involved and it is difficult to interpret the curves.

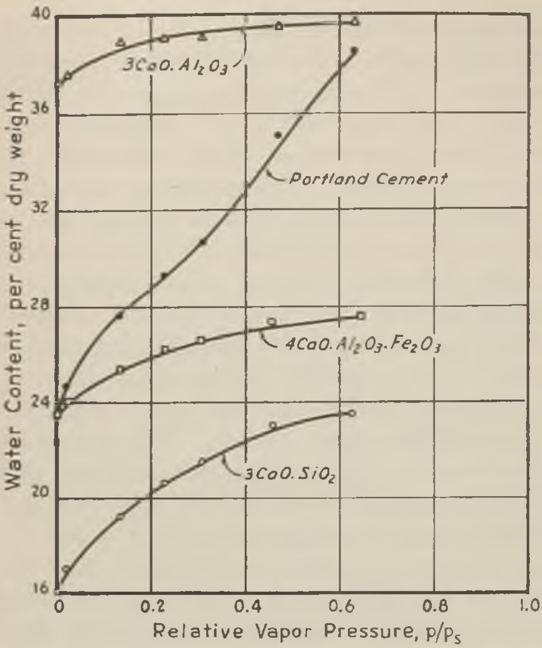



**Fig. 19 — Water content vs. vapor pressure relationship**
Berchem, Diss. Eidg. Techn. Hochschule, Zurich (1936)

area of the solid phase and other significant features of the properties and behavior of hardened paste. The presentation of such data and their interpretation are the main purpose of this paper.

#### Studies of water fixation by means of freezing tests

Studies of water fixation by means of freezing tests in various materials such as soils and plants have been reported by several investigators. Similar studies of hardened portland cement paste were reported by Giertz-Hedström(35) and by von Gronow(36). With respect to this method Giertz-Hedström(37) says: "In all these tests the treatment is comparable with a reduction in water vapor pressure, that is to say, a form of drying out but with the addition of a different complication for each method."*

In a later paper the work done in this laboratory on the freezing of water in hardened portland cement pastes will be described.

### SUMMARY OF PART 1

The material presented in Part 1 attempts to review the most significant information obtained by other investigators on the physical properties of hardened portland cement paste. It describes the experimental procedures and presents the test data of several important investigations.

*See also F. M. Lea, *Cement and Cement Manufacture*, v. 5, p. 395 (1932).

From reported microscopic studies it can be concluded that hardened cement paste is predominantly of submicroscopic texture. The only microcrystalline hydrate consistently reported is calcium hydroxide. Brownmiller found this to occur in clusters of very small crystals in the "cement gel." Although the gel state is theoretically unstable, Bogue and Lerch found no evidence of change toward the microcrystalline state over a period of 2 years. However, Kühl found evidence of such a change in pastes about 20 years old.

X-ray analyses do no more than confirm results of the microscopic method.

The relatively few reported observations made with the electron microscope indicate that the hydration products of portland cement may be colloidal but not amorphous. That is, they may be made up of submicroscopic crystals.

Several studies of physical properties of paste have been made by studying the fixation of water in hardened portland cement paste. Such studies are based on the fact that the characteristics of hydration or dehydration curves depend on both the physical and chemical characteristics of the materials involved. In hydroxides the water loses its chemical identity and appears in the structure as $OH$-groups. In many compounds it is bound molecularly by covalent bonds. In a third type of compound some of the water is bound by hydrogen bonds. In all the types of binding just mentioned the amount of water combined can usually be represented in a definite chemical formula and when the particles are of microscopic size or larger, such hydrates are stable through definite ranges of temperature and pressure.

In bodies of the zeolite type the water molecules are believed to be packed in the interstices of the solid structure and they are relatively loosely bound to the solid.

Any solid is capable of holding a small amount of water or other substance on its exposed surface by adsorption. The quantity held in this manner can be large when the specific surface of the solid is very high.

The amount of zeolitic water or adsorbed water held by a solid depends on the temperature and pressure of the water vapor surrounding the solid and the amount varies continuously with changes in either pressure or temperature.

A graph of the relationship between water content and temperature at constant vapor pressure is called an "isobar." The shape of the isobar of a hydrous solid depends on the specific surface of the solid and upon the nature of the combination between the solid and the water. Microcrystalline hydrates give stepped isobars having one or more steps.

The same hydrates reduced to submicroscopic particles of various sizes produce smooth isobars, showing more water in combination at the lower temperatures and less water at the higher temperatures than does the same material in the microcrystalline state. The isobar for zeolitic or for adsorbed water is a smooth curve under all conditions.

A graph of the relationship between water content and water vapor pressure at constant temperature is called an "isotherm." For microcrystalline hydrates the isotherm, like the isobar, is stepped. The material retains more water at low vapor pressures and less at high vapor pressures than does the same material in the microcrystalline state. This conclusion is based partly on experimental data and partly on inference from the isobaric relationships.

Under certain conditions isotherms from silica gel show a step similar to that of a microcrystalline hydrate. The resemblance is superficial, however, as other information shows that no definite hydrate exists over any given pressure range.

Isobars from portland cement paste have been obtained by several investigators. The findings of different investigators vary in some details. In general they show that the isobar is a smooth curve except for one step that is attributed to the decomposition of calcium hydroxide.

Isobars from the four principal compounds of portland cement, hydrated separately, show that the hydrates of $C_3S$ and $C_2S$ resemble that from portland cement, whereas the hydrates of $C_3A$ and $C_4AF$ show mainly the characteristics of microcrystalline hydrates, a step occurring at about 240F when vapor pressure is about 0.1 micron. The fact that Meyers found no step on the isobars for portland cement at 240F is evidence that the hydrates of $C_3A$ and $C_4AF$ that occur in portland cement are not the same, at least with respect to physical state, as those which form when these compounds are hydrated separately.

Isotherms from portland cement pastes have been obtained by a few investigators. The isotherms give information about the fixation of that part of the water that is evaporable at a constant temperature, usually near room temperature, whereas the isobars give the information about the fixation of that part of the water that is not evaporable at room temperature. The isotherms that have been obtained agree with the isobars in indicating that the water in hardened paste is *not* held as it is in microcrystalline compounds. Instead, the manner of binding is similar to that between water and silica gel.

Some studies of water fixation by a freezing-out procedure have been reported. The method is fundamentally similar to the drying-out procedure and the results obtained have about the same significance.

The data on the isothermal relationship between water content and vapor pressure obtained before the present investigation are too few to throw much light on the question of paste structure. Several of the earlier investigations were conducted under what are now known to be faulty test conditions.

## REFERENCES

(1) E. Freyssinet, *Science et Industrie*, Jan., 1933.

(2) S. Giertz-Hedström, "Proc. Symposium on the Chemistry of Cements," p. 505 (Stockholm 1938).

(3) R. H. Bogue, "A Digest of the Literature on the Nature of the Setting and Hardening Processes in Portland Cement," Portland Cement Association Fellowship Paper No. 17 (Oct. 1928).

(4) L. S. Brown and R. W. Carlson, *Proc. A.S.T.M.* v. 36, Pt. II, 332 (1936).

(5) Hans Kühl, "Cement Chemistry and Theory and Practice," p. 34, Concrete Publications Ltd., London.

(6) Ref. 5, p. 36.

(7) Henri Le Chatelier, *Compt. rend.* v. 94, p. 13 (1882) "The Constitution of Hydraulic Mortars," (1887). Translated by J. L. Mack, McGraw-Hill, New York (1905).

(8) L. T. Brownmiller, ACI JOURNAL, Jan. 1943; *Proceedings* v. 39, p. 193 (1943).

(9) B. Tavasci, Richerche sulla Constituzione del Clinker di Cemento Portland, *Giorn. chem. ind. applicata*, p. 583 (Nov. 1934).

(10) H. Insley, *J. Res.* Natl. Bur. Stds. v. 17, p. 353 (1936).

(11) W. Eitel, *Angewandte Chemie*, v. 54, p. 185-193 (1941).

(12) C. M. Sliepcevich, L. Gildart and D. L. Katz, *Ind. Eng. Chem.* v. 35, p. 1178 (1943).

(13) Stockholm Symposium, p. 513. (See Ref. 2)

(14) E. Brandenburger, *Schweizer Archiv*, p. 45, 1937.

(15) R. H. Bogue and Wm. Lerch, *Ind. Eng. Chem.* v. 26, p. 837 (1934), or Paper No. 27, Portland Cement Association Fellowship, National Bureau of Standards, Washington, D. C.

(16) H. J. Emeleus and J. S. Anderson, "Modern Aspects of Inorganic Chemistry," D. van Nostrand, Inc., p. 163, New York, 1942.

(17) R. M. Barrer, *Trans. Faraday Soc.* v. 40, p. 374 (1944).

(18) Stephen Brunauer, "The Adsorption of Gases and Vapors," Vol. I, Chapter 7, Princeton University Press, 1943.

(19) T. Hagiwara. In "Colloid Chemistry" by J. Alexander, Chapter 38, The Chemical Catalog Co., 1926.

(20) P. A. Bonsdorff, *Pogg. Ann.* v. 27, p. 275 (1833).

(21) H. B. Weiser and W. O. Milligan, *J. Phys. Chem.* v. 38, p. 1175 (1934).

(22) W. P. Kelley, Hans Jenny, and S. M. Brown, *Soil Sci.* v. 41, p. 260 (1936).

(23) H. B. Weiser, W. O. Milligan and J. Holmes, *J. Phys. Chem.* v. 46, p. 586 (1942).

(24) R. Wilson and F. Martin, ACI JOURNAL, Jan.-Feb., 1935; *Proceedings* v. 31, p. 272.

(25) F. M. Lea and F. E. Jones, *J. Soc. Chem. Ind.* v. 54, p. 63 (1935).

(26) F. Krauss and G. Jörns, *Zement* v. 20, pp. 314, 341 (1931); also *ibid* v. 19, p. 1054 (1930).

(27) K. Endell, *Zement*, v. 15, p. 823 (1926).

(28) S. L. Meyers, *Pit and Quarry*, v. 35 p. 97, July, 1942.

(29) Thomas F. Anderson, "The Study of Colloids with the Electron Microscope," from "Recent Advances in Colloid Science," Interscience Pub. Co., New York, (1942).

(30) L. Jesser, *Zement* v. 16, p. 741 (1927); v. 18, p. 161 (1929). Earlier reports in "Protokoll der Generalversammlungen des Vereines der Osterr." *Zementfabrikanten,* 1912, 1913, and 1914.

(31) J. M. van Bemmelen, "Die Absorption," Dresden, 1910.

(32) S. Giertz-Hedström, *Zement* v. 20, pp. 672, 734 (1941).

(33) Hans Berchem, Dissertation, Eidg. Tech. Hochschule, Zürich, 1936.

(34) H. Gessner, *Kolloid Zeitschrift* v. 46 (3) (1928); v. 47 (1-2) (1929).

(35) S. Giertz-Hedström, *Zement* v. 20, p. 672 (1931).

(36) H. E. von Gronow, *Zement* v. 25 (1936).

(37) Stockholm Symposium, p. 517. (See Ref. 2)

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**Part 2—Studies of Water Fixation, is scheduled for the November 1946 Journal**

[To facilitate selective distribution, **separate prints** of this title (43-6) are currently available **in covers** from ACI at 50 cents each—quantity quotations on request. **Discussion** of this report (copies in triplicate) should reach the Institute not later than March 1, 1947.]

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## ACI Standard

# Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)*

REPORTED BY ACI COMMITTEE 711

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## GENERAL

### 10. Scope or limits

(*a*) These minimum standard requirements for precast concrete floor units, are to be used as a supplement to the ACI "Building Regulations for Reinforced Concrete" (ACI 318-41)†.

(*b*) With respect to design for strength, i.e., for bending moment, bond and shear stresses, all the types referred to in 10 (d) are to be designed in accord with standard reinforced concrete theory and in accord with (ACI 318-41), except that with respect to cover, there is in some cases departure therefrom justified by the greater refinement in the finished product, both as to dimensions and to quality, when made by factory methods and with factory control. See Section 11.

(*c*) Section 103(*a*), ACI 318-41‡, recognizes and makes provision for special systems of reinforced concrete. With reference to precast floors its provisions may be invoked where necessary by the manufacturer, architect, engineer, building inspector, builder or owner, wherever this report is silent.

*The Committee's report as published ACI JOURNAL, January 1946, *Proc.* V. 42, p. 245 and as editorially revised, was adopted as an ACI Standard by the 1946 ACI Convention and convention action ratified by Letter Ballot of ACI Members August 1946.

†Reference is hereafter referred to as ACI 318-41.

‡See appendix hereto.

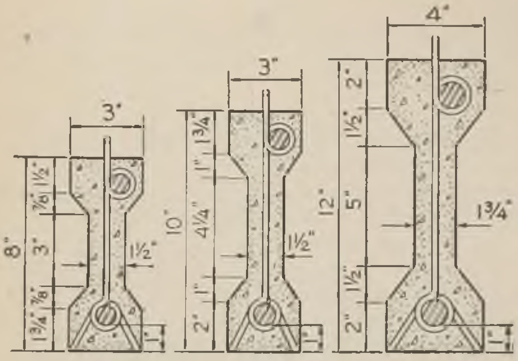
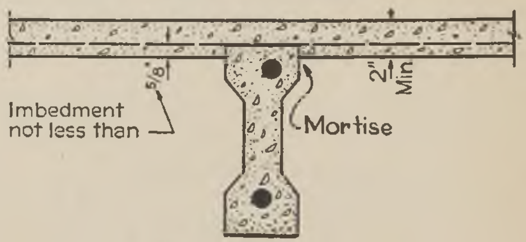



Fig. 1

Fig. 2 — Note: The mortise shall be not less than ½ in.




(*d*) Two distinct types are now manufactured:

1. I-beam type, with either cast in place or precast slab. (Fig. 1)

See section 50 for definitions. When the slab is cast in place as shown in Fig. 2 the result is a Tee beam and may be computed as such.

2. Hollow core type (Fig. 5)

Others are of channel shaped cross section often used as roof slabs, and the orthodox rectangular beam. Standard reinforced concrete theory is applicable to all.

### 11. Concrete protection for reinforcement

(*a*) Precast floor and roof units made of high quality factory controlled concrete may, when used in locations protected from the weather or moisture and with minimum fire hazards, be approved with ⅝-in. concrete cover for the reinforcing provided, however, that the concrete cover in all cases shall be at least equal to the diameter of round bars and one and one-half times the side dimension of square bars, and provided that to insure exact final location to the steel, positive and rigid devices for that purpose are employed in the manufacturing process. When the precast-members are exposed to weather, moisture or fire hazard the protective cover shall be increased to conform with section 507, ACI 318-41.†

†See appendix hereto.

## MATERIALS

### 20. Cement

(*a*) High-early strength concrete as produced with Type III portland cement or with Type I portland cement and accelerated curing is recommended. Portland cement shall conform to the "Standard Specifications for Portland Cement" (A.S.T.M. Serial Designation C150-42) and shall be Type I or Type III.

### 21. Aggregate

(*a*) Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates" (ASTM Serial Designation: C33-44), provided, however, that aggregates which have been shown by test or actual service to produce concrete of the required strength, durability, water-tightness, fire-resistance, and wearing qualities may be used under Section 302(*a*) Method 2, ACI 318-41* where authorized by the Commissioner of Buildings.

(*b*) The maximum size of the aggregate for precast joist shall not be larger than one-third of the narrowest dimension between sides of the forms of the member in which the unit is cast nor larger than three-fourths of the minimum clear spacing between reinforcing bars and sides of the forms except that where the concrete is placed by means of high frequency vibration the maximum size of the aggregate shall not be larger than one-half the narrowest dimension between sides of the forms.

(*c*) Aggregate for floor slabs shall conform to Section 21 (a) and in addition the combined aggregate shall be so graded from fine to coarse that not less than one-half nor more than two-thirds by weight of the total, based on dry materials, is retained on the No. 4 standard sieve, except that these proportions do not necessarily apply to light weight aggregates. The maximum size shall not exceed one-third the thickness of the slab.

### 22. Steel

(*a*) In the unprestressed types, the steel in the joist or floor units shall be intermediate grades Billet-Steel Concrete Reinforcement Bars (ASTM Serial Designation. A15-39) or Rail Steel Concrete Reinforcement Bars (ASTM Serial Designation: A16-39) or cold drawn steel wire for concrete reinforcement (ASTM Serial Designation: A82-34).

(*b*) Prestressed steel may be used in any of the types mentioned in section (10(d). When used and where such theory is applicable, computations for stresses, moments and allowable loads shall be in accord with the theory outlined in "Prestressed Concrete, Design Principles and Reinforcing Units", by Herman Schorer† or in a more condensed

*See appendix hereto.
†ACI JOURNAL, June 1943; *Proceedings* V. 39.

article by the same author in "Reinforced Concrete No. 6" (Portland Cement Assn.).

**23. Strength of concrete**

(*a*) Concrete for floor units made of sand and gravel, crushed stone, slag or other heavy aggregate and of a span of 12 ft. or more shall have a compressive strength of not less than 3750 psi at 28 days when tested in accordance with the applicable current standards of the A.S.T.M.

(*b*) For roof slabs or for floor units made of light weight aggregate lower compressive strengths may be permitted where the unit stresses used in design for strength and bond will satisfy the requirements of paragraph 24(a) of these standards.

**24. Unit stresses in concrete and reinforcement**

(*a*) The allowable *design* stresses in the concrete *shall* conform to the requirements set forth in Section 305 (a) and Table 305(a), ACI 318-41.†

(*b*) The allowable stresses in the steel shall conform to the requirements set forth in Section 306(a) and 306(b) ACI 318-41†.

## MANUFACTURE

**30. Workmanship**

(*a*) The finished product shall be free of honeycomb or rock pockets. The mix, the gradation of the aggregate and the workability shall be such as to insure complete filling of the form and continuous intimate bond between the concrete and all steel. To assist in attaining the latter, vibration is recommended, but any method which will meet the stated requirements and with the strength as required in unit beams or slabs or in the finished floor, is acceptable.

(*b*) Handling and conveying before curing shall be reduced to a minimum. Machinery for this purpose should be so designed that the unit will not be subject to bending or shock which will produce incipient cracks, broken edges or corners.

**31. Curing**

(*a*) The minimum amount of curing of precast units shall consist in keeping the concrete moist for at least 7 days, if made of normal portland cement and for at least 3 days if made of high early strength cement. For each decrement of 5 degrees below 70 F in the average curing temperature these curing periods shall be increased by four days for units made of normal portland cement and by two days for units made of high early strength cement. See Table 1. The average curing temperature in no case shall be less than 50 F.

(*b*) Curing by high pressure steam, steam vapor, or other accepted processes may be employed to accelerate the hardening of the concrete and to reduce the time of curing provided, however, the compressive

†See appendix hereto.

**TABLE 1 (sec 31a) — MINIMUM CURING TIME IN DAYS IN MOIST ATMOSPHERE**

| Cement | Temperature in Degrees F | | | | |
|---|---|---|---|---|---|
| | 70 | 65 | 60 | 55 | 50 |
| Normal Portland......... | 7 | 11 | 15 | 19 | 23 |
| High Early Strength...... | 3 | 5 | 7 | 9 | 11 |

strength of the concrete is at least equal to that obtained with the curing specified in Section 31a and that the 28-day strength meets the requirements of Sec. 23.

### 32. Identification and marking

(*a*) All joist, beams, girders and other floor units shall show some mark plainly indicating the top of the unit and the size of the bending moment reinforcement. This mark or symbol shall indicate the length, size and type of reinforcing and carrying capacity of the unit, and shall be shown on the placing plans.

### 33. Transportation

(*a*) After curing, units shall be so stored, stacked, loaded and transported, unloaded and placed, that no transverse or longitudinal cracks will develop. Adequate instructions should be given to handlers and insofar as possible, only experienced men should be put in charge of this phase of the work.

(*b*) To insure the eventual placement of the units in the structure without cracks, the handling, whether manually or in slings or cradles, shall be done in such a manner that bending about either the vertical or horizontal axis of the cross-section will be reduced to a minimum.

## TESTS

### 40. Beams and floors

(*a*) Where the individual unit is tested as a simple beam, it shall sustain without complete failure*, a uniformly distributed load of at least 2.25 times the design live load based on allowable stresses in bending moment and shear as given in Section and Table 305(*a*) and Section 306(*a*), ACI 318-41†

(*b*) When field tests are made they shall be made as required by and shall meet the requirements of Section 202, ACI 318-41†) making use of notation used in Section 200, ACI 318-41.†

*Failure in Section 40 is defined as, any behavior of the beam under load which indicates that the yield point of the steel has been exceeded, or that cracking of the concrete is such that it would not be permitted in a structure in regular service.

†See appendix hereto.

## I-BEAM TYPE JOISTS

### 50. Definitions

(*a*) Floors made of precast joists mortised or embedded into a monolithic floor placed or poured on the job, producing a T-beam are called *Precast joist cast-in-place concrete slab floors*, and the resisting moment may be computed as if the joist and slab form a T-beam. See Section 54.

(*b*) Floors made of precast joists over which precast slabs are laid and bonded to produce T-beam action are called *precast joist* and *slab* concrete floors.

(*c*) Floors made of precast joists over which precast slabs are laid for flooring and *not* bonded to the joist to produce T-beam action are called *independent precast joist* and *slab* concrete floors.

### 51. Sections

(*a*) The most commonly made joists of I-beam section are as shown in Fig. 1. Other sizes and shapes meeting the regulations of ACI 318-41 as to resistance to bending moment, shear, deflection and bearing, may be used.

(*b*) Since the shear and bending moment resistances are based on nominal dimensions as well as on area of the steel and allowable working stresses in concrete and steel, the following tolerances shall not be exceeded: plus or minus ⅛-in. as to width and height, and plus or minus ½ in. as to length.

### 52. Floor slab thickness

(*a*) The recommended minimum thickness of *cast-in-place* reinforced concrete floor slabs with joist heads embedded not less than ½ in. and with joist spacing less than 30 in., is 2 in. For joist spacing of 30 to 36 in. the minimum thickness of *slab* concrete floors should be 2½ in. Greater thickness may be required where unusual loads or spans are encountered. The required thickness of slabs spanning more than 36 in. shall be determined by accepted design methods.

(*b*) The recommended minimum thickness of precast slab to be used with precast joists is 2 in. with joist spacing up to 30 in. and 2½ in. with joist spacing from 30 in. to 36 in. In the case of slabs of ribbed- or channel-section, the thickness requirement applies to the portion thereof containing the tensile reinforcement.

### 53. Extra or concentrated loads

(*a*) Where the floor supports partition walls parallel to the joist or where loads heavier than the uniform load for which the floor is designed are known to be expected, joists may be placed side by side, with flanges touching, but under such conditions the joists and cast-in-place floor slabs are not to be considered as a T-beam unless the shear reinforcing loops in the joists extend into the slab.

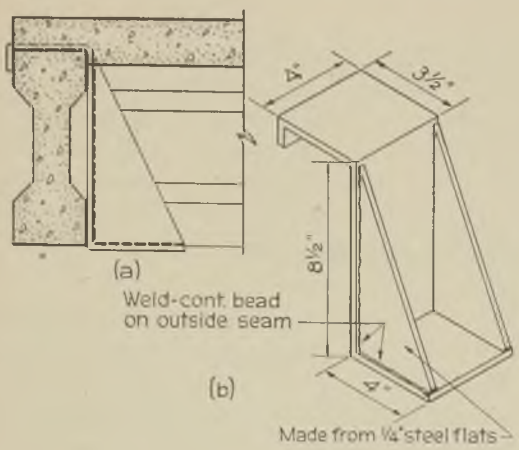



**Fig. 3—Details of joist hangers**

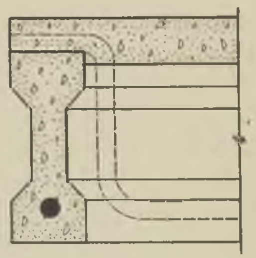


**Fig. 4—Tension bar hanger inserted in joist as cast**

(*b*) Where multiple joists are used, their strength shall be at least that of a single beam, multiplied by the number used.

**54. Design**

(*a*) Floors laid as defined in 50(*a*) and 50(*b*) of these standards may be designed as T-beams, where joists are supported at sufficient intervals to take out the sag while the cast-in-place or precast slab is being laid, the support being left in place until the concrete has hardened. Under this condition the dead load is considered to be the weight of the floor per joist plus the weight of the joist.*

(*b*) The resistance to longitudinal shear between floor and joist where the joists are embedded ½ in. may be taken as equal to the allowable shear stress for beams with no web reinforcement, but with special anchorage of longitudinal steel. Table 305(*a*), ACI 318-41†.

(*c*) Where ends of joists cannot be rested on walls, as at stair wells, etc., metal joist hangers made as in Fig. 3 may be used to provide end support, or a preformed tension bar hanger may be inserted in the joist at the time of casting (Fig. 4).

*For further details as to design for Type 1 joists, the Portland Cement Association pabmphlet "How to Design and Build Precast Joist Concrete Floors" is at present the most complete treatise.

†See appendix hereto.

**55. Holes in web**

(*a*) Because they reduce the shearing resistance, holes in the web shall be reduced to a minimum. Where found necessary they should be located as near the center of the beam as possible or at location of minimum shear. They shall be cast when the beam is made or drilled on the job (not punched) and be not more than 2 in. in diameter. No holes should be made by any mechanic on a job except after approval by and under the supervision of the architect or engineer.

**56. Installation and construction details**

(*a*) On every job there will be a need for a joist setting plan; prepared by an architect or an engineer and approved by the manufacturer. Only in this way will the owner be assured of unquestionable results. Working stresses based on maximum strength of the materials used, shall be as provided in Section 305(*a*) and Table 305(*a*), ACI 318-41*, and shall be given the architect or engineer by the manufacturer. Shears and bending moments will be properly taken into account by the architect or engineer and accepted by the manufacturer.†

(*b*) There is a need for standardization of many installation and construction details. This does not mean that innovations should be prohibited or frowned upon, but rather that an acceptable practice in handling and setting, leveling, shoring of joists, placing forms for floors, reinforcement for floors, conduits, bulkheads, stairwells, partition bearing joist, etc., should be approved by the architect, engineer and manufacturer and the building contractor informed thereof by properly drawn plans and through the supervision of the architect or engineer.

## HOLLOW CORE TYPE JOISTS

**60. Definitions**

(*a*) Floors made of precast concrete units in which some portion of the cross section between top, bottom, and sides is left out at the time of casting for purposes of reducing dead load and quantity of material used in their manufacture are called Hollow-Core-Precast Floors.

**61. Sections**

(*a*) Sections of hollow units as shown in Fig. 5 are acceptable.

(*b*) Any other sections which will provide proper protection for the steel reinforcement, and strength sufficient for handling and for carrying the loads for which they are designed and which meet all other requirements of this report will be acceptable, provided the computations for carrying capacity have been verified by a recognized testing laboratory.

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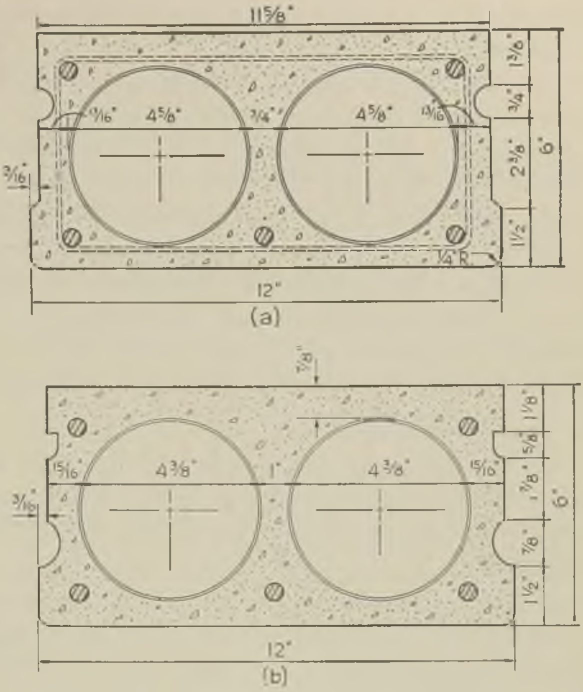
*See appendix hereto.

†The manufacturer's supervision over the placing of the reinforcement and over the factory treatment of the joist from planning the mixture to delivery on the job, will be stricter, where he is provided with data on the maximum shear and bending moment stresses as computed by the architect or engineer.

Fig. 5




**62. Design**

(*a*) Resisting moments and shear resistance of hollow core units shall be computed by the standard formulas and methods. Allowable unit stresses in concrete shall conform to the requirements of Section 305(a) and Table 305(a) ACI 318-41.*

(*b*) The allowable stresses in the steel shall conform to the requirements set forth in Section 306(a) and 306(b) ACI 318-41.*

**63. Special conditions—openings known in advance of construction**

(*a*) At stairways or other openings when no wall or girder bearing is available for one end of a floor unit, the long dimension of the opening shall be parallel to the length of the unit. Specially designed reinforced headers or curbs at the short side of such openings shall transfer their dead and live load to the longitudinal units on the side of the opening by devices or means satisfactory to the architect or engineer. Units adjacent to the side of such openings shall be designed to carry the reaction of the headers in addition to their own specified dead and live load.

(*b*) The requirements of unusual conditions, such as those outlined in 63(*a*) or others, often may be met by making use of special units,

*See appendix hereto.

some of which are wider than the standard, some deeper and some more heavily reinforced.

(*c*) Holes in the bottom or ceiling sides of units, for conduit or for hangers should be located below the hollow portion of the unit and should be cast at the time of manufacture or drilled under the supervision of the architect or engineer.

(*d*) Channeling in the top or floor side, except over the support is not permissible and channels placed over the supports shall not reduce the shear resistance below that allowable in this standard.

(*e*) Cutting of reinforcement for installation of pipes or conduit is permitted only upon approval of the architect or engineer and only after satisfying requirements specified in Section 64(*a*) (*b*) and (*c*).

**64. Cutting of holes and channels**

(*a*) No openings or channels not provided for in the structural design shall be made on the job without the specific approval of the engineer and in accord with his written, detailed instructions covering such work.

(*b*) In some cases the section of and reinforcement in the adjacent beam are such that when the span is taken into consideration the resistance to bending moment and shear is greater than that required by the live and dead loads called for by the building code or specifications. In such a case holes may be cut and curbed providing it is done in a manner to insure that the stresses on the transversely cut units will be transferred through the curb or longitudinal key to the adjoining units. In general, such cutting should be located near the quarter point of the span.

(*c*) Where holes are cut, the load normally carried by the cut units, and by the cutting transferred laterally to adjacent units may be considered to be uniformly distributed laterally for three units one foot wide on either side. With such assumption the computed stresses in the concrete may not esceed $0.45\,f'_c$ nor in any case 1500 psi compression; $0.02f'_c$ (for reinforcement without special anchorage) with a maximum allowable unit stress of 75 psi or $0.03\ f'_c$ (for reinforcement with special anchorage) with a maximum allowable unit stress of 112.5 psi in shear for units without stirrups; or 20,000 psi tension in the reinforcing steel.

(*d*) Holes in the bottom or ceiling side for conduit or for hangers should be below the hollow portion of the unit; should not be less than 1¾ in. from the longitudinal reinforcement, and may be either cast at the time of manufacture or drilled under the supervision of the architect or engineer. Hangers may be placed in the joints between the units before they are grouted.

(*e*) There shall be no channeling in the top or floor side except over the support. Channels cut over the supports are permissable only when they do not reduce the shear resistance below that allowable in this standard, and must be approved by the architect or engineer.

(*f*) Cutting of reinforcement for installation of pipes or conduit is permissible only with approval of the architect or engineer and only after satisfying requirements specified in Section 64 (*a*) (*b*) and (*c*).

**65. Installation and construction details**

(*a*) An erection or unit setting plan shall be prepared by the architect or engineer and approved by the manufacturer for each job. To provide properly for shear and bending moment stresses, the plan shall indicate all openings, stairways, etc., together with the location of points, if any, where loads are in excess of the general floor loading.

## APPENDIX

(Excerpts from Standard Building Regulations for Reinforced Concrete—ACI 318-41)

**103. Special systems of reinforced concrete**

(*a*) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with, or not covered by these regulations shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction" appointed by the Commissioner of Buildings. This Board shall be composed of competent engineers, architects and builders, and shall have the authority to investigate the data so submitted and to formulate rules governing the design and construction of such systems. These rules when approved by the Commissioner of Buildings shall be of the same force and effect as the provisions of this code.

**200. Notation**

$D$ = Deflection of a floor member under load test.
$L$ = Span of member under load test.
$t$ = The total thickness or depth of a member under load test.

**202. Load Tests**

(*a*) When a load test is required, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or, where lawful, a lower rating shall be established. The structure shall be considered to have passed the test if the maximum deflection at the end of the twenty-four hour period does not exceed the value of $D$ as given in the following:

$$D = \frac{.001\,L^2}{12\,t} \qquad (1)$$

all terms expressed in the same units.

If the deflection exceeds the value of $D$ as given in formula (1), the construction shall be considered to have passed the test if within twenty-four hours after the removal of the load the member or portion of the structure shows a recovery of at least seventy-five percent of the observed deflection.

### 302. Determination of strength-quality of materials

(*a*) The determination of the proportions of cement, aggregate and water to attain the required strengths shall be made by one of the following methods:

*Method 1—Concrete made from average materials:*

When no preliminary tests of the materials to be used are made, the water-content per sack of cement shall not exceed the values in Table 302(*a*). Method 2 shall be employed when artificial aggregates or admixtures are used.

**TABLE 302(a)—ASSUMED STRENGTH OF CONCRETE MIXTURES**

| Water-Content U. S. Gallons Per 94-lb. Sack of Cement | Assumed Compressive Strength at 28 Days—psi |
|---|---|
| 7½ | 2000 |
| 6¾ | 2500 |
| 6 | 3000 |
| 5 | 3750 |

NOTE—In interpreting this table, surface water carried by the aggregate must be included as part of the mixing water in computing the water-content.

*Method 2—Controlled Concrete:*

Water-content other than shown in Table 302(*a*) may be used provided that the strength-quality of the concrete proposed for use in the structure shall be established by tests which shall be made in advance of the beginning of operations, using the consistencies suitable for the work and in accordance with the "Standard Method of Making Compression Tests of Concrete" (A.S.T.M. Serial Designation: C39-39). A curve representing the relation between the water-content and the average 28-day compressive strength or earlier strength at which the concrete is to receive its full working load, shall be established for a range of values including all the compressive strengths called for on the plans.

The curve shall be established by at least three points, each point representing average values from at least four test specimens. The maximum allowable water-content for the concrete for the structure shall be as determined from this curve and shall correspond to a strength which is fifteen percent greater than that called for on the plans. No substitutions shall be made in the materials used on the work without additional tests in accordance herewith to show that the quality of the concrete is satisfactory.

### 305. Allowable unit stresses in concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the values of Table 305(*a*) where $f'_c$ equals the minimum specified ultimate compressive strength at 28 days, or at the earlier age at which the concrete may be expected to receive its full load.

### 306. Allowable unit stresses in reinforcement

Unless otherwise provided in these Regulations, steel for concrete reinforcement shall not be stressed in excess of the following limits:

(*a*) *Tension*

($f_s$ = Tensile unit stress in longitudinal reinforcement)
and ($f_v$ = Tensile unit stress in web reinforcement)

20,000 psi for Rail-Steel Concrete Reinforcement Bars, Billet-Steel Concrete Reinforcement Bars (of intermediate and hard grades), Axle-Steel Concrete Reinforcement Bars (of intermediate and hard grades), and Cold-Drawn Steel Wire for Concrete Reinforcement.

18,000 psi for Billet-Steel Concrete Reinforcement Bars (of structural grade), and Axle-Steel Concrete Reinforcement Bars (of structural grade).

(*b*) *Tension in One-Way Slabs of Not More Than 12 Feet Span*

($f_s$ = Tensile unit stress in main reinforcement).

For the main reinforcement, ⅜ inch or less in diameter, in one-way slabs, 50 per cent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i.

## TABLE 305(a)—ALLOWABLE UNIT STRESSES IN CONCRETE

| Description | | Allowable Unit Stresses | | | | |
|---|---|---|---|---|---|---|
| | | For Any Strength of Concrete as Fixed by Test in Accordance with Section 302 $n = \frac{30000}{f'_c}$ | When Strength of Concrete is Fixed by the Water-Content in Accordance with Section 302 | | | |
| | | | $f'_c =$ 2000 psi $n = 15$ | $f'_c =$ 2500 psi $n = 12$ | $f'_c =$ 3000 psi $n = 10$ | $f'_c =$ 3750 psi $n = 8$ |
| **Flexure:** $f_c$ | | | | | | |
| Extreme fiber stress in compression | $f_c$ | $0.45f'_c$ | 900 | 1125 | 1350 | 1688 |
| **Shear:** $v$ | | | | | | |
| Beams with no web reinforcement and without special anchorage of longitudinal steel | $v_c$ | $0.02f'_c$ | 40 | 50 | 60 | 75 |
| Beams with no web reinforcement but with special anchorage of longitudinal steel | $v_c$ | $0.03f'_c$ | 60 | 75 | 90 | 113 |
| Beams with properly designed web reinforcement but without special anchorage of longitudinal steel | $v$ | $0.06f'_c$ | 120 | 150 | 180 | 225 |
| Beams with properly designed web reinforcement and with special anchorage of longitudinal steel | $v$ | $0.12f'_c$ | 240 | 300 | 360 | 450 |
| *Flat slabs at distance $d$ from edge of column capital or drop panel | $v_c$ | $0.03f'_c$ | 60 | 75 | 90 | 113 |
| **Footings | $v_c$ | $0.03f'_c$ but not to exceed 75 psi | 60 | 75 | 75 | 75 |
| ‡**Bond:** $u$ | | | | | | |
| In beams and slabs and one-way footings: | | | | | | |
| Plain bars | $u$ | $0.04f'_c$ but not to exceed 160 psi | 80 | 100 | 120 | 150 |
| Deformed bars | $u$ | $0.05f'_c$ but not to exceed 200 psi | 100 | 125 | 150 | 188 |
| In two-way footings: | | | | | | |
| Plain bars (hooked) | $u$ | $0.045f'_c$ but not to exceed 160 psi | 90 | 113 | 135 | 160 |
| Deformed bars (hooked) | $u$ | $0.056f'_c$ but not to exceed 200 psi | 112 | 140 | 168 | 200 |
| **Bearing:** $f_c$ | | | | | | |
| On full area | $f_c$ | $0.25f'_c$ | 500 | 625 | 750 | 938 |
| On one-third area or less† | $f_c$ | $0.375f'_c$ | 750 | 938 | 1125 | 1405 |

*See Section 807. **See Section 905 (*a*) and 808(*a*).

†The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

‡Where special anchorage is provided (see Section 903 (*a*) ), one and one-half times these values in bond may be used in beams, slabs and one-way footings, but in no case to exceed 200 psi for plain bars and 250 psi for deformed bars. The values given for two-way footings include an allowance for special anchorage.

### 507. Concrete protection for reinforcement

(*a*) The reinforcement of footings and other principal structural members in which the concrete is deposited against the ground shall have not less than three inches of concrete between it and the ground contact surface. If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than two inches of concrete for bars more than ⅝ inch in diameter and one and one-half inches for bars ⅝ inch or less in diameter.

(*b*) The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or weather shall be not less than three-fourths inch for slabs and walls; and not less than one and one-half inches for beams, girders and columns. In concrete joist floors in which the clear distance between joists is not more than thirty inches, the protection of metal reinforcement shall be at least three-fourths inch.

(*c*) If the code of which these regulations form a part specifies, as fire-protective covering of the reinforcement, thicknesses of concrete greater than those given in this section, then such greater thicknesses shall be used.

(*d*) Concrete protection for reinforcement shall in all cases be at least equal to the diameter of round bars, and one and one-half times the side dimension of square bars.

(*e*) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion by concrete or other adequate covering.

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## ACI Standard

# Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)*

REPORTED BY ACI COMMITTEE 714

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## 1. SCOPE

These recommendations describe practice for use in the design and construction of concrete silos, stave, block and monolithic, for the storage of grass or corn silage.

## 2. GENERAL DESIGN RECOMMENDATIONS

### A. Foundations

The foundation should safely support the silo and might well be one of the four following types: (1) a footing only, (2) a footing and foundation wall, (3) a battered foundation wall providing a spread footing, or (4) a foundation wall flared at the bottom on both sides, the depth of flare being at least twice the projection of the flare. When foundation walls of types 2, 3, and 4, are used, they should be at least 8 in. thick, extend at least six inches above the ground line, and should be reinforced to withstand the lateral pressure of the silage.

### B. Footings

*1) Depth below ground line:* The distance from the ground line to the base of the footing should be at least 24 in. and not less than 36 in. in localities in which the ground freezes to a depth of two feet or more.

*This report from ACI JOURNAL, January 1944; *Proc.*, V. 40, p. 189, as revised ACI JOURNAL, January, 1946; *Proc.* V. 42, p. 261, was adopted as an ACI Standard at the ACI Convention 1946 subject to editorial revisions presented on a Letter Ballot, canvassed August 1946 by which the convention action was ratified

*2) Width and depth of footings:* The footing should be sufficient to carry the weight and friction load of the silo. The width and depth, therefore, depend upon the height of the silo and the load carrying capacity of the soil. Recommended footing sizes are given in Tables 1 and 2 for three different types of soil. See Appendix-B.

*3) Future Increases in height of silo:* The footings should be designed to take care of any contemplated future increase in the height of the silo.

### C. Hoop Spacing

The hoop spacing for either grass or corn silage of moisture content not over 75 percent should not exceed that given in Table 3. See Appendix-A.

### D. Floors

A concrete floor may be provided for the purpose of facilitating drainage. If provided, it should be at least four inches thick, should slope toward the drain and be constructed to permit free movement relative to the walls.

### E. Drains

The Committee recognizes the desirability of drains but does not feel justified in making recommendations until research now under way is concluded. Where a drain is provided, provision should also be made to carry liquids away from the site.

### F. Roofs

Whether or not the silo is roofed should be optional with the purchaser. If a roof is provided, it is recommended that it have a permanent opening not less than one square foot for ventilation.

**TABLE 1. DIMENSIONS OF ANNULAR FOOTINGS FOR SILOS WITH WALLS 2.5 INCHES THICK**

| Height of Silo ft. | TYPE OF SOIL | | | | | |
|---|---|---|---|---|---|---|
| | Sand and Gravel (Type 1) | | Firm clay, wet sand or clay and sand mixture (Type 2) | | Soft Clay (Type 3) | |
| | Width Inches | Depth Inches | Width Inches | Depth Inches | Width Inches | Depth Inches |
| 20 | 12 | 8 | 12 | 8 | 12 | 8 |
| 25 | 12 | 8 | 12 | 8 | 18 | 8 |
| 30 | 12 | 8 | 12 | 8 | 24 | 8 |
| 35 | 12 | 8 | 16 | 8 | 32 | 11 |
| 40 | 12 | 8 | 21 | 10 | Silos Higher Than 35 ft. Not Recommended on Type 3 Soil | |
| 45 | 13 | 9 | 26 | 13 | | |
| 50 | 16 | 11 | 32 | 15 | | |
| 55 | 19 | 13 | Silos Higher Than 50 ft Not Recommended on Type 2 Soil | | | |
| 60 | 23 | 16 | | | | |

See Appendix - B.

TABLE 2. DIMENSIONS OF ANNULAR FOOTINGS FOR SILOS WITH WALLS 6 INCHES THICK

| Height of Silo ft. | TYPE OF SOIL | | | | | |
|---|---|---|---|---|---|---|
| | Sand and Gravel (Type 1) | | Firm clay, wet sand or clay and sand mixture (Type 2) | | Soft Clay (Type 3) | |
| | Width Inches | Depth Inches | Width Inches | Depth Inches | Width Inches | Depth Inches |
| 20 | 12 | 8 | 12 | 8 | 17 | 8 |
| 25 | 12 | 8 | 12 | 8 | 24 | 8 |
| 30 | 12 | 8 | 16 | 8 | 32 | 11 |
| 35 | 12 | 8 | 21 | 10 | Silos Higher Than 30 ft. Not Recommended on Type 3 Soil | |
| 40 | 13 | 9 | 26 | 13 | | |
| 45 | 16 | 11 | 32 | 15 | | |
| 50 | 19 | 13 | Silos Higher Than 45 ft. Not Recommended on Type 2 Soil | | | |
| 55 | 23 | 16 | | | | |
| 60 | 27 | 18 | | | | |

See Appendix - B.

If a concrete roof is used, the design and construction should be such that no radial thrust is applied to the silo walls. The minimum concrete cover for reinforcement should be one inch.

### G. Chutes

The type of chute, if used, should be optional with the purchaser. Ventilation of the barn through the chute should be avoided.

## 3. MATERIALS

### A. Aggregates

Aggregates for use in the construction of silos should conform to the "Standard Specifications for Concrete and Reinforced Concrete" of the 1940 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete. The total of deleterious materials of an expansive nature such as shale, absorptive chert, etc., should not exceed 1 percent by weight in aggregate coarser than the No. 4 sieve nor more than 2 percent by weight in aggregate finer than the No. 4 sieve. For aggregates for interior finishes and mortar for laying blocks see Sections 3-F and 5-F respectively.

### B. Cement

Cement for use in the construction of silos should conform to the current specifications for Portland Cement of the American Society for Testing Materials.

### C. Steel

Metal reinforcement for use in the construction of silos should conform to Building Regulations for Reinforced Concrete (ACI 318-41) or "Standard Specifications for Concrete and Reinforced Concrete" of

## TABLE 3—HOOP SPACING FOR CONCRETE STAVE SILOS DESIGNED FOR SILAGE WITH A MOISTURE CONTENT NOT EXCEEDING 75 PERCENT

Based upon round rods of the sizes indicated of intermediate grade steel, with rolled threads. If cut threads are used the ends must be upset, or the spacing be closer. Unit tensile stress 20,000 psi (c)

| Distance from top of silo in feet | Lateral Pressure of silo in psf (a) (c) | Hoop Spacing in Inches for 9/16 Inch Diameter Rods — Silo Diameter in feet | | | | | | Hoop Spacing in Inches for ½ or 9/16 Inch Diameter Rods — Silo Diameter in feet | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 10 | 12 | 14 | 16 | 18 | 20 | 10 | 12 | 14 | 16 | 18 | 20 |
| 2½ | 12 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 |
| 5 | 33 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 |
| 7½ | 60 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 |
| 10 | 91 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 |
| 12½ | 125 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 |
| 15 | 162 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 |
| 17½ | 203 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 15 | 15 |
| 20 | 246 | 30 | 30 | 30 | 30 | 15 | 15 | 30 | 30 | 15 | 15 | 15 | 15 |
| 22½ | 294 | 30 | 30 | 30 | 15 | 15 | 15 | 30 | 15 | 15 | 15 | 15 | 15 |
| 25 | 340 | 30 | 30 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
| 27½ | 388 | 30 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
| 30 | 440 | 15 | 15 | 15 | 15 | 15 | 10 | 15 | 15 | 15 | 15 | 15 | 10 |
| 32½ | 495 | 15 | 15 | 15 | 15 | 10 | 10 | 15 | 15 | 15 | 15 | 10 | 10 |
| 35 | 550 | 15 | 15 | 15 | 10 | 10 | 10 | 15 | 15 | 15 | 10 | 10 | 10 |
| 37½ | 609 | 15 | 15 | 10 | 10 | 10 | 10 | 15 | 15 | 10 | 10 | 10 | 10 |
| 40 | 668 | 15 | 15 | 10 | 10 | 10 | 7½ | 15 | 15 | 10 | 10 | 10 | 7½ |
| 42½ | 725 | 15 | 10 | 10 | 10 | 7½ | 7½ | 15 | 10 | 10 | 10 | 7½ | 7½ |
| 45 | 793 | 15 | 10 | 10 | 7½ | 7½ | 7½ | 15 | 10 | 10 | 7½ | 7½ | 7½ |
| 47½ | 855 | 10 | 10 | 10 | 7½ | 7½ | 6 | 10 | 10 | 10 | 7½ | 7½ | 6 |
| 50 | 925 | 10 | 10 | 7½ | 7½ | 7½ | 6 | 10 | 10 | 7½ | 7½ | 7½ | 6 |
| 52½ | 990 | 10 | 10 | 7½ | 7½ | 6 | 6 | 10 | 10 | 7½ | 7½ | 6 | 6 |
| 55 | 1055 | 10 | 7½ | 7½ | 6 | 6 | 6 | 10 | 7½ | 7½ | 6 | 6 | 6 |
| 57½ | 1110 | 10 | 7½ | 7½ | 6 | 6 | 5 | 10 | 7½ | 7½ | 6 | 6 | 5 |
| 60 | 1200 | 7½ | 7½ | 6 | 6 | 5 | 5 | 7½ | 7½ | 6 | 6 | 5 | 5 |

(a) See Appendix - A.
(b) For spacings above zig-zag line use ½ inch rods. For spacings below zig-zag line use 9/16 inch rods.
(c) psi denotes pounds per sq. in., psf denotes pounds per sq. ft.

the 1940 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.

**D. Concrete**

*1) Proportioning, mixing, curing and testing concrete:* Proportioning mixing, curing and testing concrete should be in conformance with the provisions relating thereto of the 1940 "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete" of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.

*2) Concrete quality:* It is recommended to ensure a high degree of imperviousness and durability, that the concretes used in various parts of the silo should have compressive strength not less than that given in the following table:

| Location | Minimum Recommended Compressive Strength at 28 days psi |
|---|---|
| Footings | 3,000 |
| Foundation Walls | 3,500 |
| Silo Wall, Roof, Door Frames & Chute | 4,500 |

**E. Hoops**

*1) Effective area:* The effective area of threaded rods for stress calculation should be based on the diameter at the root of threads when cut threads are used and should be based on the diameter of the rod when rolled threads are used.

*2) Design unit stress:* The unit stress for design should be 18,000 psi for structural, 20,000 psi for intermediate grade steel bars and not more than 50 percent of the minimum value for the yield point given in the specification governing any special steels that may be used.

*3) Hoop-Spacing.* The maximum hoop-spacing for either grass or corn silage should be as given in Table 3. This table does not apply for silage with a moisture content above 75 percent, as closer spacing will be required for wetter silage.

*4) Lugs:* Lugs (connections for hoops of round cross-section) should be of malleable cast iron or of steel. The strength of the lug should be such that the ultimate strength of the hooping to be used with it can be developed by the lug in place. For silo diameters up to and including ten feet, at least two lugs per hoop should be used; for diameters from 10 to and including 16 ft., at least three lugs per hoop should be used; and for diameters from 16 to and including 22 ft., at least four lugs per hoop should be used.

*5) Connections:* Connections for flat hoops should develop in place twice the design load of the hoops. The number of connections per hoop should be not less than the number of lugs per hoop recommended in the paragraph next above.

6) *Tightening hoops:* The hoops should be uniformly tightened to about 50 percent of the design stress, the joints filled, the fillings allowed to harden, and the hoops again tightened, this time to full stress. Before the silo is filled the hoops on the lower two-thirds of the silo preferably should be retightened.

7) *Spreaders:* Spreaders should develop in place twice the design load of the hoops connected to them.

#### F. Interior finishes

Concrete stave silos should, and other concrete silos may, be plastered or given a cement wash inside in order to make them more nearly impervious. Materials for this should be prepared and applied as here recommended.

1) *Cement plaster:* The grading of the sand should be within the following limits: All material passing No. 4 sieve; from 15 to 35 percent retained on No. 14 sieve, and from 65 to 85 percent retained on No. 48 sieve.

Fine sand should be added to natural sand that is deficient in fines. The sand used in plaster coat should have not more than ½ percent of shale and other expansive material by weight. The mix should be 1 part of portland cement to 1½ parts of sand by weight. Retempering of the mix should not be permitted.

The wall should be wet down prior to plastering in such a manner that a proper and uniform suction is obtained. Immediately prior to applying the plaster, a thin cement grout should be well brushed into the wall surface. The plaster coat should vary uniformly from an average thickness of ¼ inch at the top to ½ inch at the bottom. It should be protected from abrasion and too rapid drying for at least three days.

2) *Cement wash:* Cement wash should be prepared by mixing portland cement with water to give a creamy consistency. The cement should be screened into the water to prevent lumping. The wash should be applied to clean and dampened walls. It should be protected from abrasion and too rapid drying for at least three days.

### 4. STAVE SILOS

#### A. Quality of staves

Staves used in silos should conform to the following Specifications:

1) *Flexural strength:* The average flexural strength of five staves at the time of delivery shall be not less than 690* psi and the flexural strength of an individual stave shall be not less than 660 psi.

2) *Absorption:* The absorption after submersion in water for 24 hours shall not exceed 6 percent by weight.

3) *Minimum thickness:* No stave shall have a thickness at any section less than 2.0 inches.

*This represents 120 lb. per inch of width on a 2.5-in. thickness of plane-faced stave.

4) *Marking:* All silos shall bear a distinctive mark of the manufacturer of the staves used in its construction.

5) *Inspection:* Proper facilities shall be provided the purchaser for sampling and inspection either at the factory or at the site of work, whichever may be specified in the contract. At least ten days from the time of sampling shall be allowed for the completion of the tests. All tests shall be made in accordance with the methods specified in sections 12 and 13.

6) *Expense of tests:* The expense of inspection and testing shall be borne by the purchaser, unless otherwise specified in the contract.

7) *Selection of staves for test:* Staves for testing shall be selected by the purchaser or by a competent representative authorized by him to do this work. Such staves shall be representative of the lot of staves from which they are selected. Full size staves shall be used in all cases.

8) *Number of staves per sample:* The sample for testing shall consist of five full size staves.

9) *Marking:* All staves selected for testing shall be permanently identified before shipment to the laboratory.

10) *Moisture content at time of test:* The staves shall be submerged in water for 24 hours prior to testing. They shall be loaded immediately after removal from submersion. Surface water may be wiped off with a damp cloth.

11) *Measure of thickness:* The thickness of plane-faced staves shall be taken as the average of five measurements made with a thickness gage reading to 1/1000 in. These measurements shall be taken along the short axis (or at the break) of the stave at one inch from each edge, at the quarter points, and at the center point. This average shall be recorded to the nearest 1/100 in.

12) *Width:* The width of plane-faced staves to be used in calculating the flexural strength shall be the laid-up width. This width shall be taken as equal to one fourth the distance measured between corresponding points on the first and fifth staves of five staves fitted together on a plane surface, so as to obtain the minimum over-all width.

13) *Position:* The stave shall be supported on a 24-in. span and loaded at midspan. The surface of the stave intended to be on the exterior of the silo shall be in tension under the test load. The end supports shall be free to rotate in directions parallel and perpendicular to the long axis of the stave and shall be sufficiently stiff so that the load is substantially uniformly distributed along their length. Padded bearing plates 2 in. wide shall be used at the supports and at the central knife-edge. The set-up shall be such that no torsional moments are developed. Special bearing blocks shall be used when testing non-planer staves.

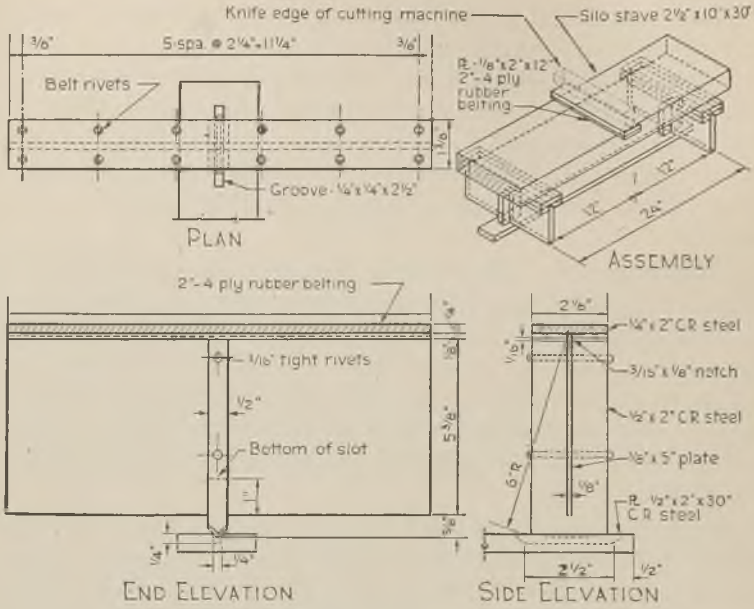



Fig. 1—Apparatus for testing silo staves

NOTE:—Fig. 1 shows the details of an apparatus for testing silo staves, which comlies with this specification.

*14) Speed of loading:* The speed of the moving head of the testing machine shall not be more than 0.05 in. per minute.

*15) Flexural strength:* The flexural strength of plane-faced staves shall be determined by the following calculation:

$$\text{Flexural strength in psi} = \frac{36W}{bd^2}$$

The flexural strength of staves with non-planer faces shall be determined by the following formula:

$$\text{Flexural strength in psi} = \frac{6Wc}{I}$$

in which $W$ = the maximum load in pounds
$b$ = the width of plane-faced staves
$d$ = the thickness of plane-faced staves
$c$ = the distance from neutral axis to extreme tensile fibre in inches
$I$ = the moment of inertia of a section normal to the long axis of the stave in inches to the fourth power.

*16) Absorption samples:* The sample for the absorption test shall consist of one of the two pieces (approximately a half-stave) of each of the five staves which result from the flexure test.

17) *Marking:* Each piece shall be marked so that it may be identified at any time with the stave from which it was taken. The marking shall not cover more than 5 percent of the area of one face of the half-stave.

18) *Drying:* The pieces shall be dried in an oven at a temperature between 100 and 110 C. (212 and 230 F.) and weighed at 24-hour intervals until the loss in weight does not exceed 0.2 percent of the previous weight.

19) *Accuracy of balance:* The balance used shall be sensitive to within 0.05 percent of the weight of the smallest piece tested.

20) *Immersion:* The dry weights of the pieces shall be obtained after which they shall be immersed in water at room temperature (60 to 80 F.), for 24 hours. They shall be removed from the water and allowed to drain for 60 seconds by placing on 3/8 in. or coarser wire mesh, visible surface water being removed with a damp cloth, and immediately weighed.

21) *Percent absorption:* The absorption is the difference between the wet and dry weight of the sample divided by the dry weight and multiplied by 100.

#### B. Joints

The joints of stave silos should be closed by pointing with cement paste or mortar, by cement washing or by plastering; or, if special joints are used, by pouring. One of these methods or a combination of these methods may be used.

## 5. BLOCK SILOS

#### A. General

Block silos are those built of precast concrete units other than staves.

#### B. Quality of blocks

Hollow blocks should conform to the current "Standard Specifications for Hollow, Load-Bearing Concrete Masonry Units" of the American Society for Testing Materials, for the grade of block having a minimum compressive strength on the gross area of 1,000 psi. Solid blocks should conform to current "Standard Specifications for Solid Load-Bearing Concrete Masonry Units" of the American Society for Testing Materials, for Grade A blocks, having a minimum compressive strength on the gross area of 1,800 psi.

#### C. Minimum dimensions

The over-all thickness of the blocks as laid in the wall should not be less than 6 inches and the face shell thickness should not be less than 1½ inches.

#### D. Provision for lateral pressures

External hooping, spaced not farther apart than every second course, should be provided to withstand the whole lateral pressure due to the ensilage. (See Table 4 for lateral pressures.)

#### E. Provision for vertical load

The maximum unit compressive stress on the net area due to the vertical load should not exceed fifty percent of the compressive strength of the unit.

#### F. Mortar

The mortar used for laying up the blocks should not be leaner than one of portland cement to two of plaster sand by weight. Fully bedded joints should be used. The aggregate should contain not more than ½ of 1 percent of shale and other expansive material. See Sec. 3 A.

### 6. MONOLITHIC SILOS

#### A. Wall thickness

The wall thickness measured in a radial direction should not be less than 4 inches if external hooping is used; and of such thickness that the metal reinforcement is protected both inside and outside by not less than 2 inches of concrete if embedded reinforcement is used.

#### B. Construction joints

Whenever it is necessary to interrupt the placing of the concrete for walls for more than two hours, the concrete should be levelled and its surface roughened by brooming after set has started but before the concrete has appreciably hardened. Just before placing is resumed, a thin cement paste should be brushed into the surface of the previously placed concrete.

### 7. DESIGN CONSIDERATIONS

#### A. Lateral pressures and hoop spacing

The lateral pressures of silage given in Table 4 comprise all the lateral pressure data known to the Committee. These data were obtained in a research study in which U. S. Department of Agriculture, the New Jersey Agricultural Experiment Station, the National Association of Silo Manufacturers, and the Portland Cement Association cooperated. The work was done at the Dairy Research Farm, Sussex, New Jersey. The data include results from four tests using corn silage and 12 tests using grass silage. The grass silage data have been separated into two groups, alfalfa and peas, according to the chief material ensiled. Variables other than those shown in Table 4 were included in the investigation.

The data of Table 4 are plotted in Fig. 2, from which it is at once evident (a) that the relation between lateral pressure and head of silage is expressed by a curved line and (b) that there is no justification for assigning different pressure relations to different ensiled materials. Study of the data indicates clearly that lateral pressures increase with moisture content and with the diameter of the silo. It should be noted also that no data exist for silage heads greater than 40 ft. In the past, the silo industry has used equivalent fluid weights of 11 lb. per cu. ft. for

## TABLE 4—LATERAL PRESSURE OF SILAGE

| Silo No. | I | II | III | IV | V | VI | VII | VIII | IX | X | XI | XII | XIII | XIV | XV | XVI |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Ensiled Material | Corn | | | | | Alfalfa | | | | | | | | Peas | | |
| Moisture Content | 70% | 81% | 74% | 74% | 71 | 70 | 78 | 66 | 71 | 68 | 68 | 77 | 77 | 64 | 74 | 72 |
| Diameter of Silo | 14′-0″ | 16′-0″ | 18′-0″ | 18′-0″ | 11.6 | 12 | 12 | 14 | 14 | 18 | 18 | 18 | 18 | 12 | 14.3 | 18 |
| Reference | B | A | A | C | A | | | | | | | | | A | | |
| h | | | | | | | | | | | | | | | | |
| 5 | 46 | 37 | 36 | 38 | 46 | 27 | 48 | 40 | 28 | 29 | 36 | 33 | 30 | 23 | 49 | 38 |
| 10 | 84 | 109 | 107 | 97 | 92 | 77 | 193 | 83 | 91 | 94 | 97 | 106 | 111 | 63 | 110 | 113 |
| 15 | 143 | 200 | 171 | 166 | 138 | 132 | 322 | 112 | 142 | 170 | 153 | 213 | 215 | 90 | 179 | 192 |
| 20 | 178 | 332 | 238 | 266 | 185 | 169 | 427 | 152 | 193 | 242 | 215 | 334 | 350 | 117 | 251 | 277 |
| 25 | 218 | 498 | 299 | | 258 | 196 | 477 | 209 | 239 | 337 | 295 | 500 | 544 | 159 | 314 | 376 |
| 30 | 234 | 645 | 391 | | | 220 | 550 | 281 | 302 | 527 | 349 | 601 | 759 | | 381 | 464 |
| 35 | 280 | 760 | 457 | | | 268 | 570 | 310 | 414 | | | 730 | 974 | | | 610 |
| 40 | 320 | 900 | 570 | | | 338 | 696 | | 520 | | | 950 | 1189 | | | 734 |

Reference A—Mimeographed paper entitled "Observations on the Storage of Grass Silage" by H. E. Besley and J. R. McCalmont, presented at the annual convention of the NASM, Chicago, Dec. 2, 1940.

B—A photographic print entitled "Silo Investigations", by Bureau of Agricultural Chemistry and Engineering, USDA, Washington, D. C.

C—Mimeographed paper entitled "A Progress Report". New Jersey Silo Research Project by H. E. Besley, W. L. Humphries, J. R. McCalmont, and W. H. Tamm. 1940.

corn silage and 20 lb. per cu. ft. for grass silage. The two heavy straight lines on Fig. 2 represent the pressure-head relations of these equivalent fluid weights. The 20-lb. per cu. ft. relation is conservative for low heads and is not suitable for extrapolation.

Because silos are built with heights greater than 40 ft., extrapolation becomes a practical necessity. Knowing the dangers of extrapolation, the curve of best fit was obtained by least square. This is the heavy curved line of Fig. 2 marked $L=3.3h^{1.44}$ in which $L$=lateral pressure in psf and $h$=vertical distance in feet from the point at which pressure is determined to the top of the silage. The lateral pressure of either grass or corn silage of moisture content not exceeding 75 percent should be calculated by this formula.

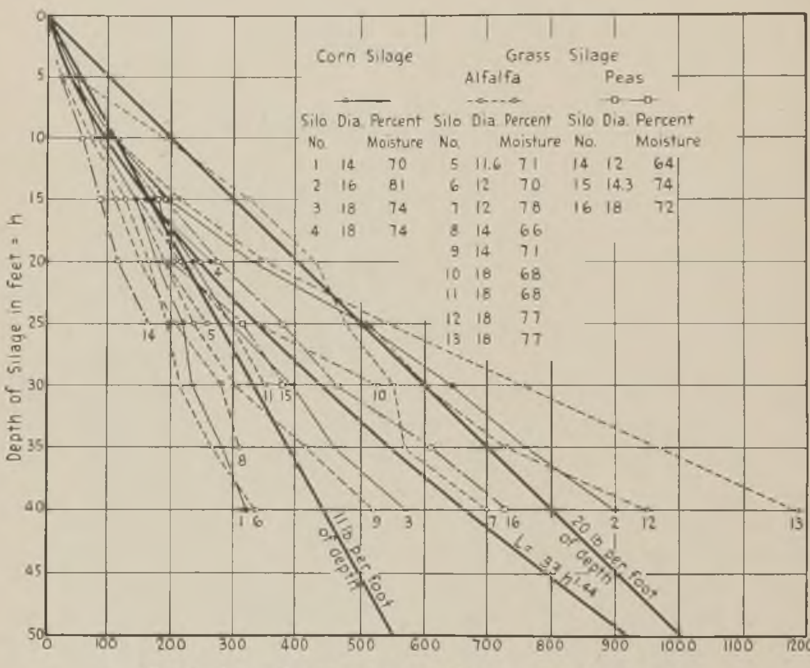



Fig. 2—Lateral pressure in pounds per square foot = L

The regression lines were nearly parallel, indicating good correlation between this curve and the data. The apparent accuracy with which the equation is expressed does not, of course, indicate that the formula will tell closely the pressure in a given silo. It is evident from Fig. 1 that the lateral pressures for individual silos may vary widely from those given by the equation. A method of calculating silo pressures which includes the major variables, such as diameter and moisture content, is highly desirable, but such a formula has not been developed, nor are the data required for its development available at the present time.

Referring again to Fig. 2, it will be seen that the lateral pressures for silos 2, 12, and 13 are markedly in excess of the lateral pressures of the curve and also that the moisture contents were 81, 77, and 77 percent respectively. For this reason, the use of the equation $L=3.3h^{1.44}$ has been limited in the "Practice" to silage having moisture contents of 75 percent or less.

The use of silage of higher moisture content would not cause collapse of the silo but might do damage requiring repairs before refilling. Using the data for silo 13 as an example, its lateral pressure at a 40-ft. head is 1.78 times the curve pressure. Assuming a unit stress of 20,000 psi in the hooping for the curve pressure, the unit stress in silo 13 would be 20,000x1.78=35,600 psi, a unit stress within the elastic limit. The strain in the steel would unquestionably exceed the elastic recovery of the concrete, and in consequence, vertical cracks would develop between the staves, thus permitting leakage. The leakage will cause surface damage to both staves and hoops but will also reduce the lateral pressure. Hence no danger beyond the possibility of a local and easily repaired surface damage will result from the use of high moisture content silage in silos with hooping designed by the curve $L=3.3h^{1.4}$.

Values of the recommended lateral pressures and the corresponding hoop spacings for ½-in. and 9⁄16-in. diameter rods and an allowable unit stress of 20,000 psi calculated by the formula below are given in Table 3.

$$s = \frac{24 f_s A_s}{LD} \qquad (1)$$

in which $s$ = spacing of hoops in inches
$f_s$ = allowable unit stress in the hoop in psi.
$A_s$ = area of hoop in sq. in.
$L$ = lateral pressure in psf.
$D$ = diameter of silo in feet inside to inside of silo wall.

**B. Vertical wall loads, widths and depths of footings**

Silage pressure on the silo walls has a vertical as well as a lateral component. The vertical component might also be called a friction load developed because of settlement or the tendency of silage to settle. Data for vertical loads on walls are available from the same source as for lateral loads. These data are given in Table 6 and are plotted in Fig. 3. The curve of best fit for the data of Table 6 was found to be

$$f = 5.5h^{1.08} \qquad (2)$$

in which $f$ = vertical wall load in psf
and $h$ = depth below top of silo in feet

### TABLE 5—WEIGHTS IN POUNDS OF HOOPING REQUIRED FOR 50 FOOT SILOS

Figures in brackets are relative weights.

| Diameter of Hooping inches | Design Stress psi | Lateral Pressure psf | Diameter of Silo in Feet: 10 | 12 | 14 | 16 | 18 | 20 |
|---|---|---|---|---|---|---|---|---|
| $\frac{9}{16}$ | 20,000 | $L=3.3h^{1.44}$ | (1.0) 851 | (1.0) 1118 | (1.0) 1452 | (1.0) 1830 | (1.0) 2200 | (1.0) 2660 |
| $\frac{1}{2}$ & $\frac{9}{16}$ | 20,000 | $L=3.3h^{1.44}$ | (0.92) 782 | (0.93) 1041 | (0.96) 1386 | (0.94) 1728 | (0.95) 2099 | (0.96) 2567 |
| a. $\frac{9}{16}$ | 18,000 | 20h | (1.25) 1062 | (1.31) 1470 | (1.25) 1825 | (1.37) 2510 | (1.41) 3110 | (1.41) 3775 |
| a. $\frac{9}{16}$ | 18,000 | 11h | (0.88) 745 | (0.85) 958 | (0.87) 1268 | (0.86) 1575 | (0.87) 1916 | (0.84) 2238 |

a. Spacings used were those given in "Silo Hoopage Standards", February 29, 1940, National Association of Silo Manufacturers.

NOTE:—Weights for a combination of ½-in. and ⅝-in. hoops are greater than for the combination of ½ and $\frac{9}{16}$ hoops.

### TABLE 6—VERTICAL LOADS ON SILO WALLS DUE TO SILAGE

| Silo No. | III | IV | VII | X | XI | XVI |
|---|---|---|---|---|---|---|
| Ensiled Material | Corn | | Alfalfa | | | Peas |
| Moisture Content % | 75 | 74 | 78 | 68 | 68 | 72 |
| Diameter of Silo | 18.0 | 18.0 | 12 | 18 | 18 | 18 |
| Reference | A | C | A | A | A | A |
| h | | | | | | |
| 5 | 39 | 27 | 41 | 26 | 24 | 46 |
| 10 | 70 | 66 | 67 | 51 | 58 | 84 |
| 15 | 95 | 98 | 95 | 92 | 81 | 128 |
| 20 | 133 | 126 | 188 | 123 | 116 | 151 |
| 25 | 176 | | 246 | 166 | 144 | 179 |
| 30 | 233 | | 265 | 225 | 176 | 199 |
| 35 | 270 | | 264 | | | 218 |
| 40 | 300 | | 307 | | | 244 |

Reference A—Mimeographed paper entitled "Observations on the Storage of Grass Silage" by H. E. Besley and J. R. McCalmont, presented at the annual convention of the NASM, Chicago, Dec. 2, 1940.

C—Mimeographed paper entitled "A Progress Report". New Jersey Silo Research Project by H. E. Besley, W. R. Humphries, J. R. McCalmont, and W. H. Tamm. 1940.

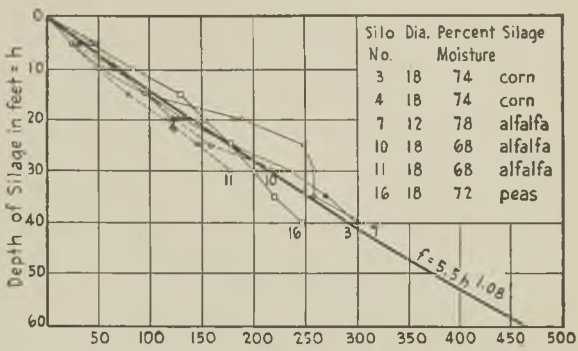
The vertical load per foot of circumference at a distance $h$ is given by

$$F = \int_0^h 5.5h^{1.08} = 2.64h^{2.08} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For a vertical cylindrical wall centered on an annular footing of any type the width at the base of the footing required to support the wall and vertical friction loads is given by formula (4):

$$w = \frac{12h_1}{p}\,(12.5t+2.64h^{1.08}{}_1) \quad \text{.......................... (4)}$$

Fig. 3 — Vertical friction load in pounds per square foot = f




in which $w$ = width of footing in inches at the base
$h_1$ = distance from top of silo to top of footing in feet
t = thickness of wall in inches
$p$ = allowable soil pressure in psf

The increase in width of a footing for a 14x40 ft. silo when the weights of the roof, chute, reinforcing steel, footing and 2½ foot foundation wall were added to the weights of the stave wall and the vertical friction load, was 6.4 percent. Considering the uncertainty as to the allowable soil pressure, sufficient accuracy is obtained by use of the vertical friction pressure and the weight of the wall. If desired, the increment in width required for additional loads can be obtained by use of formula (5):

$$\Delta w = \frac{12\Delta\, W}{p\,\pi\, D} \quad \text{................................. (5)}$$

in which $\Delta w$ = increment in width of annular ring footing for an increment in loads of $\Delta\, W$ pounds.
$p$ = allowable soil pressure in pounds per square foot
$D$ = diameter of silo in feet inside to inside of silo wall.

The widths of footings calculated by formula (4) for the recommended soil pressures are given in Table 1 and Table 2, with a 12 in. minimum width.

Wind loads need not be considered in the design of silos not exceeding 60 feet in height, because the vertical pressure due to wind is (a) small relative to the total load resulting from the gravity load of the silo and the vertical component of silage pressure, (b) of short duration when of maximum intensity, and (c) because the chance of maximum wind load and maximum silage load occurring simultaneously is small.

The width of footing required for vertical load and wind load, if considered, should be calculated by formula (6):

$$w = \frac{12h_1}{p}(12.5t+2.64h_1^{1.08}+\frac{10h_2}{D}) \qquad (6)$$

in which the significance of the symbols is as previously defined and

$h_2$ = distance from ground line to the top of the silo walls plus an allowance for the roof.

*NOTE:*—Formula (6) is based on a horizontal wind pressure of 15 psf on the vertical projection.

Plain or reinforced footings should be designed for depth according to Building Regulations for Reinforced Concrete (ACI 318-41) or the 1940 report of the Joint Committee submitting Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete.

The depth of plain footings should be calculated by formula (7)

$$d = \frac{\sqrt{p}}{131}w \qquad (7)$$

in which $p$ = allowable soil pressure in psf.

and $w$ = width of footings in inches as calculated from formula (4) or formula (6).

$d$ = depth of footing in inches.

Shear is not a limiting factor in the design of silo footings as described herein.

Recommended soil pressures: In the absence of tests or other specific knowledge of the safe bearing capacity of the soil, it is recommended that the following values be not exceeded:

| | Kind of Soil | Bearing Capacity psf | Ratio $\frac{d}{w}$ |
|---|---|---|---|
| (Type 1) | Sand and gravel | 8,000 | .68 |
| (Type 2) | Firm clay, wet sand or clay and sand mixture | 4,000 | .48 |
| (Type 3) | Soft clay | 2,000 | .34 |

These values are used in calculating widths and depths of footings given in Table 2, with the help of formulas (4) and (7), 8 in. being the minimum depth.

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# The Durability of Concrete in Service*

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Member American Concrete Institute

## SYNOPSIS

This paper discusses the problem of concrete durability with reference primarily to highway bridge structures located in regions subject to severe frost action. Four major types of deterioration are defined and illustrated and several specific matters which have bearing on the problem, including the effect of construction variables, modern vs. old fashioned cements, air entrainment and the so-called "cement-alkali" aggregate reaction, are discussed. The report concludes with a series of recommendations indicating certain corrective measures which should be taken.

## INTRODUCTION

During the summer of 1944 I had the opportunity of making a rather extensive survey of concrete highway bridges located in the States of Wyoming, Oregon, Washington and California. This survey was part of a research program recently undertaken by the Public Roads Administration for the purpose of investigating the causes of the alarmingly rapid rate of disintegration which was being observed in these and other Western States. In all some 200 structures ranging in size from small, single span bridges to major grade separation structures, and in age from 3 to 30 years, were inspected. The total time available for this survey was about 3 weeks. For this reason no detailed examinations were attempted. However, sufficient notes were taken to provide a very good record of the type and extent of the disintegration, if any, as well as the parts of the structures which were most seriously affected. Following the inspection, a report was prepared which included, in addition to a description of the types of failure observed, a discussion of certain matters which appeared to be pertinent to the problem, such as construction

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variables, modern vs. old-fashioned cements, the so-called "alkali-aggregate" reaction, air entrainment, etc. The report also contained suggestions as to corrective measures, including definite recommendations for changes in specifications.

This paper has been written more or less around the Western report and I trust that any of you who may have chanced to read that report will forgive any repetitions of thought which may appear. However, in this discussion I shall generalize my remarks with the idea of covering the problem from a national rather than from a regional standpoint. Fortunately, most of my discussion of the Western regional problem applies equally well to the broader field and this I shall endeavor to bring out in the discussion to follow.

## THE PROBLEM

I am quite sure you will agree with me that the lack of durability which is so evident in concrete exposed to severe natural weathering, is a most serious problem. The present condition of far too many concrete bridges and pavements, here in the East as well as in the West, bears mute testimony to the truth of this statement. I am also equally sure that you will agree that this is a matter with which you, as manufacturers of one of the essential components of concrete, should be vitally concerned. I am aware, of course, of the excellent work being done by your organization through the Research Foundation at the University of Maryland. In my opinion we should not only continue but should materially expand our activities along the lines represented by this project because I tell you frankly unless we can find the answer soon we are going to be faced with a drastic curtailment in the volume of concrete used in highway work. This is going to take the form of substitute types—that is, substitution of wood and steel for concrete in the construction of bridges and the substitution of bituminous types for concrete in the construction of pavements. To me this would be distinctly unfortunate. Concrete has great possibilities as a structural material. Its extreme versatility accounts for the enormous strides which have been made in its use during the past half century. Much of this work has been good. However, the defects which have developed, particularly in the newer structures, are sufficiently serious to warrant real concern. In fact, the trouble is so serious that unless we can learn to design and build structures which will have a more reasonable life expectancy than many of those built within the last 15 years or so, I am very much afraid this material will lose the high place in the construction field to which its inherent advantages entitle it. I say the last 15 years advisedly, because there is evidence from many sources that, by and large, concrete structures built after 1930 or thereabouts have not proved as durable in service as earlier structures subjected to the same weathering conditions. I shall discuss this situation more fully later on.

For the purpose of this discussion, we shall define durability as that property of concrete which enables it to resist the destructive forces produced by one or more of the following causes: (a) Natural weathering, that is, repeated freezing and thawing, heating and cooling, wetting and drying. (b) Chemical attack from the outside, as for example attack by sea water or the alkalies in soils. Also attack by chloride salts used on pavements for ice removal. (c) Expansion caused by elements within the concrete, such as unsound cement, expansive aggregate, or a chemical reaction between cement and aggregate.

In this discussion we are not concerned with so-called structural failures as such. These usually result from faulty design and in the case of bridges at least, are quite rare. However, cracking which results from structural failure is significant from the standpoint of durability because of the likelihood of moisture penetrating the cracks followed by freezing and ultimate disintegration.

## TYPES OF DETERIORATION

My inspection of Western bridges indicated to me that, aside from structural failure, the deterioration of concrete in service may be classified with respect to cause into four distinct types. Furthermore, such observations as I have made in the East lead me to believe that the description of the four types which I prepared for the Western report apply equally well to conditions in other parts of the country. The relative frequency with which the various types appear may be quite different but examples of all of them have been observed in other sections. I think you will agree with this when you hear the following description of the several types which I prepared for the Western report and which has been somewhat amplified for this discussion, principally for the purpose of bringing pavements as well as bridges into the picture.

### Type 1. Deterioration due to gradual or normal weathering.

This is indicated usually by slight surface erosion and pitting, rounded corners, etc. Pavements may show considerable surface wear, which should not be confused with scaling, as well as spalling along the edges of cracks and joints, particularly if joints have not been properly maintained. Many old bridges may also show cracks due to settlement and impact from colliding vehicles. (Fig. 1.) The edges of such cracks show little evidence of weathering. Concrete of this type has a good ring under the hammer and a good sharp chip may be obtained. The matrix is dense and crystalline in appearance and usually a bluish-gray in color. This condition is evidence of sound, durable concrete, especially when found on structures subject to severe natural weathering. We find concrete of this character on many of the older pavements and bridges.

**Type 2. Deterioration due to accelerated weathering.**

This type of failure is all too common, particularly in areas subject to severe frost action. In the case of bridges it is usually evidenced first by the formation of fine cracks on the surface of exposed members such as curbs, handrails, end posts, tops of retaining walls, wing walls, etc. They usually appear first on surfaces adjacent to construction joints or at cracks and other points where water can enter. (Fig. 2.) Cracks of this type also tend to form along the edges of thin members such as curbs and handrails. The cracks are ordinarily close to and parallel to the edge and are usually filled with a dark-colored deposit, probably calcium carbonate.

Concrete containing cracks of the type 2 variety, frequently called "D lines," has little strength and can be broken easily under the hammer. The matrix has a dull, chalky appearance in sharp contrast to the dense, compact, bluish-gray matrix usually found in good concrete. Progressive and rapid disintegration usually follows the appearance of D lines. (Fig. 3.) In the case of pavements, cracks of this type almost invariably form first along transverse joints and cracks where water can enter, later along longitudinal joints and free edges. D lines at the joints are frequently, but not always, followed by scaling and progressive spalling of the type with which you are all too familiar.

On bridges, distress of the type 2 variety, if it develops at all, usually starts in the relatively thin members of the superstructure. They are the members most directly exposed to weathering. The sections are frequently thin, necessitating the use of a rather fine graded, coarse aggregate which, in connection with the fact that these members usually contain considerable reinforcing steel, tends to result in the use of a higher water content than in the heavier members, even though the cement content in bags per cubic yard may be exactly the same. This raises a question as to whether thin sections should be used where the concrete will be subjected to severe weathering. In the case of pavements, the frequency with which distress of this type develops along transverse expansion joints, would indicate the desirability of reducing to the minimum consistent with good design the number of such joints, especially in regions where trouble of this type may be anticipated.

**Type 3. Deterioration due to chemical attack from the outside.**

Deterioration of this type is the result of chemical attack by aggressive waters, such as sea-water, acid waters carrying mine and factory wastes, water containing alkalies leached from the soil, etc. Sulfate salts in these waters attack the lime in the concrete with the formation of calcium sulfo-aluminate, a double salt which exerts great disruptive force

**Fig. 1. (top) – Type 1 deterioration. Structural damage due to colliding vehicle. Concrete sound. Bridge built in 1921, photographed in 1944.**

**Fig. 2. (center) – Type 2 deterioration. Formation of "D" lines at construction joint. Bridge built in 1931, photographed in 1944.**

**Fig. 3. (bottom) – Type 2 deterioration. Advanced disintegration. Note good concrete in rail at left illustrating non-uniformity of concrete. Bridge built in 1931, photographed in 1944.**

through the process of crystallization. Deterioration of this type starts with a general softening of the surface followed by crumbling and general disintegration. So far as this discussion is concerned, I shall treat corrosion of concrete caused by chemical attack as a special case and shall not go into detail. Failures from these causes are not nearly so widespread as those due to type 2 deterioration. Furthermore, a great deal of work has been done on this particular problem.

However, I would like to discuss a type of pavement failure which may be classified as type 3 and which is the result of using calcium or sodium chloride for ice removal. Pavements and sidewalks so treated, especially if the concrete is new, will almost invariably scale badly, the mortar surface being, in many cases, completely removed during the first winter. (Fig. 4 and 5.) Failures of this type have unfortunately been quite common, as you well know, particularly on heavily traveled roads of the Northeastern States. A specific remedy for this condition has apparently been found in air entrainment, which will be discussed later.

**Type 4. Deterioration due to abnormal expansion.**

All concrete will expand or contract when subjected to changes in either temperature or moisutre content. This property is recognized and provided for in the design of the structure. Occasionally, however, a tendency towards abnormal expansion develops which cannot be explained in the usual way. Some failures of this type which have been noted in the past were probably due to unsound cement. There is, of course, no particular mystery about this type of failure, as it is well known that so-called hard-burned free lime may exist in cement that has been improperly manufactured and that the gradual hydration of this lime within the concrete will cause disruptive expansion. Manufacturers of portland cement claim that failures of this type have been entirely eliminated by the adoption of the autoclave test for soundness of cement.

Aggregates, also, may be responsible for abnormal expansion. Either thermal or moisture effects, or both, may be involved. In Nebraska and Kansas, for example, the use of the prevailing aggregate (sand-gravel from the Platte and other rivers) in concrete pavement construction almost invariably results in abnormal expansive cracking within a few years. The blending of other types (called sweetening) with the sand-gravel usually corrects the difficulty.

In discussing the effect of aggregates on the durability of concrete, I am not thinking in terms of manifestly unsound materials, as for example, shales and cherts, which break down readily under freezing and thawing. Such materials should never be used in concrete and we have tests (unfortunately not too good) which will identify and reject them. I am, however, concerned with the possible effect of wide variations in the

Fig. 4 (top) – Type 3 deterioration. Partial removal of mortar top from pavement due to excessive use of chloride salts for ice removal. Pavement built in 1934, photographed in 1944.

Fig. 5. (bottom) – Type 3 deterioration. Complete removal of mortar top from pavement at heavily salted intersection.

thermal and elastic properties of aggregate on similar properties of the concrete in which they are used. These effects are not nearly so well understood. However, it should be quite obvious that when an aggregate having either an unusually high or an unusually low thermal coefficient of expansion is rigidly bound in concrete by a matrix having a different coefficient of expansion, internal stress is bound to develop when that concrete is subjected to temperature change. The same is true for aggregates having wide variations in their capacity to absorb moisture. Not too much attention has been paid to these matters in the past. More attention will be paid to them in the future.

The most serious form of delayed expansion which has so far been positively identified cannot be attributed to unsoundness in either the cement or aggregate alone. Rather conclusive evidence of the so called "cement-alkali aggregate" reaction has been observed in several western areas, including Wyoming, Washington and California. This type of cracking is generally assumed to be due to internal expansion resulting from a reaction between small amounts of the alkalies (sodium and potassium oxides) in the cement with certain siliceous constituents of the aggregates. In certain cases opaline silica has been identified as the reactive material in the aggregate. In other cases certain igneous rocks, mostly volcanic, have been identified as reactive. The action was first observed in Southern California by T. E. Stanton and later in the Mt. Ranier region of Washington by Bailey Tremper. It has subsequently been identified in Western Nebraska, as well as in portions of Wyoming and Idaho. It has, so far as I know, not as yet been positively identified in any State east of the Mississippi River, although there are indications that some of our troubles with concrete here in the East may be due to this cause.

So far, evidence of the alkali-aggregate reaction has been based almost entirely on observation of structures in service and on expansion tests of specimens containing high and low-alkali cements in combination with aggregates of various types. Tests and experience correlate very well insofar as the identification of combinations which cause trouble is concerned. However, the actual mechanism of the action is not yet fully understood although various agencies, including the Bureau of Reclamation and the Portland Cement Association, are working on this phase of the problem.

Failures due to this reaction are characterized by the formation of relatively wide, open cracks of appreciable depth and usually roughly parallel to the longitudinal axis. (Fig. 6.) Random or pattern cracks, also open and rather widely spaced, are frequently formed. (Fig. 7.)Failure due to abnormal expansion may or may not be followed by ordinary weathering (type 2) depending upon the severity of exposure. For

**Fig. 6. (left)—Type 4 deterioration. Wide, open cracks in wing wall. Concrete otherwise sound.**

**Fig. 7. (right) – Type 4 deterioration. Wide, open, random cracks in pedestal cap. Concrete adjacent to cracks hard and sound.**

example, certain structures in southern California which showed characteristic cracking of this type as early as 1937 are still in about the same condition, indicating that expansion has ceased. In the absence of frost no further action has developed in these particular structures. On the other hand, there is considerable evidence from structures located in higher altitudes to indicate that failures which may have been started by abnormal expansion have been greatly accelerated by freezing and thawing, ultimate disintegration resulting from a combination of the two.

Positive identification of the type or types of a particular failure is not always easy. This applies particularly to structures in which distress due to alternate freezing and thawing (type 2) may have been preceded by expansion due to an alkali-aggregate reaction (type 4). In such cases it is probable that the internal expansion due to chemical action or differential thermal stress may so weaken the concrete as to greatly lower its resistance to frost action. This theory also tends to explain the disturbing fact that some concrete structures built under modern specifications and under close engineering supervision, and which by all the rules should be durable, are not performing as satisfactorily as they should. However, the theory does not explain why many of the older structures located in the same areas are still in good condition. The cements used in the old structures (age 20 years or more) were probably as high in alkalies as were the corresponding cements made 10 or 12 years ago. The aggregates also were of the same general type. However, there were other differences in the cements which may explain the inability of some of the more recently constructed bridges to resist weathering as well as comparable structures built 20 or more years ago. This possibility will be discussed more fully later in the report.

## GENERAL DISCUSSION

In the balance of the paper I should like to discuss one or two phases of this problem which I consider quite significant. In doing so, I shall use virtually unchanged the discussion which I originally prepared for the Western report. Except in one particular, that discussion applies equally as well to the national as to the regional problem. The exception concerns the so-called cement-alkali reaction. So far as we know now, this reaction which causes type 4 expansion, is not a problem in the East, although as I previously stated, further investigation may reveal that it is the cause of some of our trouble in this part of the country also.

## CONSTRUCTION VARIABLES

It is a basic principle of concrete making that, given sound materials, durability is governed largely by the quality of the cementing ingredient —that is, the cement paste—and, further, that the quality of the paste is controlled largely by its relative water content (the water-cement ratio). This principle is as firmly established today as it ever was although the relatively new principle of air entrainment may seem at first thought to run counter to the idea that durability is a function of the density of the concrete.

The influence of the water-cement ratio on durability is so well recognized today that virtually all specifications for concrete to be exposed to severe weathering either require directly that the water-cement ratio not exceed 6.0 gallons per sack of cement, or specify requirements for minimum cement content and consistency in such a way as to insure that this value is not exceeded. A study of specifications governing construction of the various bridges inspected on the Western trip indicates that, in general, proportions were used which would insure compliance with this requirement in combining materials at the mixer. Unfortunately the process of handling and depositing concrete in the forms, even when done under good supervision, frequently results in segregation, nonuniform compaction (particularly at joints), bleeding or water gain, and so forth. Ideally, concrete should be absolutely uniform in composition but variable factors all combine to make a product which actually may be far from uniform. The result is that even though the water-cement ratio of the mix as designed may be within the required limits, the actual water content in certain parts of the structures may be too high for adequate protection. The non-uniform distribution of concrete within a member as well as batch-to-batch variations in the quality of the concrete as mixed, variations in curing conditions, etc., all undoubtedly combine to produce the nonuniformity in resistance to weathering which is so often observed in various parts of the same structures. I am convinced that at least some of the troubles which were noted during the Western inspection can be attributed to these conditions.

In this connection, I would like to suggest adoption of a modification in placing procedure recently suggested by F. R. McMillan, of the Portland Cement Association. To prevent accumulation of a layer of relatively porous concrete of high water content in the top surface of vertical lifts, Mr. McMillan proposes the following: Instead of immediately striking off the concrete when the top of the form is reached, allow it to pile up 2 or 3 inches above the form, leaving it undisturbed for an hour or so in order that the excess water resulting from bleeding may accumulate in this layer. As soon as the bleeding has ceased, strike off and finish in the usual way. The usual porous layer, instead of remaining in the concrete as a potential source of weakness, will be wasted. The only loss will be a little concrete, the cost of which is a small price to pay for the elimination of a very common defect.

## AIR ENTRAINMENT

In my opinion, the application of the principle of air entrainment offers the most promising immediate solution to the problem of how to obtain more durable concrete. Originally suggested as a means for controlling salt scaling on pavements, it now appears that, in addition, resistance to freezing and thawing in general is greatly improved by the use of an air-entraining agent in the concrete. The effectiveness of air entrainment seems to lie in the fact that numerous minute air voids are incorporated in the concrete, and that these voids seem to act as cushions against the expansive force produced by freezing water. In any event, both laboratory tests and field experience indicate that substantial improvement in durability is obtained with little sacrifice in strength provided proper control is exercised in the use of the material.

Air can be entrained in concrete in two ways: first, by the use of air-entraining portland cement, a cement in which the air-entraining agent has been interground during manufacture and second, by adding the air-entraining agent directly to the concrete at the time of mixing. For the time being the writer favors the practice of adding the material at the mixer for several reasons. The amount of air that will be entrained in any particular case depends upon a number of factors in addition to the amount of active agent in the cement. The kind and grading of the aggregates, the cement content, the consistency of the concrete, the type of mixer and the mixing time all affect the result, so that merely finding the amount of air-entraining agent to be carried by the cement, or even fixing the amount of air which will be entrained in a standard test mortar as is done in the latest revision of A.S.T.M. Specification C-175 is not sufficient. The only way in which the effect of all of the variables can be taken into account is to place control in the hands of the engineer on the job, permitting him to adjust the amount of air-entraining agent depending upon the particular conditions under which he is working.

It is important that close control be exercised because of the rather narrow limits of air content that must be maintained for optimum results. At the present time it seems that a total air content of from 3 to 6 percent based on the theoretical weight of the air-free concrete will be about right. Less than 3 percent may not be sufficient for maximum durability, whereas when the total air content exceeds 6 percent a substantial loss in strength without any further increase in durability may result. In a paper* published in June 1944 H. F. Gonnerman, of the Portland Cement Association, presents the results of tests showing the effect of using both air-entraining portland cements and air-entraining materials added to the batch at the mixer. Several air-entraining agents were tested and if properly used should give satisfactory results. However, at the moment, the material supported by the greatest background of actual experience is Vinsol resin. This material has been used to a considerable extent interground with the cement. It has also been used, in the form of a Vinsol resin-sodium hydroxide solution, by adding to the batch at time of mixing. A weight test on the fresh concrete will indicate quite accurately the amount of this solution to add in any given case to produce the required air content. In addition to Vinsol resin, the material known commercially as Darex has also been approved for use in the manufacture of air-entraining portland cement, under A.S.T.M. Specification C-175. This material may also be added at the mixer as an air-entraining agent. It has also been used to a considerable extent and has the advantage over Vinsol resin of being soluble in water. No admixture should be used that has not been throughly tested.

Although the writer favors the practice of adding the air-entraining agent at the mixer, it is realized that there are certain construction problems involved in this practice which are avoided by the use of air-entraining cement. The difficulty of properly controlling the addition on the job of very small quantities of such active materials as Vinsol resin is recognized. With Vinsol resin there is also the problem of preparing the material for use. To many engineers these practical matters

*"Tests of Concrete Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," ACI JOURNAL, June 1944, *Proceedings* V. 40.

outweigh the theoretical advantages of the procedure recommended in this report. However, it should be emphasized again that the really important matter is the amount of air which is entrained in the concrete as mixed on the job.

Not only must the air content be sufficiently high to insure the desired improvement in durability but it must not be so high as to jeopardize strength, bond-with-steel or any other essential property of the concrete. As previously stated, the optimum range appears to correspond to a total air content of from 3 to 6 percent. This is a rather narrow range and will require close control. Regardless of the method used to obtain air entrainment (by addition of an air-entraining agent at the mixer or the use of air-entraining cement), provision for such control during construction must be made in the specifications which govern the work. This can be done through the use in the field of a weight test such as A.S.T.M. C 138-44, with provision in the specifications for making such changes in materials, proportions, or methods of mixing as may be necessary to obtain the proper air content at all times. Methods of determining the air content directly have also been developed recently.* These methods have the advantage over method C 138 in that the results are independent of variations due to errors in the determination of specific gravity.

There are certain indirect benefits associated with the use of air-entraining agents that should be mentioned. They have a distinctly plasticizing effect, especially in the leaner mixes, and, in general, reduce bleeding or water gain substantially. On this account it should be possible to obtain more uniform placement and thus avoid some of the segregation troubles to which reference has already been made. The additional plasticity makes it possible to reduce the sand content about 3 percent, thus at least partially compensating for the increased yield (lower cement content) which results from the air entrainment. This adjustment also tends to overcome the loss in strength which almost invariably accompanies increase in air content.

## MODERN VERSUS OLD-FASHIONED CEMENT

Many of the older bridges covered by the Western inspections were in surprisingly good condition. In reviewing their condition with respect to age they seem to fall roughly into two groups: those built during the twenties and earlier and those built subsequent to 1930. It can be stated definitely that, on the whole, the bridges in the first group show less disintegration than those in the second group. Of 36 bridges built before 1930, 24, or 67 percent, were free from defects other than those which could be classified as type 1 deterioration. On the other hand, of

*See "Entrained Air in Concrete: A Symposium," ACI JOURNAL, June 1946, *Proceedings* V. 42.

101 bridges built after that date only 28, or 27 percent, were so classified. A similar comparison has been furnished by Glen Paxson, Bridge Engineer of the Oregon State Highway Commission. He lists 61 structures on U. S. 30, between Umatilla and Ontario. Of the 37 built prior to 1930, 29, or 78 percent, have according to his record shown no evidence of weathering, whereas of the 24 built after that date only 9, or 37 percent, are so listed.

It is reasonable to suppose that, by and large, construction procedures are just as good now as they were 20 years ago. Presumably they are better. Likewise, engineering supervision has, on the average, improved. Certainly it has not deteriorated. The general character of the aggregates has not changed, although methods of processing, grading, and other factors tending to uniformity have improved. In view of these facts, it is not surprising that engineers should wonder about the cement. It is the one remaining factor. Is there some characteristic of modern cements not found in the old-fashioned type that tends to lower resistance to weathering?

The one outstanding difference is, of course, fineness. In 1917 the A.S.T.M. specifications required that not more than 22 percent be retained on the No. 200 sieve. The current specifications for type I portland cement require a minimum specific surface of 1,600 square centimeters per gram, a limit which is roughly equivalent to 3 or 4 percent retained on the No. 200 sieve. The increase in fineness has come about gradually and largely as the result of demands on the part of users for high early strength. This demand culminated in 1930 in the adoption of specifications for high early strength cement, with still greater fineness.

Quite recently a very interesting theory has been advanced which may account for the apprent fact that modern cements, ground to a fineness corresponding to a specific surface of, say 1,800 square centimeters per gram do not make as durable concrete as the more coarsely ground cements in use 25 years ago. According to the theory, the maximum limit of 2 percent sulfur trioxide which has been carried for years in the A.S.T.M. specifications, while satisfactory for the coarser ground product, is not high enough for the more active, finer ground, modern cements. This applies particularly to cements high in tricalcium aluminate ($C_3A$) as it is largely for the purpose of regulating the hydration of this particular compound that the gypsum is employed. Recent investigations by the Portland Cement Association indicate that from the standpoint of both strength and drying shrinkage, cements of high and moderately high $C_3A$ content are improved by adding higher percentages of gypsum than are allowed by the present specifications. Furthermore, even cements of low $C_3A$ content appear to be similarly improved if the alkali content is high. There are also some indications that drying shrinkage may be influenced by the alkali content of the cement independently of the $C_3A$ content. The Association is now engaged in an intensive study of this problem, with the idea of determining by means of laboratory freezing and thawing tests whether the use of larger quantities of gypsum will improve durability. If this is found to be the case, the sooner the specifications are changed the better.

## CEMENT ALKALI-AGGREGATE REACTION

The Western inspection confirmed the belief that there is need for restrictions on the alkali content of portland cement when used in areas where alkali-reactive aggregates are found. It is true that all of the aggregates in these areas may not be reactive. However, at the present time there are no tests, so far as the writer knows, that will reveal this characteristic in a reasonable length of time. The conventional expansion bar test requires several months at least. Furthermore, as pointed out by Tremper, cements high in alkali may react with certain aggregates in such a way as to weaken the resistance of the concrete to freezing and thawing even though the reaction is not sufficient to cause expansion in the bar test. For these reasons it seems wise, at least for the time being, to restrict the percentage of sodium and potassium oxide in the cement to 0.60 or less, calculated as $Na_2O$, in all areas where there is any evidence that reactive aggregates may be found.

## RECOMMENDATIONS

The following recommendations regarding specification changes are essentially the same as those which were made in connection with the Western report. They apply with equal force to the country as a whole:

That, where concrete is to be exposed to severe frost action, specifications be revised, where necessary, to require, in addition to a suitable cement content, that the free water content of the mix shall, in no case, exceed 6.0 gallons per sack of cement.

That, where concrete structures or portions of structures (including pavements) will be exposed to severe frost action, provision be made to entrain sufficient air in the fresh concrete so as to produce a total air content of from 3 to 6 percent, based on the theoretical weight of the air-free concrete.

That the desired air entrainment be obtained preferably by adding an approved air-entraining agent to the concrete at the time of mixing, in such quantity as will maintain the percentage of air within the limits specified.

That approval of any material proposed for use as an air-entraining agent be based on data obtained from either research or field use, or both, that are sufficiently comprehensive to demonstrate to the satisfaction of the contracting agency that the proposed material, when used as required, will not seriously affect the strength or other essential properties of the concrete.

That, when the desired air entrainment is to be obtained by the use of air-entraining cement: (1) The cement meet the requirements for air entrainment given in A.S.T.M. Specification C-175-46 T; and (2) the specifications authorize the engineer to require such changes in materials, proportions, or methods of mixing as may be necessary from time to time to maintain the percentage of air within the limits specified.

That, in all areas where tests or previous experience indicate that alkali-reactive aggregates may be encountered, specifications for portland cement be modified by requiring that the total percentage of sodium oxide plus 0.658 times the percentage of potassium oxide shall not exceed 0.60.

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# Wear Resistance Tests on Concrete Floors and Methods of Dust Prevention*

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## SYNOPSIS

This paper presents a description of tests made on concrete floor specimens of various types in order to determine their resistance to wear and to investigate the character of deterioration of concrete floor rurfaces due to traffic. The results of these tests show that concrete floors provided with finish courses containing coarse aggregate up to about ⅓ inch in size and an excess of pea gravel are definitely superior to concrete floors with a finish course containing fine sand only which are common in Sweden at the present time. Moreover, this investigation has helped to elucidate the causes of the often very severe and detrimental dusting of concrete floors. The surface skin of concrete floors is of poor quality and is easily abraded. Dusting can be considerably reduced if the poor surface skin is removed by machine grinding provided that the concrete below the surface skin is of first-rate quality. The paper concludes by proposing a detailed tentative specification for concrete floor finish which differs in essentials from current Swedish practice.

## INTRODUCTION

Extensive wear resistance tests on concrete floors were made at the Swedish Cement and Concrete Research Institute, Royal Technical University, Stockholm, in the autumn of 1943, and the results of these tests were published in the Proceedings of the Institute No. 5 (e), English edition. The following is a brief account of these tests.

## DETERIORATION OF CONCRETE FLOORS

The most common types of concrete floor deterioration are:

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Fig. 1—Pit, about 6 in. in dia. and $1\frac{3}{16}$in. deep in E-cement concrete floor in storage room of department store. Traffic was comparatively dense, but not very heavy. Wheels of trucks were rubber-tired.

(1) heavy wear resulting in profuse dusting, (2) crushing of concrete floor surface resulting in pits which manifest a marked tendency to increase in width and depth, (3) scaling of the shell of mortar covering the pieces of coarse aggregate just below the floor surface and (4) cracking.

All these types of deterioration cause trouble. Dusting is frequently so copious that the goods stored in warehouses are covered with a thick layer of dust. In machine shops and factories, dusting increases wear of machinery as the dust penetrates into bearings and sensitive parts.

Pitting occurs on concrete surfaces subjected to relatively heavy traffic, particularly where trucks do not have rubber tires, and the concrete is of poor quality. Surface blemishes of this type are shown in Fig. 1. Poor quality of a concrete floor is often due to the use of inadequate concrete mixes and inappropriate methods of construction.

Deterioration of the type mentioned under (3) is common on bridge decks, and may be assumed to be caused by freezing and thawing or by shocks and impacts produced by traffic.

Cracking can be an indirect cause of serious damage as cracked concrete surfaces are particularly liable to crushing and pitting.

## THE PRINCIPAL METHODS OF CONCRETE PAVEMENT CONSTRUCTION USED IN SWEDEN

The usual method of concrete floor construction used in Sweden, where a smooth surface is required, consists in placing on a concrete base slab a floor topping made of mortar which contains 1 part of portland cement to 2.75 parts by volume of sand having a maximum grain size of $\frac{1}{32}$ to $\frac{1}{8}$ inch. The consistency of the mortar is usually as moist earth. In order to obtain a smooth and dense surface under these circumstances, the floor surface must be sprinkled with water during wood-floating and steel-trowelling. Floors of this type have given both good and bad results in practice, but in general these floors are liable to considerable wear and intense dusting.

Under the late war conditions, when standard portland cement was rationed, the same method of concrete floor construction was sometimes used, but standard portland cement was replaced by E-cement, a blend, consisting of about 60 percent portland cement and 40 percent of an inert finely divided material, such as ground stone. To compensate in a measure for the inferior quality of this cement, a richer mix (1 to 2.25) was used. Experience has shown, however, that this type of floor is a complete failure, and it has often been stated, that "such floors can be swept away."

For highly wear-resistant concrete floors some special types of wearing courses have been advertised, most of them being of German make. In these wearing courses ordinary sand is replaced, wholly or in part, by extra hard aggregate. These special finish courses are known under various trade names; for instance, Concrete Hardener, Diamantconcrete, Duromit, Hard Concrete, and Westphal. All these finishes have given good results, but such floors are, as a rule, constructed by firms specializing in this field. Moreover, these special finishes are a good deal more expensive than the ordinary types of floor toppings.

On floors of the two-course type with a special wearing course which is placed after the base slab has hardened, the topping is often subject to cracking which may be due to two causes—failure of bond between the topping and the base, and greater drying shrinkage of the topping in comparison to the base. To prevent these drawbacks, attempts have been made to abandon the use of a special wearing course and to smooth directly the surface of the base slab. This method has been successfully applied in cases where no great freedom from dust is required, and where the consistency is comparatively stiff. For this purpose, two principal types of concrete slabs have been developed, slabs usually 6 inches in thickness placed and compacted by means of screedings and tamping machines, and thinner slabs, usually from 4 to 4½ inches in thickness, placed and compacted with vibrators. The cement content is usually 6 sacks per cu. yd. in the former case and 5 sacks per cu. yd. in the latter. First-rate workmanship is stipulated in the specifications of the Royal Swedish Board of Roads and Waterways, (1), (2).* These types of slabs have met the requirements for roadways and similar purposes.

Slabs made of vibrated concrete have found particularly wide application during the past few years. They are also used for floors in factories and warehouses, and have in most cases proved suitable.

The methods outlined above represent only the more important main types. Each of these methods permits many variations in the surface finish, such as wood-floating, brushing, steel-trowelling or rolling.

## SCOPE OF THE TESTS

The new road testing machine of the Swedish Road Institute (11) was used in making these tests and reproduced actual service conditions so well that results are directly applicable in practice.

The main program consisted of wear tests on 13 concrete slabs made and treated according to different methods, and on one limestone specimen. The limestone slab was included to provide a control specimen that was as invariable as possible in its properties with which all other specimens subjected to these tests and to later investigations could be compared. In addition, similar test specimens were cast according to the same methods, but on a small scale. These specimens were subjected to wear tests in a Bauschinger testing machine at the Swedish State Institution for Testing Materials. Furthermore, shrinkage measurements were made on prisms, compression tests on cubes, bending tests on beams, and dynamic elasticity measurements on small beams. All these test specimens were made of the same concrete mixes which were used for the main test specimens.

## TESTING METHODS

The road testing machine (11) is provided with a circular testing path (Fig. 2), having an inner diameter of 14 ft. 6 in. and an outer diameter of 20 ft. The testing path is divided into seven sectors, each 2 ft. 10 in. wide and with a mean length of 7 ft. 3 in. measured on the median circle. Each sector consists of a concrete box which can be moved out of and into the test room by means of various lifting and conveying devices.

The machine is provided with a vertical shaft which carries six horizontal arms. In normal tests a driving wheel connected to a separate D.C. motor is attached to each of these arms so that the wheels travel over the testing path, and are steered by the shaft (Fig. 3). The speed of the shaft, the resistance to the motion of the wheels and the wheel pressure can be varied within wide limits.

*Figures in parenthesis refer to the bibliography at the end of the text.


**Fig. 2—Road testing machine during wear tests. At left is shown one of the two driving wheels, left center a steel truck wheel and at right a leather-covered wheel.**

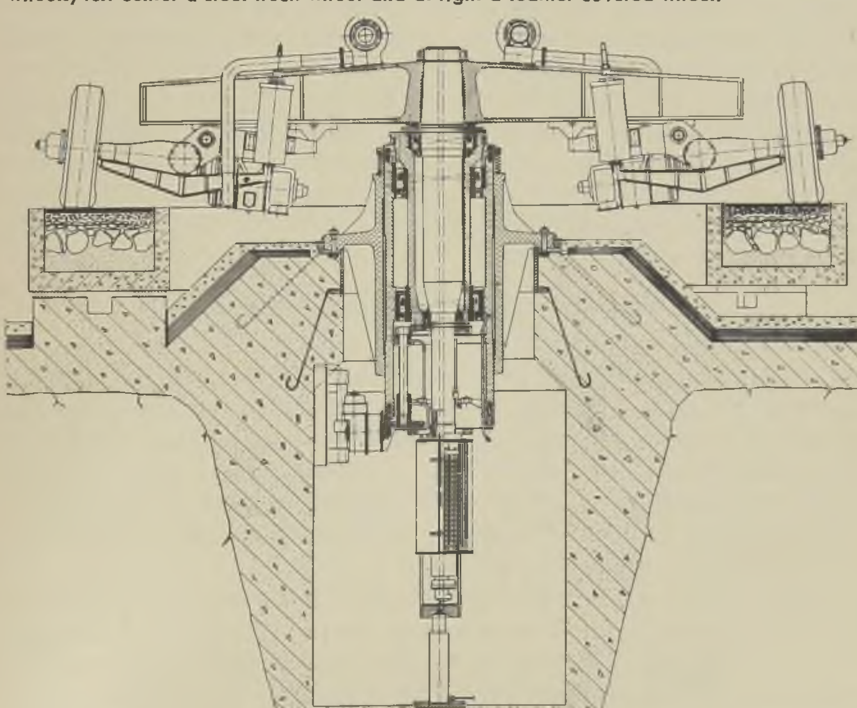


**Fig. 3—Cross-sectional view of the road testing machine in normal operation.**

For the purpose of the wear tests dealt with in this paper, the concrete floor specimens were made in the concrete boxes mentioned above, and then stored in a room adjoining the test room. A short time before the beginning of the tests, the boxes were moved into the test room and put into the road testing machine. Two different floor specimens were made in each box, each specimen being about 2 ft. 10 in. in width and about 3 ft. 7½ in. in mean length.

Two truck wheels covered with solid rubber tires and coupled to separate D.C. motors were attached to the arms of the shaft. The wheels travelled in a circular track in the middle of the testing path and revolved the shaft. Unlike the other wheels, these two wheels moved with rolling motion only. The wheel pressure was 1760 lb., the diameter of the wheel 2 ft. 1 in. and the width of the wheel rim 2⅜ in.

The other four arms of the shaft were provided with pivot under-carriages specially designed and constructed for these tests (Fig. 2). Each of two of these under-carriages were fitted with a steel truck wheel, and each of the other two with a leather-covered wooden wheel. The steel truck wheels were completely bare, 2½ in. wide 12 in. in diameter, with a pressure of 880 lb. The axle of each wheel was mounted at an angle of 4 degrees in relation to the corresponding radius of the shaft. In this way each wheel was forced to roll and to slide at the same time. The leather-covered wooden wheels were ordinary cart-wheels covered with first-rate sole leather fastened to the rim with iron brads without heads. The wooden wheels were 3¼ in. wide, 2 ft. 8 in. in dia., with a wheel pressure of 220 lbs. The wheel axle was mounted at an angle of 4 degrees in relation to the corresponding radius of the shaft. The leather-covered wheels were used to reproduce the wear due to workers in medium heavy shoes walking on the floor.

### PREPARATION OF THE TEST SPECIMENS

As mentioned above, the number of the test specimens were limited to 14. In drawing up the test program, it was found desirable to include in the tests the largest possible number of floor types and to give special attention to the floors made of E-cement concrete. Characteristics of the specimens are given in Table 1. Test specimens 1 to 9 were provided with special finish courses placed after the concrete base had hardened. Specimens 10 to 13 were of the one-course type placed in an uninterrupted sequence of operations.

The concrete used for the test specimens 1 and 4 was made in accordance with current Swedish practice, that is to say, it contained relatively fine sand (Fig. 4). This mix was too dry to allow satisfactory compacting. In order to render possible wood-floating and steel-trowelling, water was added by sprinkling the surface. A further characteristic of these

**TABLE 1—CHARACTERISTICS OF TEST SPECIMENS, SURFACE TREATMENT, AND CONSISTENCY OF CONCRETE**

| Test Specimen No. | Cement Grade | Mix by Weight (Cement: Aggregate) | W/C (Gallons per Sack) | Maximum Particle Size (in.) | Consistency of Concrete | Surface Treatment | Notes |
|---|---|---|---|---|---|---|---|
| 1 | Std. | 1:3.3 | 4.74 | 1/8 | moist earth | steel-trowelled | |
| 4 | E | 1:2.7 | 3.38 | 1/8 | " | " | |
| 5 | E | 1:2.8 | 3.27 | 5/16 | " | " | |
| 6 | E | 1:2.8 | 4.05 | 5/16 | plastic, 2¼ in. slump (4 Vebe-degrees | " | |
| 8 | E | 1:2.8 | 4.05 | 5/16 | " | " | surface treated with magnesium fluoride |
| 7 | E | 1:2.8 | 4.05 | 5/16 | " | brushed | |
| 3 | Std | 1:3.5 | 4.05 | 5/16 | stiff, 3/16 in. slump (14 Vebe-degrees) | steel-trowelled | |
| 2 | Std | 1:3.5 | 4.05 | 5/16 | " | " | |
| 9 | Std | 1:1.73 | 3.84 | 1/8 | stiff | " | |
| 10 | Std | 1:7.5 | 5.85 | 1¼ | stiff, 0 in. slump (26 Vebe-degrees) | " | |
| 11 | E | 1:6.15 | 4.30 | 1¼ | stiff, 0 in. slump (35 Vebe-degrees) | " | |
| 12 | Std | 1:6.85 | 4.74 | 2½ | stiff, 0 in. slump (29 Vebe-degrees) | brushed | |
| 13 | Std | 1:5.5 | 3.61 | 5/8 | moist earth | " | |

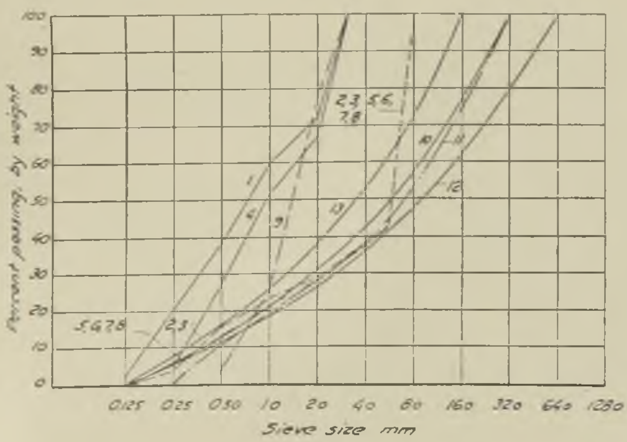



Fig. 4—Sieve analyses of aggregates used in test specimens.

**Fig. 5—Test specimen 4 using E-cement. Mix by weight 1:2.7, maximum size of aggregate 1/4 in., w/c ratio 0.30, moist earth consistency. This photograph shows how the topping was tamped.**

test specimens was the fact that all operations were made in an uninterrupted sequence (Fig. 5).

The test specimens 2, 3 and 5 to 8 were on the whole prepared in accordance with recommended American practice,(5), (10) using a coarser sand and a large percentage of pea gravel (Fig. 4).

In the case of specimen 5, the consistency was extra dry, and the concrete was therefore hard to work. Nevertheless, it was possible to carry out wood-floating and steel-trowelling without sprinkling the surface with water.

In the preparation of specimens 6, 7 and 8 the water content of the mix was increased so as to obtain a slump of 2¼ in. The consistency of this mix was too wet, but was maintained equal in all three specimens to enable the effect of several other factors to be estimated. Specimen 6 was steel-trowelled. Specimen 7 was finished by smoothing the surface with a horse-hair brush. Specimen 8 was steel-trowelled and then treated with magnesium fluoride.

The consistency of the mix used for 2 and 3 was 14 Vebe-degrees*, and the slump was $\frac{1}{16}$ in. This was found to be adequate in several respects; no segregation of water occurred; the workability was good and the floor specimens were easy to make. The only difference between these specimens was that 3 was compacted by a roller weighing 126 lb., whereas specimen 2 was compacted by hand-tamping. However, the roller was not suitable for this purpose. A distinctive feature of the methods of construction used in preparing specimens 2, 3, and 5 to 8 was that the individual operations were separated by intervals of rest which varied from half an hour to 2 hours. Owing to these intervals of rest, the tendency of the cement and very fine aggrgates to be brought to the surface was substantially decreased.

Specimen 9 was an example of "Hard Concrete Finish." The topping consisted of two courses: the lower, or intermediary, course and

*Time required (in seconds) to transform a cone of fresh concrete into a flat cylinder through vibration. See also "Measuring Consistency of Concrete Mixtures", Current Reviews, June, 1946 ACI Jl, p. 727, *Proc.* V. 42.

the upper, or wearing, course. The intermediary course was $\frac{3}{4}$ in. in thickness, and the wearing course $\frac{1}{4}$ in. The mix by weight used for the former course was 1 part of standard cement to 3.0 parts of sand, 0-$\frac{7}{32}$ in. and the consistency was that of moist earth, 3.27 gallons of water per sack. The mix by weight of the wearing course was 1 part of standard cement to 1 part of sand, $\frac{1}{64}$-$\frac{1}{8}$ in. to 0.73 part of hard granular aggregate.

Specimens 10, 11 and 12 represented vibrated concrete floors, and the base slab concrete was finished directly since no special topping was used (Fig. 6). The fact that the construction was easy in spite of the harshness of the concrete and the high percentage of coarse aggregate, is entirely due to the merits of the Vibro-Screed which did excellent compacting work, and produced a surface which was easy to finish by wood-floating and subsequent steel-trowelling.

Specimen 13 was cast in two courses. The lower course was $6\frac{3}{4}$ in. in thickness, and was made of concrete of plastic consistency, while the upper course was $1\frac{3}{16}$ in. thick, and was made of concrete of moist earth consistency. The use of concrete of plastic consistency in conjunction with a superincumbent course of concrete of moist-earth consistency proved to be inadequate since the upper course was very difficult to compact.

Specimen 14 was a limestone slab (Table 1).

The sand and gravel used for the test specimens were dried and separated by screening into fractions. Owing to this special grading, accurate and reproducible aggregate mixtures were obtained.

Fig. 6—Test specimen 10. Vibrated concrete floor. Mix by weight 1:3.2:4.3, w/c ratio 0.51, consistency 26 vebe-degrees. This photograph was taken immediately after wood-floating.

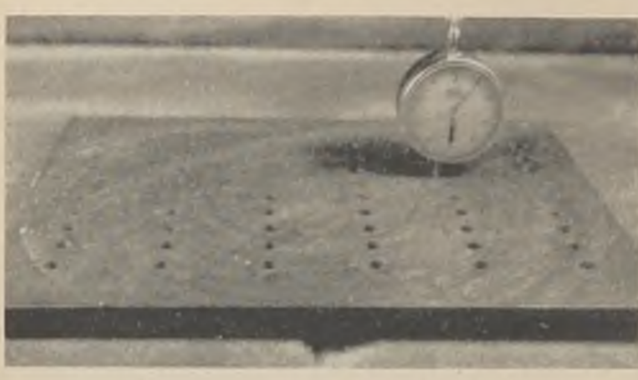


**Fig. 7—Measuring the depth of wear by inserting a dial indicator into a hole in a steel plate.**

The sieve analyses of the aggregate used for the test specimens are shown in Fig. 4. Special attention should be given to the fine sand employed for specimens 1 and 4, the discontinuous drooping curves of the specimens 2, 3, and 5 to 8 which contain a high percentage of pea gravel, and the "harmonic" grading used for specimens 10 to 13.

The concrete mixer was of an Eirich countercurrent type. The time required for each operation was accurately recorded.

All test specimens, with the exception of 2, 3, and 13 were subjected to moist curing under a layer of sawdust for three weeks after placing. Specimens 2 and 3 were moist cured for one week only, and specimen 13 for two weeks. Following the moist curing all specimens were left in the air of the laboratory.

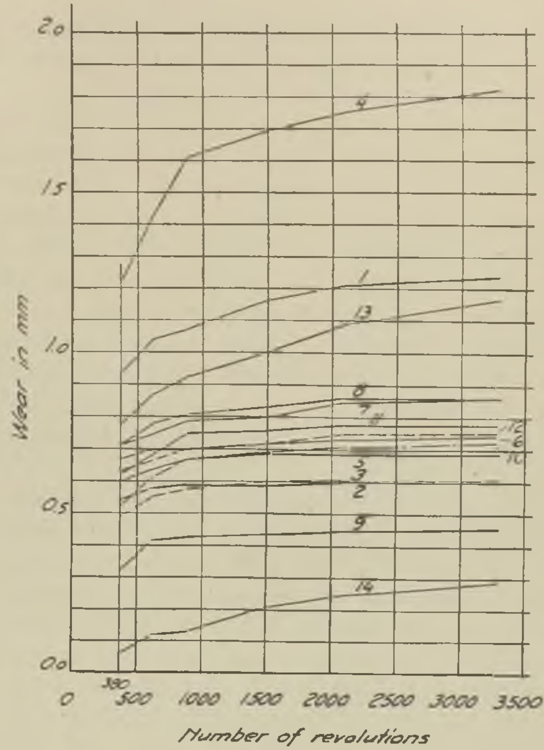
The depth of wear was measured by means of a dial indicator which was applied to the surface of the specimen at definite intervals of time. The dial indicator rested on a reference plane consisting of a steel plate which was fixed by means of three steel plugs embedded in the concrete surface of each specimen. The method of wear measurement is illustrated in Fig. 7.

### TEST PROCEDURE

To begin with, the road testing machine made a total of 3200 revolutions, while the wear was measured from time to time. During this *first test series* the two leather-covered wooden wheels travelled in the outer track, the two rubber-covered driving wheels in the middle track, and the two steel truck wheels in the inner track. It was found that the rubber-covered wheels, which rolled without sliding, did not produce any measurable wear of the concrete surface, whereas the effect of the other wheels was clearly noticeable.

After this first test series the attachments of the steel truck were moved over to the middle track and provided at the same time with new, unused steel truck wheels. These wheels travelled in the middle

Fig. 8—Wear results, series 1. Steel truck wheels.




track simultaneously with the driving wheels. This was done for the purpose of making control tests by means of the steel truck wheels. This *second test series* comprised a total of 1300 revolutions.

Then followed a *third test series*. This series, which comprised a total of 9000 revolutions, was made after the surface of some specimens were subjected to machine grinding in order to remove a surface layer $\frac{1}{16}$ to $\frac{1}{8}$ inch in thickness. Completely new leather-covered wooden wheels, exactly like the others, were used for this test series.

## RESULTS OF THE TESTS MADE IN THE ROAD TESTING MACHINE

In the graphs of Fig. 8 the depth of wear is represented as a function of the number of revolutions of the shaft for test series 1 during which the steel truck wheels travelled in the inner track, and Fig. 9 shows the corresponding results of test series 2 during which the steel truck wheels travelled in the middle track. In the latter test series a particularly large number of observations was made at the beginning.

These graphs show clearly that the wear of the concrete surfaces of specimens 1 to 13 at the beginning of the tests was greater, and sometimes much greater, than later. This implies that the surface skin was less resistant to wear than the concrete located immediately underneath. Furthermore the curves show that the results obtained for the different specimens vary within wide limits.

The character of the curves referred to above explains how dust is formed on concrete floors. The dust obviously comes from the poor surface skin of the concrete and is easily rubbed off the surface by the traffic, regardless of intensity. This assumption was also confirmed by direct observations made in the course of the tests.

For the purpose of dust prevention on concrete floors it is therefore necessary to remove the loose dusting surface skin in some suitable manner. However, in order to obtain a first-rate floor in this way, the concrete located immediately below the surface skin must, of course, offer greater resistance to wear than this skin. In practice this is often *not* the case, and then it is impossible to improve a poor floor by removing the surface skin.

The surface skin may be removed in many different ways. During construction, the surfaces of several test specimens, such as 7, 12, and 13, were brushed with a horse-hair brush in order to remove scum and laitance formed on the surface. Any favorable effect of this treatment was not noticed during the tests. In the case of one of the test specimens an attempt was made to remove the surface skin by acid treatment, but the result was still worse because the acid penetrated below the skin and decreased the strength of the concrete. On the other hand, the tests showed that machine grinding resulted in a marked improvement of the surface.

It may be seen from Fig. 9 that the thickness of the loose surface skin varied from about $\frac{1}{64}$ to $\frac{1}{16}$ in., depending on the specimen. Before a skin of this thickness is worn off in practice, great quantities of dust will have formed. If all values of wear obtained in the first test series (Fig. 8) are plotted after deduction of the values corresponding to 625 revolutions, the graph shown in Fig. 10 will be obtained. The curves are plotted to the same scale as in the previous graphs. As may be seen from this figure, the wear is considerably reduced after 625 revolutions.

An examination of the comparative quality rating of the test specimens, as represented in Fig. 8, shows that the difference between them is very great. The worst specimen is 4 which is provided with a finish course containing only fine sand and E-cement of moist-earth consistency which represents a concrete floor of the ordinary type much used in Sweden during the past few years.

The worst specimen next to the foregoing is the corresponding specimen 1, made of standard cement. Then follows specimen 13 made of combination concrete. It may seem surprising that this specimen did

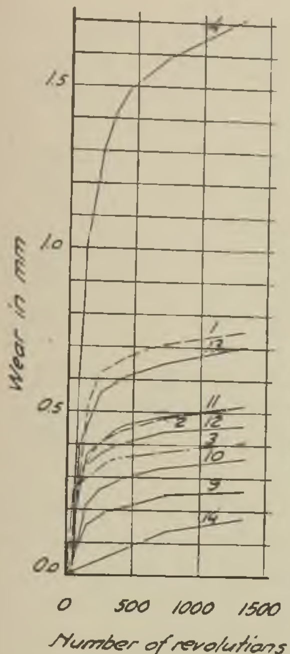



**Fig. 9 (left)—Wear results, series 2. Steel truck wheels.**

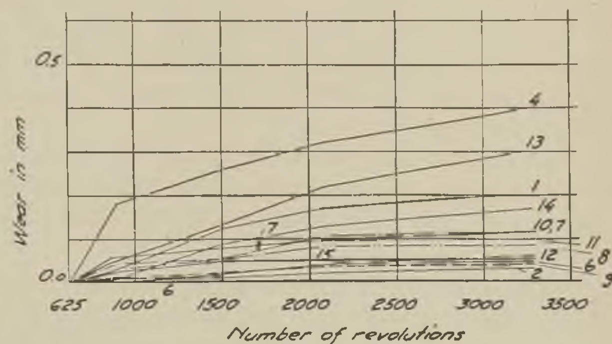



**Fig. 10 (above)—Wear results, series 1, after 625 revolutions. Steel truck wheels.**

not turn out better, but this is due to the fact that the cement content in this case was much lower than in the foregoing specimens, and that proper tamping of the surface of the specimen proved impossible because the concrete base was too wet. An intermediate group without any notable variations in comparative rating comprises the specimens provided with a separately placed wearing course containing a high percentage of pea gravel (special attention is directed to the fact that both standard cement and E-cement specimens belong to this group) and the specimens made of vibrated concrete, were also without any notable differences between standard cement and E-cement specimens. Among these specimens is also specimen 8 which is provided with a wearing course of E-cement concrete containing pea gravel, and which was treated with magnesium fluoride. This specimen is rather worse than the corresponding specimen 6 which was not treated with magnesium fluoride. This may be regarded as a pure chance occurrence.

The fact that an E-cement concrete floor *can* be as good as a corresponding floor made of standard cement concrete, may be explained by two circumstances, first the slightly higher cement content of all specimens of the former type and second the complete removal of sand below $\frac{5}{512}$ in. in grain size. Nevertheless, this fact is remarkable in any case. It shows that good floors may well be made of E-cement concrete provided that they are constructed in the right way.

In order that a fair comparison should be made between the specimens provided with a separately placed finish course and the specimens made of vibrated concrete, it must be borne in mind that the cement content of the former was only 60 percent of the cement content of the latter.

The graph shows that the best concrete specimen was the specimen provided with a topping of "hard" concrete. The loose surface skin on this specimen was very thin, but it was present, nevertheless.

The graph shows, furthermore, that the limestone specimen proved to be superior to all concrete specimens, but this is due to the inferior surface skin of the latter. If the surface skin is removed, the comparison is partly reversed.

The comparative rating is on the whole the same in Fig. 9 which represents the values obtained from a similar series of tests, that is series 2, during which the steel truck wheels travelled in the middle track.

An examination of the comparative rating of the specimens according to the depth of wear *after 625 revolutions* (Fig. 10) shows that the ordinary E-cement floor specimen 4 has given the worst results also in this case. Next follows specimen 13 made of combination concrete, and then the ordinary standard cement floor specimen 1. The wear of all other specimens was very small. It is to be observed that the limestone specimen is no longer the best. On the contrary, it is worse than most other types, and even worse than some specimens containing E-cement. Hence it follows that, apart from the weak surface skin, a first-rate concrete floor offers a greater resistance to wear than a floor made of limestone slabs. According to this graph the best specimens are now the specimen made of "hard" concrete and the specimen provided with a separate wearing course of sand, pea gravel and standard cement (Specimen 2). This graph is a convincing proof of the fact that first-rate floors can also be made of E-cement concrete since the curve for specimen 6 almost coincides with the curve for the "hard" concrete specimen.

The limestone specimen curves in the graphs are not rectilinear because the steel truck wheels were gradually polished by wear with the result that the wear of the surface of the specimen became less heavy.

Fig. 11 shows some results obtained from test series 3 during which wear was measured on the surface of the specimens after a layer of $\frac{1}{64}$ to $\frac{1}{16}$ inch in thickness had been removed by machine grinding. The results refer to the wear tests made by means of the steel truck wheels. It may be seen from this graph that the concrete floor specimens made in accordance with usual Swedish practice provided with finish courses of E-cement or standard cement and fine sand form an especially poor quality group. This is due to the fact that the mortar was much too dry during the placing of the specimens. Therefore, it was impossible to

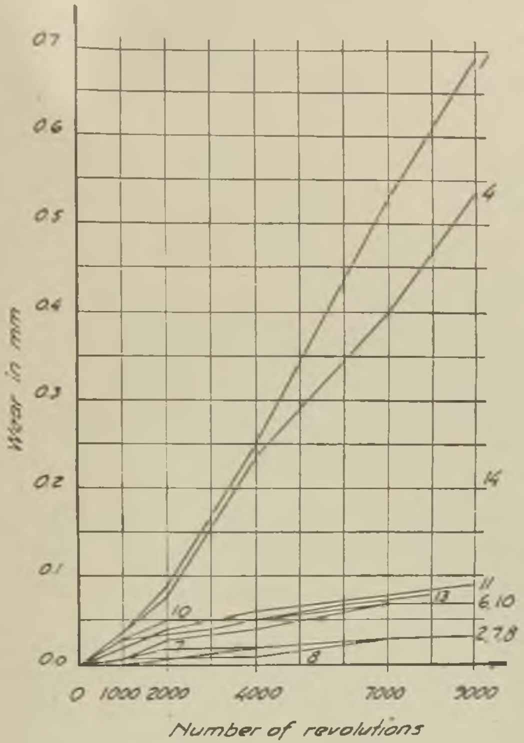



Fig. 11—Wear results, after the removal of $\frac{3}{32}$ in. to $\frac{1}{8}$ in. by grinding. Steel truck wheels.

compact the mortar so as to obtain proper density. It was only the surface that was rendered dense because it was sprinkled with water during wood-floating, but the concrete immediately below the dense surface remained porous, and it is this porous concrete that accounts for the great depth of wear.

Fig. 11 shows very clearly that the wear can be reduced to a minimum by removing the loose surface skin by machine grinding. However, grinding is likely to give good results only in those cases where the concrete located immediately below the skin is of first-rate quality. In practice the floors may be ground without any considerable additional expense, by means of grinding machines designed specially for this purpose.

Fig. 12 shows the depth of wear made by the leather-covered wooden wheels as measured during the tests. The relative order of the curves is in part reversed as compared with those of the foregoing figure since the position of specimen 1 has improved, while that of specimen 7 ranks lower. This may be due in part to accidental circumstances and in part to the different behavior of the concrete surfaces in the cases of light and heavy traffic. Light traffic results in uniform wear, while heavy

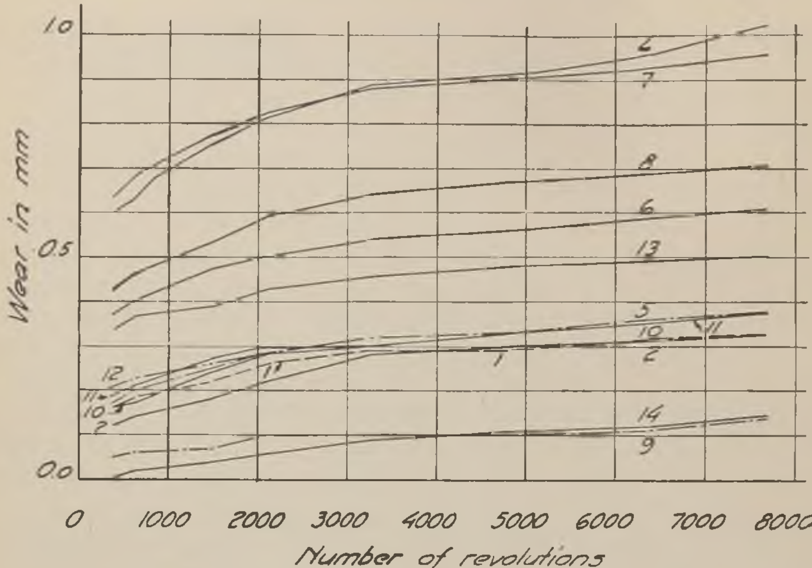



Fig. 12—Wear results. Leather-covered wooden wheels.

traffic produces wear in conjunction with local defects due to crushing and impacts. In these tests, however, which represented light traffic, it was for the most part the weak skin that was slowly worn off the surface of all specimens, and this skin cannot be regarded as representative of the quality of a floor in the long run. Series 3 included also a number of measurements of the wear produced by leather-covered wheels on those surfaces where the skin was removed by grinding, and these measurements result, on the whole, in the same comparative rating as in the steel truck wheel tests, except that the expected comparative rating of specimens 1 and 4 was obtained in this case (Fig. 13).

## RESULTS OF VISUAL INSPECTION OF THE TEST SPECIMENS DURING AND AFTER THE TESTS

As a general estimate of all test specimens, with the exception of 1 and 4, it may be stated that they withstood the tests very well. In the beginning, before the layer of laitance was worn off, profuse dusting was observed, but as wear proceeded, dusting decreased, and after a relatively short testing time dusting was considerably reduced on most specimens.

In addition to pure wear, other types of deterioration were observed on several test specimens. Pitting occurred in particular on specimen 4, and also on specimen 1. The pitted areas on these specimens formed obvious weak points which gave rise to heavy deterioration during continued testing. On those test specimens which were not provided with

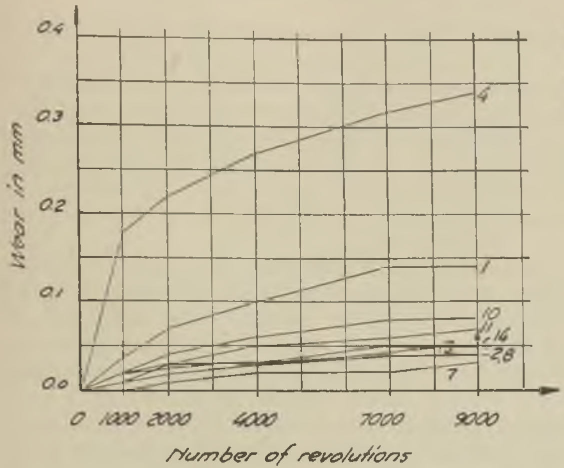



Fig. 13—Wear results, after the removal of 3/32 in. to 1/8 in. by grinding. Leather-covered wooden wheels.

Fig. 14—Test specimen 11, series 1, after 3200 revolutions. Steel truck wheels. Vibrated E-cement concrete floor. The coarse aggregate is laid bare by the crushing of top mortar layer.

any finish course, the steel wheels crushed the shell of cement mortar that covered the particles of coarse aggregate located near the surface (Fig. 14).

## PROPOSED SPECIFICATION FOR CONCRETE FLOOR FINISH

The quality of a concrete floor depends primarily on correct and accurate construction. In view of the fact that it is almost impossible to improve a poor concrete floor after completion, it is extremely important to make certain that the floor is correctly constructed from the beginning. Therefore the authors present for consideration the following specification for the construction of concrete floors, for the main part in accordance with American recommendations.

1. If possible, the floor topping shall be placed simultaneously with the placing of the the base slab.
2. If this is not practicable for some reason or other, so that the floor topping is placed after the concrete base has already hardened, the surface of the base slab shall be thoroughly cleaned prior to applying the topping. No later than one day after placing, the surface of the base slab shall be brushed with a steel wire brush so as to remove all laitance and to lay bare the uppermost particles of coarse aggregate. It is important that this be done no later than one day after placing since it is then easier to clean the surface.
3. Before placing the wearing course, the surface of the base slab shall be kept thoroughly wet for at least 3 days. The concrete of the base slab should be saturated with water so that its expansion after placing the topping will be negligible. In this way the stresses due to shrinkage of the topping will be reduced, and this is important since these stresses give rise to cracking and disruption of bond if they become too high.
4. Immediately before applying the wearing course, a thin coat of 1:1 cement-sand grout shall be broomed into the surface of the base. The surface shall not be excessively wet and must have dried several hours so as to become absorbent. Absolutely dry sand shall be mixed with cement, dry so that no setting reactions can occur if the mixture is left to stand for some hours. This mixture shall then be applied to that part of the floor which is to be treated shortly, and shall then be broomed into the surface with a stiff broom, water being added at the same time with a watering-can. The cement grout formed in this manner shall be thoroughly broomed into the surface so that no portions of the surface remain bare. The coat of cement grout covering the surface shall be very thin. The cement grout shall not be applied so early that the surface of the base can become dry before the topping course is placed, that is, it shall be applied no earlier than half an hour prior to the placing of the topping course.
5. The mix proportions of standard cement to aggregate shall be from 1:3.5 to 1:4 by weight. The aggregate shall contain sand and pea gravel not exceeding $\frac{11}{32}$ to $\frac{13}{32}$ in. The ratio of sand smaller than $\frac{5}{32}$ in. to pea gravel shall be from 1:1 to 1:1.5 by weight. The water cement ratio shall be determined with a view to insuring good workability of the concrete, i.e. a slump of about 7/8 inch. The grading of the sand is of less importance provided that the sand is neither extremely fine nor coarse. A recommendable sand grading curve is the straight line that connects the point 3 percent for $\frac{5}{1024}$ in. mesh to the point 100 percent for $\frac{5}{32}$ in. mesh in the logarithmic grading chart. In those cases where the wearing course is compacted by vibration, the mix proportions of standard cement to aggregate should be changed to 1:4.5 to 1:5 by weight, whereas the ratio of sand to coarse aggregate may remain as is specified in the above.
6. The topping shall be placed as usual so as to obtain a finish thickness from 1 to 1¼ inches. The topping shall be brought to grade with a strike-off board guided by laths placed and accurately set beforehand. The topping shall first be screeded to a level about $\frac{3}{32}$ inch above the finish floor grade. After the topping has been struck off for the first time, it shall be compacted with a tamper. Compacting shall be done thoroughly so as to insure that the concrete is consolidated into a dense mass throughout. After compacting the surface shall be struck off to the finish grade. The surface shall then be wood-floated. Floating shall be reduced as much as possible in order to prevent excessive quantities of fines from being brought to the surface. After wood-floating the surface shall dry until no excess water is left, so that the surface feels just damp when it is wood-floated for the second time. Immediately after second wood-floating the surface shall be steel-trowelled.

7. If sunlight is likely to strike upon the floor surface, it shall be protected by canvas.
8. After the floor has hardened one day, it shall be subjected to moist curing which shall continue at least one week, but two or three weeks if possible.
9. If the floor surface is to be finished by machine grinding, it should be done after the surface has hardened sufficiently, that is, after one week at least. Too early grinding may result in dislodgment of coarse aggregate, deteriorating the floor.

## BIBLIOGRAPHY

(1) "Arbetsbeskrivning för beläggning av cementbetong," Kungl. Väg- och Vattenbyggnadsstyrelsen, Stockholm 1939. ("Specification for Cement Concrete Pavements and Floors." Royal Swedish Board of Roads and Waterways, Stockholm 1939.)

(2) "Arbetsbeskrivning för tunn beläggning av cementbetong på bärigt underlag," Kungl. Väg- och Vattenbyggnadsstyrelsen, Stockholm 1939. ("Specification for Thin Cement Concrete Toppings on Load-Bearing Base Slabs," Royal Swedish Board of Roads and Waterways, Stockholm 1939.)

(3) "Good Practice in Concrete Floor Finish." Report of Committee 802. JOURNAL of Am. Concrete Inst. *Proc.* 1930, p. 520.

(4) Schuman, L. and Trucker, J.: "A portable Apparatus for Determining the Relative Wear Resistance of Concrete Floors." Journ. of Research of the National Bureau of Standards. Vol. 23, No. 5, Nov. 1939.

(5) "Concrete Manual," United States Department of the Interior, Bureau of Reclamation, Denver, Colorado, 1939, p. 252.

(6) Scripture, E.: "Metallic Aggregate in Concrete Floors," JOURN. of Am. Concr. Inst., Sept.-Oct. 1936.

(7) "Concrete Wearing Surfaces for Floors." Report of Committee 804. JOURN. of Am. Concr. Inst. Sept. 1938.

(8) "Roughing Floor Slab to Bond Finish?" JOURN. of Am. Concr. Inst. Febr. 1939.

(9) "How to Dustproof a Concrete Floor?" JOURN. of Am. Concr. Inst. Nov. 1940.

(10) "Finishing of Unformed Surfaces." Manual of Concrete Inspection. Am. Concr. Inst. 1941.

(11) Kullberg, G.: "Statens Väginstituts provvägsmaskin," Statens Väginstitut. Medd. 69, 1944. ("Road Testing Machine of the Swedish Road Institute," Swedish Road Institute, Report No.69, 1944).

(12) Wästlund, G. och Gedda, S.: "Några avnötnings- och hållfasthetsprov på betong för beläggningsändamal." *Cement och Betong*, hafte 4, 1939. ("Wear and Strength Tests on Concrete for Pavements and Floors," *Cement och Betong*, hälfte 4, 1939.)

(13) Roelofsen, E.: "Beschouwingen over speciale betondeklagen voor bedrijfsvloeren en daarmede samenhangende vraagstukken." *De Ingenieur* no 45 Bt. 1937.

(14) Guttmann, A. und Seidel, K.: "Ueber die Druckfestigkeit, Stossfestigkeit und Abnutzbarkeit von Beton." *Zement*, S. 233, 1936.

(15) Grün, R.: Abschnitt "Chemie der Behandlungsmittel." Chemie für Bauingenieure und Architekten, S. 90, Berlin 1942.

(16) Granström, S. O.: "Betonggolv. Några nya gjutningsmetoder." Stockholm 1944.

(17) Jackson, F. H. and Pauls, J. T.: "Wear of Concrete Pavements Tested." *Public Roads*, May 1934.

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# Current Reviews

## of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

### Reinforced concrete marine forts and floating docks

*Concrete and Constructional Engineering*, V. 41, No. 4 (Apr. 46), pp. 97-106

Reviewed by GLENN MURPHY

This article consists of an abstract of a paper read before the Royal Society of Arts by G. A. Maunsell in which he described some unique examples of reinforced concrete construction. Included are descriptions of a series of forts constructed as cylindrical towers and planted in the open sea and a series of floating docks intended to be towed over sea. Photographs and detailed drawings of the precast ribs and wall slabs for the floating dock units are included.

### Krazen Bridge, Switzerland

*Concrete and Constructional Engineering*, V. 41, No. 4 (Apr. 46), pp. 107-108

Reviewed by GLENN MURPHY

This article describes an open spandrel simple arch with a span of 442 ft. The two arch ribs are 7 ft. 8 in. deep by 5 ft. 2 in. at the crown and 14 ft. 2 in. deep by 8 ft. 6 in. wide at the springs. Vibrated concrete developing a cube strength of 6400 psi, at 28 days was used throughout the structure. Chrome steel having a tensile strength of 82,500 psi. and a yield point of 54,500 psi. was used for reinforcement. Working stresses up to 26,000 psi. were used in the steel.

### A new insert for expansion joints in concrete surfacings

G. Von Seggern, *Strassenbau*, 1942, 33 (21/22), 121-2. *Road Abstracts*, XIII, No. 6, June 4, 1946.

HIGHWAY RESEARCH ABSTRACTS

A description is given of a tubular expansion joint insert of stainless steel with closed ends which is claimed to be more capable of adaption to slab movements than the wooden insert generally used. The insert occupies only the lower part of the joint; the upper 1.6 in. is filled with bituminous filler. The dowels, hooked at alternate ends, are welded to the insert, thereby causing it to adhere more closely to the concrete and prevent percolation of the filler.

### Repair of reinforced concrete damaged by fire

*Concrete and Constructional Engineering*, v. 41, No. 4 (Apr. 46), pp. 109-111

Reviewed by GLENN MURPHY

This article calls attention to two recent publications issued by the Building Research Station of the Scientific and Industrial Research which discussed fire damage and repair of reinforced concrete construction. Studies indicate that fire damage begins in concrete at temperatures between 200 C and 300 C. The color of the burned concrete serves as a general index of the fire damage. The Building Research Station recommends that repairs may be effected satisfactorily if not more than one quarter of the reinforcement is exposed. They suggest that it is not worthwhile to repair a slab which has a sag more than about 1 percent of the span nor one in which the cracks appear through the full thickness of the slab. Details of repair are also recommended.

### Studies of slab and beam highway bridges

Bulletin No. 363, "Part I. Tests of Simple-Span Right I-Beam Bridges," by Nathan M. Newmark, Chester P. Siess, and Robert R. Penman, Engineering Experiment Station, University of Illinois.

Release of Univ. of Ill. Exp. Station

Laboratory tests made on fifteen I-beam bridges are reported. The purposes of these tests were: (1) to compare measured strains at various points on the structure with values computed from theory; and (2) to determine the ultimate capacity of the bridges and their manner of failure.

These tests were intended to supplement the analytical studies reported in Bulletin No. 336, "Moments in I-Beam Bridges." According to the authors, the effect on the behavior of the structure of the discrepancies in the assumptions upon which the analysis is based is in general of importance only quantitatively. The qualitative picture of the action of the I-beam bridge that is given agrees very well with the observed behavior; that is, the relative importance of the variables, the effects of changes in proportions, the locations of the maximum moments, the manner of initial failure, all are indicated faithfully by the theory.

The test results are presented in detail in the new Bulletin No. 363, and comparisons are made with theoretical calculations in practically all cases. Copies of the publication will be mailed free on request for a limited period from the Engineering Experiment Station, University of Illinois, Urbana, Illinois.

### Less sand and water in air-entraining batches improves concrete

Myron A. Swayze, *Civil Engineering*, V. 16, No. 7, July 1946, pp. 301-303

Reviewed by J. R. Shank

The author makes a number of statements concerning the action of air-entraining cements and agents added at the mixer, and discusses the subject generally as in a text book. He cites a number of construction jobs where air-entraining agents were used. A few of the statements are: "More workable, more durable concrete can be obtained with mixes containing less water and less sand—and an air-entraining cement, or by adding an air-entraining agent at the mixer." "This concrete, with its tiny air cells, is more resistant to frost action and salt action and less permeable to water." "When concrete containing air is placed and finished, a notable feature is the greater cohesion of the paste, resulting in decreased segregation of the solid components of the mix. Separation of water at the top is materially reduced. Finally, the hardened concrete has much improved resistance to the action of frost, and to the action of salts commonly applied for removing ice from the surface." and "In the laying of concrete pavements, floor slabs and such, there is decidedly less tendency for separation of mortar and coarse aggregate in the screeding operation." He calls attention to reduction of bleeding and gives bases for designing air-entrained concrete, both for ground-in cement air-entrainers and those added at the mixer.

### Use of air entraining concrete in pavements and bridges

Current Road Problems Bulletin 13 (May, 1946) Highway Research Board

Reviewed by R. A. Marr, Jr.

This bulletin discusses the application of the principle of air entrainment to the problem of increasing the durability of structures such as pavements and bridges which are subjected to weathering. Certain substances added in very small quantities to concrete introduce into the mass a larger amount of air than is found in ordinary concrete. This "entrained air" appears to exist in the form of minute disconnected bubbles well distributed through the mass. Their presence materially alters the properties of both the plastic mixture and the hardened concrete. For example, air-entraining concrete is considerably more plastic and workable than ordinary concrete. Segregation in placement and water-gain after finishing are both reduced. Resistance of such concrete to alternate freezing and thawing is markedly improved. The ability of air entraining concrete pavements to resist scaling due to the use of salt or calcium chloride for ice removal has been demonstrated by both laboratory tests and field observations in service. The entrainment of air tends to reduce the strength of the concrete. At the present time it is generally agreed that, for a properly designed mix, a total air content of from 4 to 5 percent by volume will give a satisfactory improvement in durability without serious loss in strength. However, any marked increase in entrained air above the amounts recommended will still further decrease the strength of the concrete without commensurate improvement in durability.

Methods of securing air entrainment by the use of special air entraining cements, as well as by the addition of air entraining materials at the mixer are described and the advantages and disadvantages of each method discussed.

The bulletin concludes with a discussion of field problems incident to the use of this principle with special reference to ready mixed concrete.

Appendices include specifications for air entraining cements and for aggregates, instructions for using certain proprietary admixtures, a suggested procedure for evaluating air entraining admixtures, and a method of determining the air content of concrete.

**The resistance of road surfacings to tank traffic**

A. R. Collins and D. B. Waters, Road Research Laboratory, Department of Scientific and Industrial Research, England. Road Research Laboratory, Harmondsworth, West Drayton, Middlesex.

Highway Research Abstracts

This report describes investigations into the resistance of road surfaces to wear by intense tank traffic. Both concrete and bituminous surfacings have been investigated, using special testing methods, including a tank wear testing machine and an indentation test.

With concrete surfacings the results show that: (1) The rate of wear increases rapidly with increase of intensity of pressure between the tank track and the road. (2) The rate of wear in the initial stages is governed mainly by the crushing strength of the concrete, irrespective of the age and water-cement ratio. The mix proportions appear to have a minor effect. (3) The initial rate of wear of concrete with a crushing strength of 2,000 lb. per sq. in. is about five times that of concrete having a strength of 4,000 lb. per sq. in., but above 4,000 lb. per sq. in. the increase in wear resistance is not very great. (4) The type of aggregate used has an important effect on the later stages of wear of medium and low strength concrete, best results being given by crushed igneous rocks and worst results by flint gravels. The type of aggregate has, however, little effect on the wear resistance of concrete having a strength of more than 6,000 lb. per sq. in.

Some results of a full-scale road test are included and these corroborate the main conclusions. They show that concrete having a crushing strength of more than 4,000 lb. per sq. in. gives satisfactory service. They also emphasize the importance of joint design and construction in concrete roads.

With bituminous surfacings the results show that: (5) The best resistance to wear is given by mastic asphalt. Recently developed dense tar surfacing and rolled asphalt also give good results. The binder content has a marked influence on the resistance to wear; there is an optimum value above and below which wear increases. (6) In the case of the dense tar surfacing, the resistance to wear increases with increase in viscosity of the tar binder. (7) Resistance to wear of rolled asphalt is increased with increase in filler content. (8) To avoid undue indentation of mastic asphalt by standing tanks, the grit content of the mastic should lie between 45 percent and 50 percent., using an ungritted mastic containing 13 to 15 percent of 30-40 penetration bitumen and a limestone powder having 45 to 60 percent passing 200 mesh. (9) Within the limits ¾-in. to ⅛-in., the size of the grit added to the mastic makes no significant difference in the resistance to indentation.

Recommendations are given for the construction of tank roads in concrete and bituminous material.

### Permeability of concrete

L. Boyd Mercer, *Commonwealth Engineer*, July and August, 1945. Reviewed by L. P. Witte

Factors influencing the permeability of concrete are discussed and the theory on which the coefficient of permeability of concrete is determined is analyzed; with the influence of each of the many factors such as surface tension, weight of water, diameter of tube, being considered in this analysis. Completing this study the author surveys the laboratory test methods employed in the United States and other countries.

The apparatuses used by numerous laboratories are described in detail and a number of procedures used are finally separated into the so-called bomb method where the water is supplied into the center of specimen under pressure; the porous pot method in which the permeability is measured through the walls of a container; the input method where the amount of water under head is measured; and the output method where the quantity of discharged water is determined. From this study of the methods of preparing the specimens for test and measuring the flow of water during test, the author decides upon a test specimen 1½-in. thick, cut from a 6-in. diameter by 12-in. high cylindrical test specimen. This 1½-in. thick disk is considered as being thin, but the author feels that through such a thin disk rapid test results can be obtained which are very reliable. The rate that water passes through the disk under a head of 100 lb. psi. was automatically recorded by means of a pluviograph.

The author concluded from his tests that there was a marked difference in the permeability of disk cut from top of a 6-in. x 12-in. specimen as compared to those 3-in. or 4-in. down from the top. His results also show a very marked difference between the permeability coefficient of specimens cast and tested in a vertical direction as compared with those cast horizontally but tested for permeability in a vertical direction. He explains this difference as being due to the voids which occur under the aggregates due to settlement and water gain while the concrete is still plastic. He feels that the results reported by others regarding the effect of water cement ratio and maximum size of aggregate on the permeability of concrete are correct, and points out that the coefficient may vary from approximately unity for mortars with water cement ratio of .45 to about 1,500 for 9-in. maximum aggregate with water cement ratios of .90. This conclusion is based mainly on the results of tests made by the Bureau of Reclamation.

The author finally concludes that the problem of designing water tight concrete is not so much a problem in the design of proper mixture as one of handling and placing the material in its plastic state so that segregation does not occur and so that the flow through construction planes and joints are reduced to a minimum and that other conditions for water tight construction will be met. He contends that the only permeability result that has any real value in connection with a water retaining structure made of concrete is that which is made upon the structure as a whole.

### Statically indeterminate structures

LAWRENCE C. MAUGH, 1st Ed., 338 pages, John Wiley & Sons, New York: $5.00

Reviewed by HALE SUTHERLAND

Professor Maugh (University of Michigan), has given us a well written mature work, closely knit throughout, which assembles in compact space the modern theories and methods of dealing with indeterminate structures, that is, chiefly by successive approximations. Each chapter concludes with several references to advanced treatises and to noteworthy research papers. A brief appendix (13 pages) presents a series of diagrams for the determination of the fixed-ended moments and coefficients for beams with variable moments of inertia; these include the well known charts in the Portland Cement Association Bulletin S.T. 41. The discussion throughout is illustrated by many well chosen examples with numerical solutions. Eighty-one problems are given, which greatly aid the teacher who plans to use this valuable text. Competency, compactness and refreshing originality are its notable characteristics.

In the first three chapters are given principle and practice relative to beam and truss deflection and to stress determination by the slope deflection and moment distribution methods. Despite evident attempt at brevity it is noteworthy that the reciprocal theorem is given in relatively general form rather than the more familiar $\Delta_{ab} = \Delta_{ba}$, that the application of the Williot-Mohr diagram to the determination of the relative displacements in quadrangular frames is shown, and that the effect of shearing deformation in moment distribution calculations is discussed. The brief fourth chapter gives the application of moment distribution to building frames with vertical loading and provides a sound basis from which to proceed to study of detailed rules and design procedures. Here the reader is introduced, though without his knowledge, to the first step essential in the methods now being sought for replacing successive approximations by a single distribution at each joint with simultaneous relaxation of all joints. He also learns very briefly about fixed points.

In chapter five multi-storied frames with vertical and with sloping columns are analyzed, first, by moment distribution using auxiliary restraining force systems at the joints, and, second, by the panel method. Iteration is presented for the avoidance of direct solution of a series of simultaneous equations. The chapter concludes with the application of moment distribution to the determination of secondary stresses in trusses. The extension of slope deflection and moment distribution to frames with members of variable moment of inertia follows in the next chapter. The principal illustrations deal with a three-span rigid frame bridge.

The somewhat routine problems of continuous trusses and bents are enlivened in chapter seven by the presentation of the case of the bent with stepped columns supporting crane girders at intermediate level.

Following very brief conventional treatment of unsymmetrical two-hinged and hingless arches, the eighth chapter proceeds to apply moment distribution to curved members as they appear in single span and continuous bents. The fixed arch equations, using as redundants the reaction elements at one support, are applied to stiff rings, closed frames and to fuselage frames for aircraft. The simplification of the arch equations through use of the elastic center is also considered. After dealing with the basic cases of tension and compression members subjected to transverse loads, the next chapter gives a brief treatment of the suspension bridge with a practical note on the flexible arch. The final chapter deals briefly with frames with semi-rigid connections, with space frames and with shearing stresses in thin-walled closed sections.

### Heavy loads on bridges

*Roads and Road Construction*, V. XXIII, No. 269, May 1, 1945, London.

HIGHWAY RESEARCH ABSTRACTS

In the 31st annual report of the Country Roads Board of Victoria, Australia, reference is made to the necessity for allowing unusually heavy loads to cross bridges of various types, and to the general precautions specified in the permits. The conventional Class A loading used in design on rural main roads consists of a vehicle with a gross weight of 37,500 lb. distributed over two axles 12 ft. apart and with wheels at 6 ft. centers, the rear axle carrying 25,000 lb. This load is applied on every 10-ft. traffic lane, and is preceded and followed by a uniformly distributed load of 100 lb. per sq. ft. An impact factor of 1.3 is used in design.

Heavy loads investigated have been carried on specially designed vehicles fitted with pneumatic tires with good articulation of wheels and axles, giving very good distribution of the concentrated loads. Calculations have been made by the Board's bridge engineers and checked in some instances by deflection and stress measurements. The following details relate to particular types of bridges.

On concrete slab bridges measured deflections were always much less than calculated, due partly to a probable under-estimation of the modulus of elasticity of the concrete, and also to the effect of lateral distribution. The same is generally true of concrete tee-beam bridges, except that the lateral load transfer effected depends on the relative stiffness of the beams and deck, and this needs consideration in each case. Also near piers and abutments, where little lateral load transfer can occur, shear may be a governing factor. This is especially true of the older concrete structures constructed prior to about 1920, the early designs being deficient in shear reinforcement.

On timber stringer bridges, deflections observed agreed fairly well with calculated values, due allowance being made for transfer across the stringers. The amount of such transfer varies considerably, as in some cases the decking, and in more recent design the cross beams, may be continuous across the width of the structure, and in other cases the individual pieces may only extend for half the width. On steel girder and truss bridges the deflections observed agreed closely with calculated values after correction for distribution across the structure.

One of the investigations made concerned the passage of a load of 81.6 tons across Lynch's Bridge. This structure contains five main spans of 70 ft., each span consisting of six rows of steel plate girders made integral with the concrete deck by steel stirrups welded to the top flange of the girder, the plate girders being propped up at the third points during the casting of the deck, so as to keep the dead load of the latter off the girder, and to allow for the composite concrete and steel sections to take dead load forces as well as those from live load. The lines of girders are 8 ft. apart, the depth of the web plate is 54 in., and the deck is 8½ in. thick, with a 6-in. haunch over the top flange of the girders.

The special load and the tractor hauling it comprised altogether a train of seven axles, the distances between them being 18 ft., 25 ft., 4 ft. 3 in., 32 ft., 4 ft. 3 in., and 4 ft. 3 in. The front axle with only two tires carried 1.6 ton; the second axle with four tires, 9.5 tons; the third and fourth axles with four tires, 12.25 tons per axle; and the three rear axles or rows of wheels, carrying a combined load of approximately 46 tons comprised four wheels with dual tires on each, the center pair of wheels being 3 ft. apart center to center, and the similar distance from the outer wheels being 3 ft. 9 in. The whole group of twelve wheels at the back of the load was fully articulated.

Several passages of this vehicle with its load were made over the structure on different occasions, deflection tests being made on the first occasion and extensometers being used on later occasions. The maximum deflection under the two central main girders was 0.18 in. compared with a calculated deflection of 0.19 in., assuming a modulus elasticity of steel ten times that of concrete. The measured stress in the lower flange of the girder was 4,000 psi. compared with a calculated stress of 4,100 psi. The stresses in the composite section due to this exceptionally heavy and relatively concentrated load were actually much less than those produced by the class A design loading. This was no doubt due to the good distribution of the isolated load across the structure, and to the practically complete absence of impact.

The deflections of the various girders indicated that 30 percent of the total live loads was carried by each of the two central girders, 15 percent by those next to them, and 5 percent by the the outer girders. It was estimated that approximately 50 percent of the transfer of load across the structure was made by steel across frames which exist at the center of each span, the balance being transferred through the very stiff concrete deck. Calculations indicate that this transfer of load between girders by means of the deck involved transverse stresses in the deck approximately 10 percent above those caused by full class A loading.

The tests made on this bridge provide a useful verification of the assumptions made in the design of the comparatively long span composite steel girder and concrete deck type of construction. A small permanent set was observed after the first passage of the heavy load, but this was considered to be due to slight slips occurring between the steel and concrete sections of the composite structure, while adjusting itself to the load, the structure being unable to return to its original position because of friction between the various parts. This conclusion was supported by the fact that considerable noise occurred during the first crossing, due to structural adjustment, whereas in subsequent crossings there was very little noise.

From this and other deflection observations it is concluded also that with multi-wheeled bogies, well articulated and equipped with pneumatic tires, impact can be neglected.

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# ACI News Letter

Vol. 18 No. 2 JOURNAL of the AMERICAN CONCRETE INSTITUTE October 1946

## In the foregoing Proceedings pages of this Journal you will find, this month:



## News Letter Contents this Month

# REPORT OF THE 1946 NOMINATING COMMITTEE

The 1946 ACI Nominating Committee; A. J. Boase, Chairman, Frank E. Richart, Roy N. Young, Lewis H. Tuthill. P. J. Freeman, D. E. Parsons, Roy W. Crum, Morton O. Withey report the following nominations for offices whose terms expire at the 1947 convention:

**President**

STANTON WALKER, Engineering Director, National Sand & Gravel Association and of the National Ready Mixed Concrete Association, Washington, D. C.; an ACI Member since 1921; member of Standards Committee since 1937, and of the Advisory Committee, as Chairman Department 200, General Properties, from 1938. Became Chairman of the Advisory Committee, February 1945. Member Board of Direction since 1940 when he became Director Fourth District by appointment to fill a vacancy and twice elected; elected Director-at-Large for a three-year term, 1943 and Vice-President, 1945. Re-elected Vice-President, 1946.

**Vice-President** (to succeed himself)

ROBERT F. BLANKS, Chief, Engineering and Geological Control and Research, Bureau of Reclamation, Denver, Colo.; ACI Member since 1932; member Publications Committee since 1941; Chairman since February 1945; Member Board of Direction as Director Sixth District 1941-42; appointed by Board March 1944 to be Director-at-Large to fill a vacancy and re-elected February 1945; Elected Vice-President, 1946; Chairman Committee 613, which reported "Recommended Practice for the Design of Concrete Mixes,"now become a Standard of the Institute. Has written and participated in the writing of many technical contributions to ACI work.

**Vice-President**

HERBERT J. GILKEY, Head Department Theoretical and Applied Mechanics, Iowa State College, Ames, Ia.; ACI Member since 1924; Wason Medalist for Most Meritorious Paper, 1939; Member Standards Committee and chairman since March 1944; member Publications Committee since 1939; member Board of Direction, as Director Sixth District, 1937- 38 and elected to the same position 1945 and 1946.

**Director-at-Large** (to succeed himself)

HARRY F. THOMSON, Vice President, General Material Co., St. Louis, Mo., an ACI Member since 1928, elected Director Sixth District, 1939 and 1940, appointed 1942 to fill a vacancy as Director-at-Large, elected to succeed himself for a 3 year term in 1944. Mr. Thomson is author of two JOURNAL papers and was a member of the Publications Committee 1940-41.

**Regional Director, First District** (to succeed himself)

PAUL W. NORTON, Consulting Engineer, Boston, ACI Member since 1931; appointed Director First District, to fill vacancy caused by Mr. Kennedy's appointment as Director-at-Large, 1945. Elected Director, First District, 1946. Member of Advisory Committee 1940-44.

**Regional Director, Second District** (to succeed himself)

ROY R. ZIPPRODT, Research and Consulting Engineer, Committee on Reinforced Concrete Research, American Iron & Steel Institute, ACI Member since 1920 and Secretary ACI Committee 318, Standard Building Code. Member ACI Publications Committee 1939-41. Author of several ACI papers. Elected Director, Second District 1946.

**Regional Director, Third District**

WILLIAM H. KLEIN has been a member of the Institute since 1924 and a member of the Publications Committee since 1943. He was graduated from the University of Michigan with the degree of B. S. in Chemical Engineering in 1906 and since then has devoted his professional career to the manufacture of portland cement and the production of sand and gravel for use in concrete.

From 1906-11 he was with the Kansas Portland Cement Company as Chemist, Plant Engineer and Superintendent. From there he went to the Dixie Portland Cement Company and in 1926 became Southern Operating Manager of the Pennsylvania-Dixie Cement Corporation. From 1936 until July of this year he was Vice-President and General Operating Manager of that corporation. He also directed the operations of the Dixie Sand and Gravel Corporation from 1915 until 1946. Mr. Klein is also a member of the American Society of Testing Materials and has served as a member of Committee C-1 on Cement since 1918.

**Regional Director, Fourth District,** (to succeed himself)

H. P. BIGLER, Executive Vice President, Connors Steel Company, Birmingham, Alabama; graduated from Iowa State College with a B. S. in Chemical Engineering, 1922; Assistant Engineer of Materials, Illinois Division of Highways 1922-1925; Partner, Hurden Construction Company, Springfield, Illinois 1925-1926; Director, Rail Steel Bar Association, Chicago 1926-1941; Assistant to the President, Connors Steel Company 1942-1943; Executive Vice President, 1944 to date. Member of Board of Directors, Connors Steel Company; Consultant and member of Board of Directors, Rail Steel Bar Association; Chairman, A.S.T.M. Subcommittee V of A-1 on Reinforcement Steel; Chairman (during World War II) WPB Technical Advisory Committee on Reinforcement Steel; Member A.C.I. Committee 318-Standard Building Code; Member Joint Committee on Concrete and Reinforced Concrete, representing A.S.T.M.; Member American Iron and Steel Institute;

Member WPB Industry Advisory Committee, Rail Steel Industry; Founders' Committee, Southern Building Codes Congress and member of its Engineering Committee; Member American Iron and Steel Institute Committee on Reinforced Concrete Research.

Mr. Bigler was appointed Regional Director, Fourth District by the Board of Direction in Feb. 1946 to fill the vacancy left when Frank H. Jackson was appointed Director-at-Large.

**Regional Director, Fifth District**

A. J. Boase, Manager of the Structural Bureau of the Portland Cement Association; ACI Member since 1933; author of many papers and reports; chairman of Committee 317 which prepared the "Reinforced Concrete Design Handbook"; chairman of Committee 315 which just completed its "Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures"; chairman of Committee 318, Standard Building Code; chairman of Committee 323; member of Committee 321; former member of the Publications Committee 1941-44.

Mr. Boase is a graduate in civil engineering from the University of Colorado and received his M.S. from the University of Pennsylvania. His entire professional life has been in structural design work of one kind or another. For eight years following college, he was engaged in designing dams and tunnels in the Rocky Mountain region, principally for the Boston Colorado Power Co. For three years he was manager of the Fair Engineering Co., Denver, then joined the staff of the civil engineering department of the University of Pennsylvania, and later was made head of the civil engineering department of Pennsylvania Military College. He left this work to become regional structural engineer for the Portland Cement Association at Philadelphia, and in 1932 became manager of the Structural Bureau.

**Regional Director, Sixth District**

Charles H. Scholer, Head, Department of Applied Mechanics, Kansas State College, Manhattan, Kansas, has been an ACI Member since 1924 and was Director, Sixth District, 1934-35. He is Chairman of Department of Materials and Construction of the Highway Research Board; Chairman of Technical Committee of the Joint Research Project on Durability of Concrete (sponsored by H.R.Bd., A.S.T.M. and ACI) and is associated with many other technical committees.

This, briefly, is his record: B.S. degree in Civil Engineering, Kansas State College, 1914; head chainman U. S. General Land Office surveys, Santa Fe, N.M., 1914-15; chainman, maintenance of way and realignment problems, Atchison, Topeka and Santa Fe, R.R., Topeka, Kan. 1915-16; Assistant Engineer in charge of surveys and preparation of plans, and supervision of the construction of roads, pavements and bridges for the Extension Division, Kansas State College, Manhattan, Kan. 1916-17; Assistant Bridge Engineer in charge of design and supervision of construction of highway bridges for the Kansas State Highway Commission 1917-18; Engineer, design of artillery ammunition and its components for the Ordnance Department U. S. Army, 1918-19; Professor and Head, Department of Applied Mechanics, in charge of Road Materials Laboratory of the Engineering Experiment Station (official laboratory of the Kansas Highway Commission) Kansas State College, Manhattan, since 1919 except for leave of absence to take charge of investigational work in New York and Pennsylvania for the Portland Cement Association Feb. 1, 1939 to Sept. 1, 1940. He is a consultant on cement and concrete problems for various commercial concerns, the U. S. Army Corps of Engineers, and the Portland Cement Association.

**Nominating Committee**

The Nominating Committee also presented 20 candidates from among whom five are to be chosen to serve with the three latest past-presidents as the 1947 Nominating Committee—the candidate receiving the most votes on the letter ballot to be Chairman. The 20 candidates:

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|---|---|
| R. D. Bradbury | W. F. Kellermann |
| S. J. Chamberlain | H. L. Kennedy |
| M. N. Clair | H. S. Meissner |
| H. F. Clemmer | F. N. Menefee |
| A. B. Cohen | N. M. Newmark |
| H. E. Davis | J. R. Nichols |
| A. Foster, Jr. | J. R. Shank |
| V. L. Glover | B. W. Steele |
| F. B. Hornibrook | M. A. Swayze |
| V. P. Jensen | Bailey Tremper |

## New Members

The Board of Direction approved 87 applications for Membership (67 Individual, 7 Corporation, 5 Junior, 8 Student) received in August.

After taking into consideration a few losses by death, resignation, and for non-payment of dues these new Members bring our total as of Sept. 1, 1946 to 2778.

Aaron, Rollin G., 220 W. Andrews Drive, Atlanta, Ga.

Aicher, John B., 1 Lehigh Parkway, West, Allentown, Pa.

Albin, Boris, Ave. Tamaulipas 152-5, Mexico, D. F.

Alden, George C., 7056 N. Boston, Portland, Ore.

Alvarado R., Jose M., Manduca a Ferrenquin No. 133—Apto. B, Caracas, Venezuela, S. A.

Ammann, Othmar H., 111—8th Ave., New York 11, N. Y.

Araque, Pablo Quilez, Avenida Morelos, 24 Departamento 6, Mexico, D. F.

Ayer, Eugene D., Ready Mixed Concrete Co., 1800 E. 46th Ave., Denver 16, Colo.

Barnes, P. H., Concrete Industries, (Australia) Ltd., Box 3388 R. G. P. O., Sydney, Australia

Basalt Rock Company, Inc., 8th & River Sts., Napa, Calif. (A. G. Streblow)

Belger, Wm. M., 4229 Lugo Ave., Lynwood, Calif.

Bhagat, D. G., Executive Engineer, Public Works Dept., Karachi, India

Bradley, W. A., P. O. Box 2529, Greensboro, N. C.

Britson, Ralph A., Roland, Iowa

Bryant, Fred J., Box 474, Fabens, Texas

Brzozowicz, C. Peter, Canadian Breweries Ltd., 16 Gould St., Toronto, Ont., Canada

Buffalo Electro-Chemical Co. Inc., Station B, Buffalo 7, N. Y. (James F. Whalen Jr.)

Cain, Gerald I., 912 W. Harvard, Champaign, Ill.

Calcano, Gustavo Maggi, Este 4—No. 99, Caracas, Venezuela, S. A.

Carter, George R., Brookings Bldg., Duquoin, Ill.

Casado, Carlos Fernandez, Ingeniero de Caminos, Velazquez 69, Madrid, Spain

Chun, James H. H., 1127 Banyan St., Honolulu, Hawaii

Cohen, Sidney Phillip, P. O. Box 6749, Johannesburg, S. Africa

Colin, Edward C., 410 E. Chalmers St., Champaign, Ill.

Colorado State Highway Dept., State Capitol Annex, Denver, Colo. (James D. Bell)

Crumrine, Harold E., Branson St., La Fontaine, Ind.

Davila, Rafael, South Dakota State College, Brookings, S. Dakota

Dickinson Jr., W. Dewoody, Dickinson & White Inc., 115 N. Spring St., Little Rock, Ark.

Eager, William L., 254 New Custom House Denver 2, Colo.

Fox, Joseph H., 1601 Empire Bldg., Birmingham 3, Ala.

Fucik, Frank M., Charles R. Watts & Co., 6th & Leary Way N. W., Seattle, Wash.

Ghose, N. K., P. O. Saidpur, Dt. Rangpur, Bengal, India

Gilbert, Vincent C., 48 Stone Rd., Belmont, Mass.

Gillan, Gerald K., University of Missouri, Columbia, Mo.

Godoy, Silvio Uzcategui, c/o Velutini & Bergamin C. A., Sur 4 No. 46, Caracas, Venezuela, S. A.

Goodkind, Morris, New Jersey State Highway Dept., State House Annex, Trenton, N. J.

Gorden, Lawrence A., 4423½ Wesley Ave., Los Angeles 37, Calif.

Green, Murray, 10334—123rd St., Edmonton, Alta., Canada

Hastings, Garland M., 116 Audubon Road, Oak Ridge, Tenn.

Hemb, H. B., 5150 Church St., Skokie, Ill.

Hiner, C. W., Portland Cement Association, 504 S. 18th St., Omaha 2, Nebr.

Hsi, Ching-yao, 1463 Race St., Denver 6, Colo.

Hsu, H. Y., U. S. Bureau of Reclamation, Denver 2, Colo.

Hubbard, Lewis R., 1105 E. 40th St., Cleveland 14, Ohio

Hudspeth, Harry T., 4 E. 32nd St., Baltimore, Md.

Huie, Samuel L., 8422—108 St., Richmond Hill 18, N. Y.

Iyengar, C. L. N., c/o The Concrete Association of India, P. O. Box 138, Bombay, India

Johnson Jr., Orestes B., 5852 S. Michigan Ave., Chicago 37, Ill.

Kerr, A. W., A. W. Kerr & Associates, 407 S. Dearborn St., Chicago 5, Ill.

Knobel, Werner, Avenue Des Alpes 38, Lausanne, Switzerland

Kornacker, Frank J., 38 S. Dearborn St., Chicago 3, Ill.

Labrecque, Andre, Labrecque, LeBlanc & Labrecque, 10 Ouest, Rue St-Jacques, Montreal, Que., Canada

La Grelius, A. W., 523 N. 64th St. Seattle 3, Wash.

Landdeck, Norbert E., U. S. Engineer Office, 1709 Jackson St., Omaha 2, Nebr.

Lee, Tsin-shen, c/o Librarian, U. S. Bureau of Reclamation, Denver 2 Colo.

Linthicum, H. M., 1882 S. W. 14th Terr., Miami 35, Fla.

Loo, Pai-tsang, 1315 Adams St., Denver 6, Colo.

Losinger Inc., Monbijousrrasse 49, Bern, Switzerland
(Werner Knobel)

Lovering Harry D., Lovering Construction Co., 616 Guardian Bldg., St. Paul 1, Minn.

Ludington's Sons Inc., I. M., 720 Lexington Ave., P. O. Box 105, Rochester, N.Y.

Ma, Chun-shou, c/o Librarian, U. S. Bureau of Reclamation, Denver 2, Colo.

Mathews, Harvey, 1720 California St., Denver 2, Colo.

Moore, Lewis E., 285 Columbus Ave., Boston 16, Mass.

Murtaza, S. G., The Fairfax Hotel, 2100 Massachusetts Ave., Washington, D. C.

Nazario, William Mora, 27 Wilson St., Ponce, Puerto Rico

New Zealand Public Works Dept., Wellington, New Zealand
(T. G. Beck)

Ochse, John N., F. H. McGraw & Co., 1505 First National Bank Bldg., Pittsburgh 22, Pa.

Ove, Edward N., U. S. Engineer Office, 1709 Jackson St., Omaha 2, Nebr.

Padhi, Ananda Chandra, International House University of California, Berkeley 4, Calif.

Payne Jr., Chas. W., The Austin Co., 16112 Euclid Ave., Cleveland 12, Ohio

Pehrson, G. A., 1312 Old National Bank Bldg., Spokane 8, Wash.

Post, Raymond G., 713 Reymond Bldg., Baton Rouge 6. La.

Ready Mixed Concrete (Queensland) Pty. Ltd., Box 842-1, G. P. O., Brisbane, Australia
(R. Wilson)

Reyhner, Theodore O., Box 163 University Station, Grand Forks, N. D.

Rivero, Manuel Ray, Lealtad 624, Havana Cuba

Ros, M. R., Schwendenhaustr 9, Zurich, Switzerland

Rylander, P. N., No. 8 Parkway Dr., Pelham 65, N. Y.

Scarborough, R. L., U. S. Bureau of Reclamation, Friant, Calif.

Seif, Emanuel, 150 E. 95th St., Brooklyn 12, N. Y.

Stork, Jiri, c/o Fellowship Section UNRRA, Washington 25, D. C.

Ullman, R. W., 3057 Edgehill Rd., Cleveland Hts. 18, Ohio

Waltz, Wayne W., 711 Sixth St., Brookings S. Dakota

Warner, H. W., Columbus Testing Laboratory, 755 N. High St., Columbus, Ohio

Wastlund, Georg, Royal Technical University, Stockholm, Sweden

Werner, Donovan, Skanska Cementaktiebolaget, Malmo, Sweden

Yang, Fang-yu, c/o Librarian, U. S. Bureau of Reclamation, Denver 2, Colo.

Yang, Ter-Kung, c/o Library, U. S. Bureau of Reclamation, Denver 2, Colo.

**The Report of ACI Committee 315, "Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures" is currently the best seller among ACI special publications.**

# WHO'S WHO in this JOURNAL

## T. C. Powers and T. L. Brownyard

are co-authors of "Studies of the Physical Properties of Hardened Portland Cement Paste", a paper of nine parts to be published in seven installments. The first installment appears on p. 101 of this JOURNAL.

Mr. Powers, an ACI member since 1927, twice a Wason Research Medalist, is not by any means new to ACI *Proceedings* pages. A member of the Portland Cement Association research staff since 1930, he organized the Division of Basic Research in 1940 and was in charge as Assistant to the Director of Research until November 1944 when he made Manager of Basic Research, the position he now holds.

Mr. Brownyard, a native of Michigan, attended high school at Cedar Springs, Michigan prior to entering Western Michigan College. After two years there, he taught chemistry at the Fremont, (Michigan) High School for four years. In the fall of 1929 he entered Massachusetts Institute of Technology and the following spring he was awarded the Francis P. Garvan Fellowship at Johns Hopkins University where he received his PhD degree in 1934. After another year at Johns Hopkins he spent a year each with the Standard Lime and Stone Company, Baltimore and the Standard Oil Company of Indiana.

Doctor Brownyard came to the Portland Cement Association in July 1937 as research chemist and remained until January 1943. The next three years were spent as an air navigator in the U.S. Navy. He returned to the Portland Cement Association in December 1945 but in June 1946 returned to the Navy Department as a civilian scientist.

Dr. Brownyard is a member of the American Chemical Society, Sigma Xi, Phi Beta Kappa, Phi Lambda Upsilon, Kappa Rho Sigma and Gamma Alpha.

## Frank H. Jackson

a member of the Institute since 1924, member of several committees and of the Board of Direction, needs no introduction to most ACI JOURNAL readers. He is Principal Engineer of Tests for the U. S. Public Roads Administration and the author of many papers on concrete and related subjects. His paper entitled "The Durability of Concrete in Service" appears on p. 165 of this JOURNAL.

## Georg Wastlund and Anders Eriksson

are co-authors of the paper entitled, "Wear Resistance Test on Concrete Floors and Methods of Dust Prevention", which appears on page 181.

Professor Wastlund, a new member of the Institute, appears as a Journal author for the first time although he has written many technical papers which have been published in Europe.

He was graduated as a civil engineer from the Royal Technical University, Stockholm in 1928 and received his doctor's degree in 1934. Following graduation he was an assistant at the Structural Strength Laboratory of the University from 1929 to 1936. He was chief structural engineer at Skånska Cementgjuteriet, Malmo, Sweden from 1936 to 1941 when he became professor of Structural Engineering and Bridge Building at the Royal Technical University. He is also a director of the Swedish Cement and Concrete Research Institute.

Professor Wastlund has visited the United States twice recently; once in the fall of 1945 to study the design and construction of airfield concrete pavements and once last summer as a leader of a group of 32 civil engineering students from the Royal Technical University to study civil engineering works in the eastern states.

Mr. Eriksson graduated as a civil engineer from the Royal Technical University of Stockholm in 1940. Following graduation he was associated with the Swedish Cement Association in Stockholm in research on concrete for defense purposes with particular studies on the behavior of concrete under dynamic load.

In 1942 Mr. Eriksson joined the staff of the Swedish Cement and Concrete Research Institute. He is at present working on investigations of new methods for measuring the workability of concrete.

## Honor Roll

February 1 to July 31, 1946

The Honor Roll for the period February 1, to August 31, 1946 finds T. E. Stanton leading with 22 members and J. L. Savage a close second with 20.

| | |
|---|---|
| **T. E. Stanton** | 22 |
| **J. L. Savage** | 20 |
| **E. W. Thorson** | 10 |
| **Walter H. Price** | 9 |
| **C. C. Oleson** | 7 |
| **Anton Rydland** | 6½ |
| **Charles E. Wuerpel** | 5½ |
| **Jacob Fruchtbaum** | 5 |
| **Karl W. Lemcke** | 4½ |
| **Martin Kantorer** | 4 |
| **James A. McCarthy** | 4 |
| **Newlin D. Morgan** | 4 |
| **K. E. Whitman** | 4 |
| **Hernan Gutierrez** | 3½ |
| **E. F. Harder** | 3½ |
| **A. Amirikian** | 3 |
| **Birger Arneberg** | 3 |
| **D. R. Cervin** | 3 |
| **Raymond E. Davis** | 3 |
| **Ray C. Giddings** | 3 |
| **C. A. Hughes** | 3 |
| **Henry L. Kennedy** | 3 |
| **James J. Pollard** | 3 |
| **F. E. Richart** | 3 |
| **A. J. Boase** | 2½ |
| **W. A. Carlson** | 2½ |
| **T. F. Collier** | 2½ |
| **H. M. Hadley** | 2½ |
| **Miguel Herrero** | 2½ |
| **Alberto Dovali Jaime** | 2½ |
| **F. N. Menefee** | 2½ |
| **E. M. Rawls** | 2½ |
| **H. F. Thomson** | 2½ |
| **Stanton Walker** | 2½ |
| **Wm. R. Waugh** | 2½ |
| **Rene L. Bertin** | 2 |
| **J. F. Barton** | 2 |
| **Emil W. Colli** | 2 |
| **Aloysius E. Cooke** | 2 |
| **H. B. Emerson** | 2 |
| **Issac Hausman** | 2 |
| **Denis O. Hebold** | 2 |
| **F. R. McMillan** | 2 |
| **Robert L. Mauchel** | 2 |
| **J. J. Mullen** | 2 |
| **Dean Peabody** | 2 |
| **Henry Pfisterer** | 2 |
| **Raymond C. Reese** | 2 |
| **R. D. Rogers** | 2 |
| **Simeon Ross** | 2 |
| **Moe A. Rubinsky** | 2 |
| **H. C. Shields** | 2 |
| **LeRoy A. Staples** | 2 |
| **Flory J. Tamanini** | 2 |
| **J. W. Tinkler** | 2 |
| **Lewis H. Tuthill** | 2 |
| **I. L. Tyler** | 2 |
| **C. S. Whitney** | 2 |
| **B. G. Anderson** | 1½ |
| **R. H. Bogue** | 1½ |
| **R. W. Crum** | 1½ |
| **H. J. Gilkey** | 1½ |
| **E. J. Glennan** | 1½ |
| **T. R. S. Kynnersley** | 1½ |
| **John J. Murray** | 1½ |
| **Y. G. Patel** | 1½ |
| **J. C. Pearson** | 1½ |
| **R. D. Rader** | 1½ |
| **N. L. Shamroy** | 1½ |
| **Thomas C. Shedd** | 1½ |
| **A. L. Strong** | 1½ |
| **John Tucker Jr.** | 1½ |
| Paul W. Abeles | 1 |
| Julius Adler | 1 |
| Kasim Atlas | 1 |
| J. F. Barbee | 1 |
| Hugh Barnes | 1 |
| H. J. Bateman | 1 |
| F. J. Beardmore | 1 |
| R. F. Blanks | 1 |
| Joseph W. Body | 1 |

H. A. Bradt....................1
Raymond L. Brandes..............1
J. M. Breen....................1
S. D. Burks....................1
M. E. Capouch..................1
R. W. Carlson..................1
T. J. Cavanagh.................1
Frank W. Chappell...............1
Anthony D. Ciresi...............1
A. R. Collins...................1
John Conzelman.................1
Holly A. Cornell................1
W. S. Cottingham................1
G. H. C. Crampton...............1
Charles A. Daymude..............1
F. K. Deinboll..................1
C. T. Douglass..................1
Clifford Dunnells................1
D. W. Faison....................1
H. F. Faulkner..................1
Jess Fellabaum..................1
Alexander Foster................1
Herman Frauenfelder.............1
Meyer Fridstein.................1
J. K. Gannett...................1
D. E. Gondolfi..................1
E. Gonzales Rubio...............1
Fred A. Gorham..................1
L. E. Grinter....................1
Ernst Gruenwald.................1
Elliott A. Haller.................1
M. J. Hawkins...................1
Elmo Higginson..................1
Frank R. Hinds..................1
G. H. Hodgson...................1
W. M. Honour...................1
O. W. Irwin.....................1
M. E. James....................1
R. O. Jameson...................1
Axel H. Johnson.................1
Wm. R. Kahl....................1
R. R. Kaufman..................1
Thomas M. Kelly................1
Edgar R. Kendall................1
Edward F. Keniston..............1
Orville Kofoid..................1
Guy H. Larson...................1
H. Walter Leavitt................1
Elbert F. Lewis..................1
Bartlett G. Long.................1
F. A. Luber.....................1
James E. McClelland.............1
M. J. McMillan..................1
Edward P. McMullin..............1
F. R. MacLeay...................1
D. G. Marler....................1
Charles Miller...................1
Hugh Montgomery................1
Robert B. B. Moorman............1
Rene Pulido y Morales............1
Ben Moreell.....................1
I. Narrow.......................1
Wm. T. Neelands................1
Paul W. Norton..................1
Ben E. Nutter...................1
Philip Paolella...................1
D. E. Parsons...................1
A. F. Penny.....................1
Richard L. Pinnell................1
Harry W. Piper..................1
C. J. Posey.....................1
Herman G. Protze Jr.............1
C. C. Pugh......................1
Erik Rettig......................1
George P. Rice..................1
E. O. Rosberg...................1
Arthur Ruettgers.................1
John A. Ruhling..................1
G. R. Schneider..................1
Herman Schorer..................1
John C. Seelig...................1
George G. Smith.................1
J. H. Spilkin....................1
H. D. Sullivan...................1
J. Antonio Thomen...............1
Hugh F. Tolley...................1
Bailey Tremper..................1
Harold C. Trester................1
Oscar J. Vago...................1
Joseph J. Waddell...............1
W. W. Warzyn...................1
David Watstein..................1
Piers M. Williams................1
George C. Wilsnack..............1
Ralph E. Winslow................1
Douglas Wood...................1
Ray A. Young...................1

The following credits are, in each instance, "50-50" with another Member.

A. Arnstein
J. B. Baird
Michel Bakhoum
Sabahattin Basman
Paul L. Battey
E. Ben-Zvi
E. O. Bergman
Hugh Bigler
Carlos Blaschitz
G. C. Britton
Ernest L. Brodbeck
Fred Burggraf
Fred Caiola
Miles N. Clair

Chas. A. Clark
A. B. Cohen
Sam Comess
W. A. Coolidge
R. A. Crysler
Atahualpa Dominguez
G. J. Durant
E. E. Edwards
A. C. Eichenlaub
Axel Erikeson
Harry R. Erps
Cevdet Erzen
E. E. Evans
K. P. Ferrell
G. V. Gezelius
Howard A. Gray
Per O. Hallstrom
Hunter Hanly
Shortridge Hardesty
H. L. Henson
A. W. Hicks
Meyer Hirschthal
Ralph B. Horner
Fred Hubbard
Manuel Castro Huerta
H. D. Humphries
W. C. Huntington
Frank H. Jackson
V. P. Jensen
Paul A. Jones
O. G. Julian
George L. Kalousek
W. D. Kimmel
Lane Knight
Wm. Lerch
Raul Lucchetti
J. A. McCrory
Douglas McHenry
Ian Macallan
J. B. Macphail
Sidney M. Major Jr.
Charles Mannel
George W. Meyer
Eugene Mirabelli
C. C. More
A. B. L. Moser
Fernando Munilla
T. D. Mylrea
D. Lee Narver
H. T. Nelson
Syberen Frank Nydam
Wm. D. Painter
George P. Palo
R. S. Phillips
David Pirtz
Robert B. Provine
Frank A. Randall
T. E. Shelburne
C. E. Shevling
Harold Oliver Sjoberg
Marvin Spindler
Charles M. Spofford
D. J. Steele
Hale Sutherland
M. O. Sylliaasen
LeRoy A. Thorssen
Zaldua Uriarte
Jose Vila
D. S. Walter
Ray V. Warren
J. C. Watt
Herbert J. Whitten
Eugene P. H. Willett
C. T. Wiskocil
Ernest B. Wood
R. B. Young

## Roderick B. Young

The Board of Direction of the Institute at a recent meeting adopted the following resolution in recognition of the 16 years spent by Mr. Young as a member of the Board: "to Roderick B. Young, an Expression of Appreciation:

"For sixteen years a member of the Board of Direction of the American Concrete Institute as Director, Vice President, President and Past President.

"Through your calm deliberations, your sound judgment, your quiet initiative, your careful planning, your ceaseless efforts you have brought a mark of distinction to the Board and to the Institute as a whole such as has been contributed by few men.

"We, the Members of the Board of Direction of the American Concrete Institute, in meeting assembled in the city of Buffalo on this the 21st day of February in the year nineteen hundred and forty-six, offer this testimonial in recognition of your retirement from the Board and in appreciation of your brilliant and unselfish services throughout the years."

## Morton O. Withey

Members of the Institute and friends will be glad to learn of the recent appointment of Professor Withey as Dean of the College of Engineering, University of Wisconsin, in June of this year.

Dean Withey received his B.S. degree from Dartmouth College in 1904 and his C.E. degree from Thayer School of Engineering in 1905. With the exception of four months as an apprentice at the Illinois Steel Company, he has spent his entire professional career at the University of Wisconsin, having joined the College of Engineering faculty in September 1905 as an instructor and holding successively the positions of assistant professor, associate professor, professor, chairman of the Department of Mechanics and now, Dean.

Aside from his teaching, Dean Withey has done research on properties of the materials of construction in the fields of masonry, concrete and steel. He has served as chairman and member of research committees of the A.S.C.E., A.S.T.M. and the Highway Research Board.

A member of the Institute since 1921, Dean Withey is the author of several papers and reports, a Wason Research Medalist, chairman of ACI Committee on Post War Planning, member of the Advisory Committee, member of the Board of Direction since 1937 as Director, Vice-President, President and Past-President. He is also a member of Wisconsin Society of Professional Engineers (Past-President), A.S.T.M., S.P.E.E. and National Society of Professional Engineers.

## Admiral Moreell

Announcement has been made of the retirement of Admiral Ben Moreell, Past-President of ACI, from active duty in the U. S. Navy and of his election as president of the Turner Construction Co., New York. It seems appropriate at this time to publish the following paragraphs which appeared recently in *Engineering News-Record* in the section entitled "Our Engineer Admirals" over the initials of Editor Waldo G. Bowman:

"When recently the President recommended and Congress approved the pinning of the four stars of a full admiral of the United States Navy on the broad shoulders of Ben Moreell, they honored themselves and the American people as well as the recipient. Other individuals and organizations in growing numbers since even before the war have taken satisfaction and pride in bestowing recognition for attainment on this relatively young officer of the Navy's Civil Engineer Corps. This article is no exception. Ben Moreell, now Chief of the Material Division in the Office of the Assistant Secretary of the Navy, is one of the great men of our times, and he got that way by supplementing splendid natural abilities with a devotion to duty and a will toward hard work that has been plain for all to see.

"First staff officer ever to attain four star rank in the Navy, and the first full admiral since the Annapolis Naval Academy was founded not to be graduated from that school. Ben Moreell's true stature is measured by activities and accomplishments far wider in scope than the Navy's business, great as that has become. Before the war his standing in the field of reinforced-concrete design was such that he could be classed among the "long haired" practitioners of the art, although in manner and appearance he is at the opposite pole from the usual idea of deeply scientific men. Since the war he has been handed as extra-curricular activities two of the toughest jobs that have been dumped on the President's doorstep—to operate the strike-bound oil refineries and pipe-lines, including negotiation of a settlement of the controversy, and to operate the coal mines that John L. Lewis and the owners bequeathed to the government. Ben Moreell is the personification of the idea held by most engineers, except Ben Moreell, that one of their training and abilities could run most jobs in or out of government better than anyone else.

"To characterize a man such as Admiral Moreell is not easy, to put him in some generally recognized category impossible. Now 54 years old, he is a big strapping 220 pounder who was a football and track star in college. He is tough and he is charming, often both at the same time.

"Always making the most out of every assignment, Admiral Moreell spent the year from June, 1932, to June 1933, at the Ecole Nationale des Ponts et Chausees, in Paris and returned to introduce this country to the advantages of Freyssinet's high-strength prestressed concrete, his jacking method of decentering concrete arches and of Mesnager's use of hinges in reinforced-concrete structures. Assigned, on his return, as assistant design manager of the bureau with personal supervision for a new ship model testing basin at Carderock, Md., Admiral Moreell applied this recently accumulated knowledge to the design of the first articulated concrete structure in this country.

"Foresight and drive also produced the Seabees, that force of construction men inaugurated, built up and commanded by Admiral Moreell, which covered itself with glory in building bases, manning floating drydocks, stevedoring ships and fighting when need be from Normandy to Tokyo. At the end of the war there were 240,000 men and 10,000 officers in the Seabees, and 85 percent of them were overseas. And speaking in statistics, the five-year war construction program directed by Admiral Moreell as Chief of the Bureau of Yards and Docks, involved a construction investment in 900 Naval bases, including 300 advance bases, of over 10 billion dollars. In comparison, during the 21 previous years, including World War I, Yards and Docks construction totaled only one-third of a billion dollars.

"Now that he wears four of those stars he might be considered to have reached the top. But Ben Moreell, member of Sigma Xi, honorary scientific fraternity Tau Beta Pi, honorary engineering fraternity, holder of all top medals of the American Concrete Institute, honorary member of the American Society of Civil Engineers, revered by construction men as one of them, looked upon by labor as a welcome adviser and honest critic, has grown with every job, and his friends say he will continue to grow."

## John J. Earley

In appreciation of Mr. Earley's long and active participation in Institute affairs the Board of Direction recently passed the following resolution which has been sent to Mrs. Earley and which has been made a part of the minutes of the Board meeting:

"It is with deep regret that the American Concrete Institute notes the passing of John J. Earley, Member of the Institute for 28 years, and of this Board for three periods, 1924-26 as Director 4th District and Vice-President, 1932-34 as Director 4th District; 1936 to 1943 as Vice-President and Past-President—12 years of Board service in which his contributions to the development of concrete and of its literature were unique, bringing to the Institute the viewpoint and the experience of an artist and craftsman. His achievements won him the Wason Medal for the "Most Meritorious Paper" of 1923; The Turner Award in 1934 for "Outstanding Achievement in Developing Concrete as an Architectural Medium", and election to Honorary Membership in 1943. He will be remembered not only for his studious art and exacting craftmanship as shown in Chicago's "Fountain of Time". Wilmette's Baha'i Temple and many churches and other structures enriched by his plastic mosaic, in which concrete is dominated by a wide range of color, but also, by those who know him best, as a conversationalist whose well-informed talk was similarly enriched by his rare sense of life, drama and color."

## James A. Halkins

Word has recently been received of the death on June 12, 1946, of Mr. Halkins, vice president of the Waylite Company and a member of the Institute since 1940.

Mr. Halkins received his Civil Engineer Degree from the University of Pennsylvania with the class of 1922. Following his graduation, he served in an engineering capacity for the Illinois Glass Company. For the past ten years he has been employed by the Waylite Company, a Corporation manufacturing lightweight aggregate, and only recently was signed as Vice-President of the concern. During the last seven years Mr. Halkins became well known in engineering circles in the East in his capacity as manager of the concern's eastern plants. He is well remembered in the Middle West for his engineering work in the Chicago area.

## Henry Dievendorf Dewell

member of the American Concrete Institute since 1928, died at his home in Berkeley, California, March 20, 1946. He was born in Springfield, Ohio on October 24, 1881.

Mr. Dewell received his engineering education at the University of California, graduating with a degree of Bachelor of Science in 1906. Following his graduation he became member of the staff of Howard and Galloway, Architects and Engineers, at a time when this firm was very active in the reconstruction of San Francisco after the earthquake of April 18, 1906. In 1912 he became chief structural engineer for the Panama-Pacific International Exposition and later became assistant superintendent of building construction and engineer for domestic water supply for the Exposition.

From 1915 until his death Mr. Dewell was engaged in private practice in San Francisco. He specialized in structural work and in particular was an authority on earthquake-resistant construction. In addition to his practice, from 1918 to 1920, he lectured at the University of California and from 1931 to 1945 he was a member of and for many years President of the California State Board of Registration for Civil Engineers.

Mr. Dewell served as a Director of the American Society of Civil Engineers from 1925 to 1927 and as Vice President in 1934 and 1935. He was also President of the San Francisco Section of the Society in 1930.

In addition to his membership in the American Concrete Institute Mr. Dewell was a member of the American Society of Civil Engineers; the Seismological Society of America; the Advisory Committee on Vibration Research at Stanford University; the Structural Engineers Association of Northern California; the American Society for Testing Materials; and the National Committee on Wood Utilization. He was elected to Sigma XI, Tau Beta Pi and Chi Epsilon honorary scholarship societies and held membership in the Engineers Club of San Francisco and the Commonwealth Club of California.

## Supervisor of concrete research, Australia

The Australian Council for Scientific and Industrial Research is interested in securing the services of an officer qualified to supervise concrete research in Melbourne, Australia. A Building Materials Research Section has recently been established, and the Officer-in-Charge of the Concrete Section would be required to set up projects of a fundamental nature on the properties of concrete and cement products.

The position would be classified as Principal Research Officer, and the salary would be approximately 900 pounds per annum for a permanent appointment. The fares of the appointee and his family would be paid to Australia, and a small additional amount would be made available to assist in the transportation of household effects.

Interested persons may obtain further details by writing Mr. N. A. Whiffen, Officer-in-Charge, Scientific Research Liaison Office, Room 523, 1785 Massachusetts Ave., N.W., Washington 6, D.C.

## Inland licenses Carnegie-Illinois and other subs of U. S. Steel to manufacture Hi-bond reinforcing bar

Negotiations by Carnegie-Illinois and other subsidiaries of the U. S. Steel Corporation for a license to manufacture and sell the Inland Hi-bond reinforcing bar for concrete construction have been completed, according to H. H. Straus, vice-president of the Inland Steel Company, which was responsible for the development of the bar. As a result of the licensing agreement, he said, the reinforcing bar will soon be available in greater quantities throughout the United States.

# ACI publications in large current demand

## Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures (1946)

Reported by ACI Committee 315, Detailing Reinforced Concrete Structures, Arthur J. Boase, Chairman, this book reached the top of the ACI "best seller" list within one month of its distribution to all ACI members in good standing in July 1946. It is a large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$2.50; to ACI Members—$1.50.

## ACI Standards—1945

148 pages, 6x9 reprinting ACI current standards: Building Regulations for Reinforced Concrete (ACI 318-41); three recommended practices: Use of Metal Supports for Reinforcement (ACI–319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—all between two covers, $1.50 per copy—to ACI Members, $1.00.

## Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

## ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

## "The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members $1.00.

## Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

For further information about ACI Membership and Publications (including pamphlets presenting Synopsis of recent ACI papers and reports) address:

**AMERICAN CONCRETE INSTITUTE New Center Building Detroit 2, Michigan**

# 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.**

# SYNOPSES of recent ACI Papers and Reports

**Institute papers of this JOURNAL Vol. 18 which are currently available. Unless otherwise noted separate prints are 25 cents each. Starred ★ items are 50 cents. Please order by title and title number.**

## REINFORCED CONCRETE COLUMNS UNDER COMBINED COMPRESSION AND BENDING.................... 43-1

HAROLD E. WESSMAN—Sept. 1946, pp. 1-8 (V. 43)

Algebraic methods available heretofore for the analysis of the reinforced concrete column subject to combined compression and bending have usually involved the solution of a complex cubic equation and have taken considerable time when applied to particular problems. A new method of successive approximations converging rapidly to an exact answer and avoiding the use of the cubic equation is presented in this paper. The key to the method is the reciprocal relationship existing between the load axis and the neutral axis of the transformed section. The method may be applied to any shape of cross section and any arrangement of reinforcing steel, providing there is one axis of symmetry and the plane of bending coincides with this axis. The theory behind the method is presented and illustrated with three typical problems.

## EFFECT OF MOISTURE ON THERMAL CONDUCTIVITY OF LIMEROCK CONCRETE........................ 43-2

MACK TYNER—Sept. 1946, pp. 9-20 (V. 43)

The coefficient of thermal conductivity, $k$, of limerock concrete is a function of temperature, composition and density or moisture content. No attempt has been made to measure the effect of temperature on $k$. Holding the temperature reasonably constant, the effect of composition on $k$ has been measured for two limerock concrete mixes (1:5 and 1:7 by volume). The 1:5 mix has a $k$ that is 10 per cent larger than the $k$ for the 1:7 mix.
With the temperature and composition held constant, the effect of moisture on $k$ for the 1:5 and 1:7 mixes has been measured. The moisture content has a very profound effect on $k$, e.g. increases of moisture from zero to 5 per cent increases the $k$ of 1:5 mix by 23 per cent and from zero to 10 per cent increases the $k$ by 46 per cent. Concretes should be kept dry if their maximum heat insulation effect is desired.

## ★CEMENT INVESTIGATIONS FOR BOULDER DAM—RESULTS OF TESTS ON MORTARS UP TO AGE OF 10 YEARS..................... 43-3

RAYMOND E. DAVIS, WILSON C. HANNA and ELWOOD H. BROWN—Sept. 1946, pp. 21-48 (V. 43)

The effects of composition and fineness of the laboratory cements employed in cement investigations for Boulder Dam upon strength, volume changes, and sulfate resistance of mortars, are reported for ages up to 10 years. For both wet and dry storage conditions, factors for each of several ages are given which indicate the contribution of each of the four major compounds present in portland cement to tensile and compressive strengths and volume changes.

## ★ANALYSIS AND DESIGN OF ELEMENTARY PRESTRESSED CONCRETE MEMBERS......................... 43-4

HERMAN SCHORER—Sept. 1946, pp. 49-88 (Vol. 43)

The purpose of this paper is to outline the analysis and design of elementary prestressed concrete members, such as beams, columns, ties, etc., subjected to internal and external axial forces and bending moments. The internal stresses, caused by the action of the prestress forces, are combined with the stresses due to external loads in three typical loading stages. The first stage considers the stress condition resulting from the simultaneous application of all sustained loads. The second stage determines the stress changes due to normal live loads, based on a truly monolithic participation of the entire concrete area. The third stage assumes a cracked tension zone, which condition introduces the derivation of ultimate stresses and clarifies the influence of the prestress action on the type of failure. The analytical expressions are simplified by means of convenient ratios, which essentially define the sectional shape, the effective steel prestress, and the concrete fiber stresses. Numerical examples serve to illustrate the various steps.

## ★STUDIES OF THE PHYSICAL PROPERTIES OF HARDENED PORTLAND CEMENT PASTE (Part I) .....43-5a

T. C. POWERS and T. L. BROWNYARD—Oct. 1946, pp. 101-132 (V. 43)

**IN NINE PARTS**

**Part 1.** **A Review of Methods That Have Been Used for Studying the Physical Properties of Hardened Portland Cement Paste**

**Part 2.** **Studies of Water Fixation**
**Appendix to Part 2**

**Part 3.** **Theoretical Interpretation of Adsorption Data**

**Part 4.** **The Thermodynamics of Adsorption**
**Appendix to Parts 3 and 4**

**Part 5.** **Studies of the Hardened Paste by Means of Specific-Volume Measurements**

**Part 6.** **Relation of Physical Characteristics of the Paste to Compressive Strength**

**Part 7.** **Permeability and Absorptivity**

**Part 8.** **The Freezing of Water in Hardened Portland Cement Paste**

**Part 9.** **General Summary of Findings on the Properties of Hardened Portland Cement Paste**

This paper deals mainly with data on water fixation in hardened portland cement paste, the properties of evaporable water, the density of the solid substance, and the porosity of the paste as a whole. The studies of the evaporable water include water-vapor-adsorption characteristics and the thermodynamics of adsorption. The discussions include the following topics:

1. Theoretical interpretation of adsorption data
2. The specific surface of hardened portland cement paste
3. Minimum porosity of hardened paste
4. Relative amounts of gel-water and capillary water
5. The thermodynamics of adsorption
6. The energy of binding of water in hardened paste
7. Swelling pressure
8. Mechanism of shrinking and swelling
9. Capillary-flow and moisture diffusion
10. Estimation of absolute volume of solid phase in hardened paste
11. Specific volumes of evaporable and non-evaporable water
12. Computation of volume of solid phase in hardened paste
13. Limit of hydration of portland cement
14. Relation of physical characteristics of paste to compressive strength
15. Permeability and absorptivity
16. Freezing of water in hardened portland cement paste

## MINIMUM STANDARD REQUIREMENTS FOR PRECAST CONCRETE FLOOR UNITS .................. 43-6

REPORTED BY ACI COMMITTEE 711—Oct. 1946 pp. 133-148 (V. 43)

**Supersedes 40-17, 42-11.**

These minimum standard requirements are to be used as supplements to the ACI "Building Regulations for Reinforced Concrete" (ACI 318 41). With respect to design for strength, i. e., for bending moment, bond and shear stresses, all types shall be designed in accord with standard reinforced design theory and ACI 318-41. With respect to cover, there is in some cases departure therefrom justified by the greater refinement in the finished product when made by factory methods with factory control. Precast floor systems with I-beam type and hollow core type joists are covered. Appendix contains applicable sections of the ACI code (ACI 318-41). This report, originally published in Feb. 1944 Journal, has been revised by the committee and adopted by the Institute as an ACI Standard, Aug. 1946. The committee consists of F. N. Menefee, Chairman, Warren A. Coolidge, R. E. Copeland, Clifford G. Dunnells, H. B. Hemb, Harve Kilmer, Glenn Murphy, Gayle B. Price, John Strandberg, J. W. Warren, Roy R. Zipprodt.

## RECOMMENDED PRACTICE FOR THE CONSTRUCTION OF CONCRETE FARM SILOS .................... 43-7

REPORTED BY ACI COMMITTEE 714—Oct. 1946, pp. 149-164 (V. 43)

**Supersedes 40-10, 42-12.**

These recommendations describe practice for use in the design and construction of concrete silos—stave, block and monolithic, for the storage of grass or corn silage. The report is the work of the committee consisting of William W. Gurney, Chairman, J. W. Bartlett, Walter Brassert, Claude Douthett, Harry B. Emerson, William G. Kaiser, R. A. Lawrence, G. L. Lindsay, J. W. McCalmont, Dalton G. Miller, C. C. Mitchell, K. W. Paxton, B. M. Radcliffe, Charles F. Rogers, Stanley Witzel. It was adopted by the Institute as an ACI Standard Aug. 1946.

## THE DURABILITY OF CONCRETE IN SERVICE ........................ 43-8

F. H. JACKSON—Oct. 1946, pp. 165-180 (V. 43)

This paper discusses the problem of concrete durability with reference primarily to highway bridge structures located in regions subject to severe frost action. Four major types of deterioration are defined and illustrated and several specific matters which have bearing on the problem, including the effect of construction variables, modern vs. old fashioned cements, air entrainment and the so-called "cement-alkali" aggregate reaction, are discussed. The report concludes with a series of recommendations indicating certain corrective measures which should be taken.

## WEAR RESISTANCE TESTS ON CONCRETE FLOORS AND METHODS OF DUST PREVENTION ........... 43-9

GEORG WASTLUND and ANDERS ERIKSSON—Oct. 1946, pp. 181-200 (V. 43)

This paper presents a description of tests made on concrete floor specimens of various types in order to determine their resistance to wear and to investigate the character of deterioration of concrete floor surfaces due to traffic. The results of these tests show that concrete floors provided with finish courses containing coarse aggregate up to about ⅜ inch in size and an excess of pea gravel are definitely superior to concrete floors with a finish course containing fine sand only which are common in Sweden at the present time. Moreover, this investigation has helped to elucidate the causes of the often very severe and detrimental dusting of concrete floors. The surface skin of concrete floors is of poor quality and is easily abraded. Dusting can be considerably reduced if the poor surface skin is removed by machine grinding provided that the concrete below the surface skin is of first-rate quality. The paper concludes by proposing a detailed tentative specification for concrete floor finish which differs in essentials from current Swedish practice.

**Mark your Calendar—ACI's 43rd Annual Convention, at Cincinnati, February 24-26, 1947**

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