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Fig. 1—Hand-rail posts at the Snake River Bridge.

Fig. 2—Culvert headwall at the Franklin Avenue Undercrossing in Bend.

distortion that concrete members will stand without serious injury is surprising in view of the general opinion that concrete is a stiff and brittle material. The cause of damage of this kind can be determined easily, and the repairs can be expected to effect a permanent cure. Deterioration caused by some weakness of the material itself is more difficult to treat. The cause of the trouble is obscure and remedial work is often only a temporary check to the deterioration.

The first alarming example of this second type of disintegration to be noticed on the Oregon highway system showed up on the Snake River Bridge at Ontario, Ore. This bridge consists of four 200-ft. steel truss spans with concrete deck, curbs and piers, and with a short concrete viaduct approach at each end. A steel handrail was used on the truss span and a concrete handrail on the approach spans. The bridge was built in 1924 and by 1928 wide cracks had opened up in the piers and in the larger members of the handrail. These cracks were peculiar in that little, if any, disintegration of the concrete accompanied the cracking. The cracks were relatively wide, an open width of as much as three-eighths inch was found, but the concrete along the edges of the crack appeared sound. Fig. 1 shows a massive handrail post with the wide cracks showing in the top. Fig. 2 shows a head wall of a culvert built ten years before the picture was taken. The cracking occurred in the first five years after the structure was built, and has shown little or no progress since that time. The fact that these cracks occur in members not subject to structural stress indicates a lack of any connection between stress and this type of cracking.

An attempt was made to seal the cracks in the Snake River Bridge piers with a heavy mixture of white lead and oil forced in under pressure. The continued opening of the cracks broke the seal, and the pier shafts and webs were encased with concrete in 1934. This new concrete was waterproofed in 1937 and no further trouble has occurred. The handrails were not encased, and the cracks have shown no increase in width for the past ten years. It seems that this type of cracking takes place during the first few years after the concrete is placed, and if not complicated by other types of disintegration, then ceases or at least progresses very slowly. There are a few examples of this type of cracking in the Willamette River Valley where only a few cycles of freezing and thawing occur each year. Fig. 3 shows a handrail post on the Willamette River Bridge at Albany in 1937. No repair work has to be done, and there has been no apparent change in the condition in the last eight years.

At the time these structures were built it was not common practice to make alkali determinations on the cement used. It so happened, however, that most of this kind of trouble developed in concrete made with cement manufactured during the first two years operation of one

Fig. 3—Hand-rail post at the Willamette River Bridge in Albany.

cement plant. Samples were obtained from the corners of a bin which probably represented the first cement stored. These samples showed an alkali content of 1.68 per cent, calculated as Na_20. It is probable that this type of cracking can be attributed to the interaction of a high-alkali cement and a reactive aggregate, as no noticeable cracking of this type has been found when these same aggregates and cements running from 0.6 to 0.8 percent alkali were used. Structures built in the same area with the same aggregates and with cement from the same mill after a change in the source of raw materials, have shown no disintegration of this type.

When the cracking has been serious enough to necessitate repair, the surfaces have been chipped away and the members encased with concrete made with low-alkali cement and aggregates known to be non-reactive. The preparation of the surfaces and the placing of the concrete is carefully done to insure a good bond. Where an encasement exceeding four inches thick is placed, anchor dowels and mesh reinforcement are used.

The cause of this disintegration, which manifests itself by wide cracks, is quite clearly related to the alkali content of the cement and the reactiveness of the aggregates. The fact that it occurs in locations having mild climate and few cycles of freezing and thawing as well as in the colder climates indicates that temperature change is not the primary cause. After this cracking has occurred, moisture penetrates the concrete more easily and disintegration due to freezing and thawing is accelerated. This type of disintegration, while serious in the few structures that have been affected, is not of great concern in Oregon. The cause and means of prevention are known.

During the last 15 years another type of disintegration has developed that is much more serious than the type described above. The first indication of trouble usually shows up within four or five years after construction. A large number of fine cracks appear. These are closely spaced and parallel with the edges of the affected members. The cracks are filled with a grey deposit and are generally called "D-line" cracks. In so far as has been observed, this type of disintegration has been confined to the eastern section of the state and to the higher parts of the Cascade Mountains. It appears to be definitely a function of the number of cycles of freezing and thawing. In the western section of the state where a few cycles only of freezing and thawing occur annually, no trouble has been experienced.

This disintegration, so far, has been confined to handrails, curbs, wing wall tops, and other members above the roadway surface. Members which are protected by the roadway slab have not been affected. These observations point toward the conclusion that the tendency to disintegrate, due primarily to a weakness of the concrete itself, can be overcome, or at least retarded, by waterproofing the surfaces of members exposed to rain or snow.

One significant point brought out in the analysis of data from the frequent inspections of the bridges on the highway system is the correlation between the date of construction and the presence or absence of this "D" line disintegration. A tabulation of all concrete bridges on a 228-mile section of the Old Oregon Trail Highway (U. S. 30) between Umatilla and Ontario was made recently. There are 61 structures in this section. Of these 61 structures, 37 were built prior to 1930 and 24 were built from 1930 to 1937. Of the 37 older structures, only seven show evidences of "D-line" disintegration, and five of these were built with the high-alkali cement previously referred to. The disintegration on the other two structures occurs at the end of a wing wall and in a section of curb where laitance indicates improper placing. Of the 24 structures built from 1930 to 1937, 15 show "D" line disintegration in more or less degree, and four of the other nine structures were waterproofed within two years after their construction.

The construction of these 61 bridges covers a period of 20 years. In general aggregates from the same local sources were used. We are not ready to admit that the later methods of proportioning, placing, and curing are inferior to those used 20 years ago. The remaining ingredient is the cement. At about 1930, finer grinding of cement became general practice to meet the demand for higher early-strength concrete. The strength was obtained, but it seems probable that it was at a sacrifice of durability.

By 1937 the extent of "D-line" disintegration in the highway structure was causing serious concern. The determination of the basic cause of the trouble is too complex a problem for a single state highway organization, and but little progress along this line has been made. A maintenance procedure has been developed, however, which has been quite successful in repairing damage that has already occurred and in preventing further progress of the disintegration. This procedure consists in the removal and replacement of the disintegrated concrete and waterproofing of the surfaces with a linseed oil and white lead paint coat. This procedure follows, in general, the suggestions made by F. R. McMillan, Director of Research of the Portland Cement Association.

The following is an outline of the procedure used by the Oregon State Highway Department in patching disintegrated concrete and waterproofing concrete surfaces for the prevention of disintegration.

When disintegration of the commonly called "D-line" type has taken place, it usually affects the edges and corners of curbs, handrails, wing walls, and other exposed members. To prevent further progress it is necessary to remove completely all disintegrated material, great care being taken to reach sound concrete beyond the extent of the disintegrated area. This can be accomplished by the use of hammers and chisels on small areas or by paving breakers or chipping hammers on larger areas. If the disintegration has reached an advanced stage, the complete removal and replacement of the affected portion of the structure may be necessary. The importance of the removal of all traces of disintegrated material cannot be overemphasized. Experience has shown that often the workman will remove all visible affected material, then place a patch on what was, in his judgment, sound concrete, only to discover later that the material adjacent to the patch continues to disintegrate.

After the removal of the disintegrated concrete a patch is applied, the success of which depends upon securing a bond to the parent concrete, overcoming the tendency of the patch to shrink after placement, and proper curing. All places to be patched should be chipped out to secure not less than three-fourths-inch thickness for the patch. The edges of the patch should be square and not feathered. All surfaces should be clean and rough so as to secure a good bond and should be saturated thoroughly by several applications of water. The preshrinkage of mortar is required for all patches. This is done by mixing the mortar well ahead of use and letting it stand. The time required for preshrinkage of mortar varies with the different brands of cement and conditions of temperature and humidity, and is best determined by experiment on the

Fig 4—Sidewalk on Burnt River Bridge before repairing.

Fig 5—Sidewalk on Burnt River Bridge after repairing.

job. In general the mortar thus preshrunk should be susceptible to use without the addition of water before reworking it for application. After the patch has been applied, proper care in curing must be taken by keeping the patch covered with wet burlap for six to ten hours, after which it can be covered with damp sand or burlap until the concrete has thoroughly hardened.

New concrete patches should be allowed at least two weeks to dry out before applying the waterproofing treatment. New concrete should be

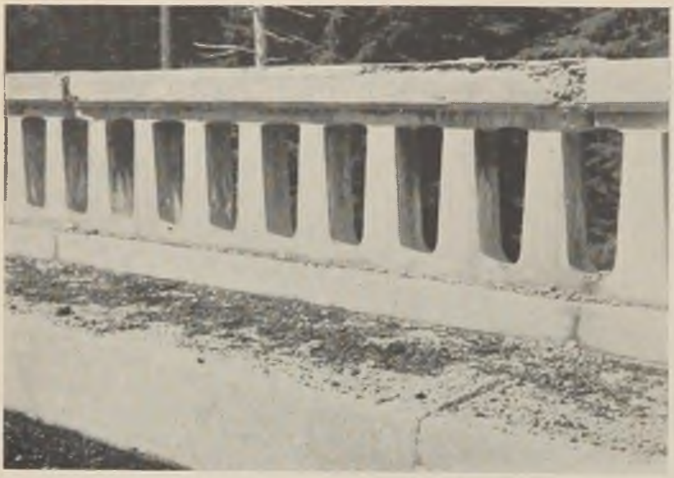

Fig. 6—Handrail on Santiam River Bridge before repairing.

Fig. 7—Handrail on Santiam River Bridge after repairing.

given a neutralizing wash before the application of the linseed oil used in the waterproofing treatment. A solution consisting of three pounds of zinc sulphate crystals to a gallon of water is brushed over the surface to be treated and allowed to dry for 48 hours. When thoroughly dry, all crystals on the surface are removed by wire brushing. This treatment is not necessary on old concrete.

Before the waterproofing treatment is applied, it is necessary that the concrete surface be clean and dry. Dust and loose material can be removed with a wire brush. Road oil or grease can be removed by scrubbing with gasoline or a solvent. Efflorescence can be removed by scrubbing with a ten per cent solution of hydrochloric acid. When water is used in cleaning, ample time must be allowed to permit thorough drying of the concrete surfaces before applying the waterproofing.

After the surface has been properly prepared and is clean and dry, two coats of hot linseed oil are to be applied. The first coat consists of a mixture of 50 per cent raw linseed oil and 50 per cent turpentine heated to 175 F. and applied with an ordinary paint brush. Better results will be obtained if the air temperature is above 65 F. The first coat is allowed to set 24 hours before applying the second coat. After the first coat of the linseed oil-turpentine mixture has set, spots will usually be noticed where the concrete is more porous than the remainder of the surface treated. These spots should be spot treated with the hot mixture and allowed to set before the second coat of linseed oil is applied.

The second coat consists of undiluted raw linseed oil heated to 175 F. and applied in the same manner as the first coat. When this coat is thoroughly dry, the surface is ready for a paint coat.

The entire surface treated with oil is given two coats of white lead and oil paint, tinted to the desired shade. A concrete color can be obtained by the addition of lampblack and raw sienna ground in oil. The white paint used in Oregon has the following formula:

Paint Composition	Per cent
Pigment not less than	70
Vehicle not more than	30
Pigment Composition	
White lead carbonate	40.0 to 45.0
Titanium barium pigment	35.0 to 40.0
Zinc Oxide	15.0 to 20.0
Tinting pigments, if required	0.0 to 5.0

The first coat is thinned by the addition of two quarts of turpentine and two quarts of boiled linseed oil to the gallon oa paint. The second coat is thinned with about one quart of boiled linseed oil to the gallon of paint so as not to give a heavy pigment coat that will be susceptible to scaling, but which is still heavy enough to brush out uniformly and evenly.

This waterproofing treatment is not a cure for the basic weakness of the concrete which makes it susceptible to disintegration. If properly applied and maintained, it promises to postpone serious disintegration for many years.

Fig. 4 shows a sidewalk where the edges have developed "D-line" disintegration and the concrete has crumbled away. Fig. 5 shows the same sidewalk after patching and waterproofing. Fig. 6 shows a handrail with "D-line" disintegration well advanced. Fig. 7 shows the same handrail after the affected concrete has been chipped out and the member built up with new concrete, but before the waterproofing was applied. These figures show that concrete members which have been seriously affected can be repaired and made serviceable and presentable.

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Should Portland Cement Be Dispersed?*

By T. C. POWERS†
Member American Concrete Institute

SYNOPSIS

A development of definitions of *wetting* and *dispersion* is followed by a discussion of dispersion of portland cement.

From elementary principles it appears that a wetting agent is unnecessary, for portland cement is highly hydrophilic.

The dispersed state of portland cement in water is defined as that state in which interparticle attraction in a fresh paste is absent or so weak that it has no appreciable effect on the physical properties of the fresh paste. Experiments and reasoning from general principles indicate that dispersion would be undesirable because it would increase the rate and amount of sedimentation and promote particle-size segregation in cement paste; it would destroy the plasticity of the pastes and give them the properties of a fluid, a probably undesirable change; it would have no beneficial effect on rate of hydration during the early stages through an increase in exposed surface area because the whole surface is normally exposed to water even when the particles are flocculated.

A reduction in interparticle attraction short of actual dispersion should reduce the water required for a given slump, but it would not improve workability except in unusually rich mixes. It would increase bleeding.

Air entrainment requires an increase in paste content and reduction of water content to maintain a given slump. It reduces strength but improves frost resistance. It improves workability and reduces bleeding.

Air entrainment together with some reduction in interparticle attraction affects paste content and water requirement in the same way as air entrainment alone, but the increase in paste content is smaller and the reduction in water content is greater than when there is no reduction in interparticle attraction. Air entrainment offsets the undesirable effects of reducing interparticle attraction on plasticity and reduces bleeding.

Some agents do not affect the chemical processes of hardening; their effects on strength can be predicted from the voids-cement ratio. Others tend to retard hydration unless they contain an accelerating agent. Such agents have different effects with different cements.

*Received by the Institute March 19, 1945.
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INTRODUCTION

This discussion was prepared in response to many requests for information on various admixtures sold as dispersing agents. The questions are usually one of two types: (1) What is the effect of Admixture X on concrete? and (2) What is back of the idea of dispersion of cement? This discussion will deal primarily with the question of what is back of the idea of dispersing cement—not with the actual merits of preparations that have been sold as dispersing agents.

Some materials sold as dispersing agents may have merit because of effects not related to dispersion. However, they cannot be endorsed on the basis of some of the claims made for them. In explaining results obtained or hoped for the assumptions have been made that a normal cement paste is composed of individual flocs (clusters) of cement grains such as may be seen when a small amount of cement is suspended in a large volume of water; also that water for hydration does not penetrate the flocs and hence that the cement cannot hydrate as well as it might; that the water that is "trapped" in the flocs does not contribute to workability. It is then further assumed that the flocs of cement grains can be caused to disintegrate, that is, that the grains in a floc can be dispersed, by adding a suitable agent which causes the grains to acquire electrostatic charges and thus to become mutually repellent; and that when the cement grains are thus dispersed they hydrate more rapidly and to a greater extent than when flocculated. Moreover, it is assumed that when the particles are brought to a state of mutual repellency, the amount of bleeding or "settlement-shrinkage" is reduced. Dispersion is said also to improve resistance to frost action.

Most of these assumptions seem plausible, but they are not compatible with the results of recent researches on the nature of cement paste and on the whole are believed to be untenable on either a theoretical or empirical basis. The basis for this conclusion will be given briefly in this discussion.

The question to be discussed can easily become confused by the fact that some preparations used as dispersing agents are actually mixtures containing not only a dispersing agent but also an accelerator such as calcium chloride, and by the fact that most, if not all, such materials cause more or less air-entrainment. Yet *all* the effects of such mixtures have been attributed, by some, to the supposed dispersion of the cement particles. It is therefore necessary to remember that what may be said in the following discussion against the idea of dispersion does not necessarily impugn any particular preparation unless the predominant effect of the preparation is that of dispersing the cement.

Owing to the manner in which various types of materials of this class have been introduced to the concrete industry, there is some confusion with respect to the terms "wetting agent," "dispersing agent," and "air-entraining agent." Therefore, a secondary aim of this discussion is to clear away some of the confusion with respect to terminology. It is especially important to grasp the fact that certain difficulties surround the use of the term "dispersion" as applied to portland cement. To bring out the nature of these difficulties, it is necessary to present a brief discussion of fundamentals. This discussion leads to a definition of dispersion of portland cement paste on which the final part of the paper is based. The reader is asked to avoid applying the final conclusions concerning dispersion as defined here, to dispersion defined in some other way.

The discussion of fundamentals will be found rather sketchy. For those seeking fuller information a guide to the literature is provided.

THE PHENOMENON OF WETTING*

When a drop of liquid is placed on a clean surface of a solid, it may either spread on the surface or remain as a more or less flattened drop, showing a definite angle of contact with the surface. When the liquid spreads spontaneously, wetting is said to be complete (contact angle = zero) and, if the liquid is water, the solid is called hydrophilic. If water does not spread but establishes a finite contact angle, wetting is incomplete or partial and the solid is regarded as hydrophobic.[1b] Partial wetting, that is, the formation of a contact angle, denotes that the force of adhesion between the solid and the liquid (which tends to cause the liquid to spread over the surface) is less than the surface tension of the liquid, which tends to cause the liquid to gather itself into a drop. Conversely, wetting is always complete when the force of adhesion exceeds the surface tension of the liquid.

A wetting agent is usually a material composed of elongated organic molecules, the molecules having an affinity for the solid at one end and affinity for the liquid at the other. Such an agent forms a layer of molecules on the solid, the molecules being so oriented as to present a wettable surface to the liquid.

When the force of adhesion between a powdered solid and a liquid is less than the surface tension of the liquid, the powder and the liquid are difficult to mix unless a suitable wetting agent is used. Or if the adhesion tension exceeds the surface tension only slightly, a wetting agent will be noticeably helpful. On the other hand, if the solid and liquid show a strong mutual attraction, the liquid will spread over the solid surface without outside aid. For any given case the surface tension and adhesion tension may be measured and the need for a wetting agent

*See Ref. 1 at end of paper.

judged from the results. It is simpler, however, merely to observe directly whether wetting is complete or not. For example, if the specific gravity of the solid exceeds that of the liquid, small particles of the solid, when scattered on the surface of the liquid, will sink readily if the contact angle is very small or zero, or they will remain on the surface if the contact angle is sufficiently large, as does an oiled needle on water. In the case of portland cement and water it is still simpler merely to observe the absorption when water is placed in a crater of dry cement; capillary absorption is readily apparent unless the cement contains a "water-repellent" substance.

Any such criteria applied to portland cement would show that the wetting of portland cement by water could hardly be improved by a wetting agent, unless the cement has acquired a "water-repellent" coating. The degree of solubility of the constituents of cement and, in fact, the rapid formation of hydrates that occurs immediately on contact with water show that portland cement has a strong affinity for water. The attraction is so strong that each cement grain becomes completely surrounded by water even though in a dilute suspension the grains are clustered. This is shown by the fact that during sedimentation of a concentrated suspension, i.e., during bleeding, the *whole* surface area of the cement is effective in regulating the rate of flow of water.[2] So far as the writer knows, no one has seriously contended that cement needs a wetting agent.

When a powder is readily wetted, so that each grain becomes surrounded with water, the grains are necessarily separated, at least by a thin film of water. Because of this, the wetting of a powder by immersion in a liquid is sometimes referred to as dispersion. However, it is advantageous and in line with modern textbook terminology to draw a distinction between wetting and dispersion. As described above, the term *wetting* pertains to the spontaneous spreading of a liquid over a surface. It applies to any shape of surface, not to particles alone. Dispersion, on the other hand, pertains only to particles.* It is the opposite of flocculation, agglomeration, or coagulation. Wetted particles may be in either a flocculated or a dispersed state. Flocculated particles in a liquid may be held together by forces acting across separating films of liquid; hence, a flocculated state is not necessarily one requiring particle-to-particle contact.

These remarks give a general idea of what is meant by dispersion, flocculation, and wetting, but a definition of dispersion wholly adequate for the present discussion cannot be given without considering some additional details of the phenomenon.

*But not necessarily to *solid* particles only.

INTERPARTICLE FORCES

According to present-day theories, the interparticle force in an aggregation of particles in a fluid medium is made up principally of the following components:

(1) An ever-present force of attraction (van der Waal's forces) which causes adjacent particles to adhere; and

(2) An electrostatic force of repulsion that opposes the force of attraction. This repulsion is strongly dependent on the environment of the particles and is therefore subject to control.

The control of interparticle force may involve controlling the kind and concentration of electrolytes, or the use of certain kinds of organic molecules or colloids.

Because of the nature of the relationships between distance of separation and intensity of repulsion and intensity of attraction, two particles may have minimum potential energy when they are separated by a small but definite distance.[3,4] Hence, even in the flocculated state, suspended particles may tend to remain slightly separated.*

DISPERSION

Spontaneous dispersion

Dispersion has been pictured as a spontaneous process whereby particles bearing like electrostatic charges "spring apart and stay apart" by distances easily observed with the microscope. However, even under the most favorable conditions, the forces of repulsion in a suspension are effective over only very short distances. Indeed, if the particles were separated only by the effective distance of repulsion, they would appear to be in contact under ordinary magnification. Nevertheless, if the particles are very small, they may be capable of dispersing themselves by their Brownian motion, far beyond the range of interparticle forces. Brownian motion is a haphazard movement, or vibration, caused by unbalanced impacts of the molecules of the surrounding medium. If a particle is small enough, the forces of the impacts received simultaneously from different directions do not balance and the particle thus acquires motion.

Particles having Brownian motion wander at random and tend to bounce away from each other when they collide. If the forces tending to keep the particles in motion exceed the force of attraction between the particles, then a state of dispersion will spontaneously be maintained. On the other hand, if the forces of attraction between the particles exceed the forces tending to keep the particles in motion, then a state of flocculation, or agglomeration, will persist. Note that interparticle

*For a thorough discussion of interparticle forces, see Ref. 5.

repulsion is not necessary for dispersion; but, of course, the more the interparticle attraction is cancelled by interparticle repulsion, the smaller the amount of kinetic energy required to keep the particles dispersed.

Brownian motion can occur to a significant degree only among very small particles—colloidal particles. Hence, portland cement cannot be caused to disperse spontaneously, for cement particles are predominantly microscopic, not colloidal.[6]

Mechanical dispersion

Even when spontaneous dispersion is not possible, dispersion can be effected mechanically. Stirring a suspension tends to separate the particles, particularly if the stirring is violent. The degree to which the particles will become separated on stirring depends largely on the intensity of the interparticle attraction. Of course, mutually repellent particles should be more easily dispersed than those which have some tendency to stick together.

* * *

The discussion immediately above was presented to show that some of the phenomena pictured in connection with the use of dispersing agents with portland cement actually can occur only among particles that are of truly colloidal dimensions. It must be added that whether or not spontaneous dispersion takes place is of little practical consequence. *The important question is whether it is necessary or desirable to cancel or reduce the forces of interparticle attraction that normally predominate over the forces of repulsion in a suspension of cement particles in water.*

INTERPARTICLE ATTRACTION AND PASTE PROPERTIES

Although the influence of interparticle attraction on the physical properties of pastes will be discussed more fully in the following section, two important effects may be mentioned at this point. The first is that the greater the interparticle attraction, the stiffer will a paste seem to be when it is stirred. In comparatively concentrated suspensions such as cement pastes, where interparticle attraction predominates over repulsion, the suspension behaves more like a solid than a fluid.[7a, 7b]* The other effect appears when suspensions are allowed to settle. If the forces of attraction predominate, the large and small particles settle together. If the forces of repulsion predominate or if the net force of attraction is very weak, particles that would remain in contact when quiescent become separated as they fall through the liquid during sedimentation.

The cause of this may be seen by considering two particles of different size adhering to one another at the beginning of their fall through the

*It behaves like a solid in that it is capable of withstanding small shearing forces. See page 129, "Effect of Flocculation on Plasticity."

liquid. The drag of the liquid on the small particle will be greater per unit mass than that on the large one; therefore, a force tending to separate them will develop. If this force, which depends upon the difference in the size of the particles, exceeds the force of attraction, the particles will separate, the larger particles falling more rapidly. When the attractive forces are very weak, and especially when the particles are mutually repellent, the sediment that is formed tends to be non-uniform in composition, the proportion of coarse particles increasing toward the bottom of the sediment. Moreover, the sediment formed is compact and difficult to redisperse, whereas a flocculated sediment is bulky, soft, and easily restored to its original state in the suspension. This feature of the behavior of the dispersed suspension is very significant with respect to the question of dispersing cement, as will be developed below.

DEFINITION OF DISPERSION APPLICABLE TO CEMENT PASTE

The foregoing discussions serve to show that a criterion for dispersion applicable to colloidal suspensions is not altogether applicable to temporary suspensions of non-colloidal particles; dispersion cannot mean exactly the same thing for both types of suspension. Yet circumstances seem to require using the same term for both cases; indeed, there is considerable justification for it. The dilemma is avoided by thinking in terms of interparticle attraction instead of the repulsion that is implied by the word "dispersion." A suitable definition of dispersion can be set up in terms of the influence that interparticle attraction has on certain properties of suspensions, particularly, the force-flow relationship. When interparticle attraction is absent or negligible, a suspension that is not too concentrated flows like a true fluid; but when interparticle attraction is not negligible, the suspension acquires the properties of a plastic solid to some degree. Also, in a *dilute* suspension of particles, segregation of sizes takes place during sedimentation if the interparticle attraction is absent or weak and it does not take place if interparticle attraction is strong. In view of such observations as these the following definition of dispersion is used in the discussion of cement paste that follows:

> *When interparticle attraction in a fresh cement paste is so weak that it has no appreciable effect on the behavior and physical properties of the paste, the particles in the paste may be said to be dispersed.*

By this definition we would call any suspension dispersed in which the interparticle attraction is zero or negative. This would not disagree with other definitions applied to colloidal suspensions. But we would also call a suspension dispersed if the interparticle force was positive, but too weak to have an appreciable effect on the physical properties of the suspension. It should be noted that the definition does not rest on the presence or absence of particle-clusters; neither does it imply that dispersion is a spontaneous process.

This definition admits the possibility of various degrees of interparticle attraction among the particles in flocculated suspensions, that is, suspensions in which the attraction has an effect on flow-properties, etc. Consequently, we can discuss two questions, one pertaining to the desirability of producing dispersion, and one pertaining to the desirability of changing the intensity of interparticle attraction in a flocculated paste.

DISPERSION OF PORTLAND CEMENT

Flocculated state of normal portland cement paste

As implied at various points in the foregoing discussion, there is no question but that cement particles in a normal paste are flocculated. This may be seen by microscopic examination of cement-water mixtures sufficiently dilute to transmit light or by observation, at low magnification, of the texture of pastes through the wall of a glass container.

The process by which cement becomes flocculated may be visualized as follows: Under the action of a mechanical mixer, cement particles probably tend to be dispersed during the first few seconds of contact with the water. Chemical reactions begin immediately and continue at a relatively high rate for a period of not over five minutes—probably less than two minutes—during which time the electrolyte concentration in the mixing water increases rapidly. The electrolytes (apparently the hydroxyl ions) bring about flocculation of the cement particles.

During the same period a coating of hydrates forms on the cement grains. Once this coating has formed, and the electrolyte solution has reached full strength, the rate of reaction becomes very low and thus the cement remains comparatively dormant chemically. This state lasts for a considerable period, usually about an hour. The paste remains plastic during this period, and, if left undisturbed, undergoes sedimentation ("bleeding").

EFFECT OF FLOCCULATION ON AMOUNT OF SETTLEMENT

Except for certain features that need not be discussed here, the sedimentation of cement pastes has been shown to be essentially like that of other concentrated suspensions of flocculated mineral powders. Various experiments have been carried out that reveal the effect of flocculation on this process. Steinour[2b] observed the sedimentation of emery particles both flocculated and not flocculated. Typical results are given in Table 1:

TABLE 1

Material	Diameter, microns	Fluid Content, %	Amount of Final Settlement, % of original height	
			Dispersed	Flocculated
Emery A	12.2	65	36	17
A	12.2	80	63	44
B	9.6	75	55	35

As shown in the last two columns of the table, flocculation *reduced* the amount of settlement at all concentrations.

The amount of settlement is influenced by the concentration of the flocculating agent. This is deduced from data such as the following obtained by the writer.[8]

TABLE 2—SEDIMENTATION OF PULVERIZED SILICA IN LIME-WATER
(60 ml of solution per 100g of silica)

Initial Concentration of $Ca(OH)_2$ in Mixing Water	Rate of Settlement cm/sec x 10^6	Final Settlement per Unit of Original Height, %
¼ saturated solution	58	10
½ saturated solution	53	5
saturated solution	53	1

Steinour[9] in tests on another silica-powder observed similar but less pronounced effects, the settlement being 12 and 19 per cent for saturated and 1/6-saturated solutions, respectively, with the initial fluid content at 60 per cent.

Results of some experiments with portland cement are given in Fig. 1. This shows the volumes of sediment formed from dilute suspensions (initial fluid-content 86 per cent by volume) of cement in various mixtures of ethyl-alcohol (denatured) and water, ranging from all water to all alcohol. One point represents the results obtained in pure toluene.

The addition of alcohol up to about 50 per cent by volume increased the sedimentation-volume;* higher concentrations of alcohol reduced the volume, finally to a point below that for water alone. All suspensions were flocculated to the extent that no separation of particle sizes could be observed, except the one in straight alcohol. In the latter, most of the cement settled out quickly, the largest particles concentrating toward the bottom of the sediment while the very finest flour remained in suspension, even after 24 hours, probably because of Brownian motion.

*Defined as the ratio of bulk volume of the sediment to the solid volume of the particles.

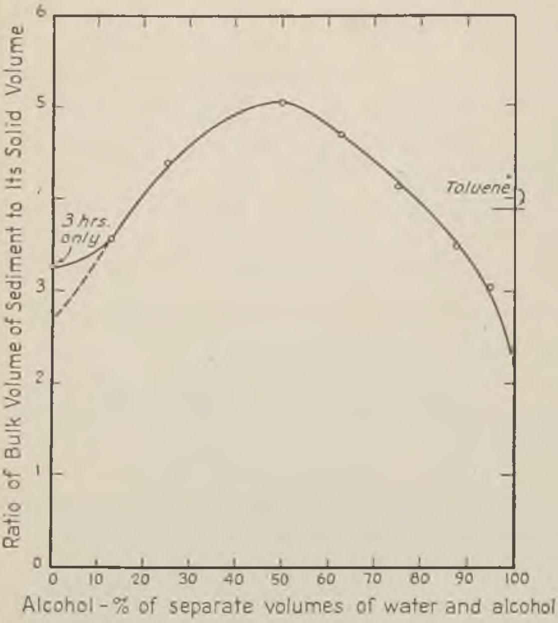


Fig. 1—Sedimentation volumes of cement in alcohol-water solutions and in pure toluene. — Period of settlement 24 hours, except as noted.
Cement 15754

It is reasoned that in settling from dilute suspensions the particle-flocs, in making contact with the sediment, tend to form arches or bridges enclosing relatively large spaces which contribute to the bulkiness of the sediment.[10, 11] The greater the extent to which such arches are able to withstand the pull of gravity, the greater will be the final volume of the sediment. It seems reasonable to assume that the greater the interparticle attraction the greater the strength of the arches and the bulk of the sediment. Therefore, the factors that control interparticle attraction should control sedimentation volume.

The reversal in the slope of the curve of Fig. 1 can be accounted for as follows:* The interparticle force in any given suspension is determined by the kind and concentration of substances in solution and by the force of adhesion between the particles and the liquid medium. Electrolytes in solution or certain types of organic molecules tend to make the particles electrostatically repellent. (Under most circumstances the electrostatic repulsion is at a maximum at very low electrolyte concentrations.) Also, the adhesion of the liquid to the surface and the adsorption of ions or molecules from the solution by the surface tend to cancel some of the particles' surface energy, the surface energy being the source of the force of attraction between the particles. When alcohol is added, it apparently reduces the force of adhesion† between the water and the cement particles

*This explanation is necessarily speculative, since specific data on interparticle forces in this system are lacking. Other explanations can be devised.

†This is directly indicated by the fact that the higher the alcohol content, the lower the dielectric constant of the solution. See Ref. 4.

and thus increases the net force of attraction. At the same time, the addition of alcohol decreases the solubility of the cement constituents and thus favors an increase in electrostatic repulsion, a change tending to offset the effect of the decrease in the force of adhesion between liquid and solid. Evidently, from Fig. 1, one effect is predominant at low alcohol concentration and the other effect is predominant at high concentration.

These data demonstrate that the forces of interparticle attraction in cement-water paste are not as high as they might be and that if a change in the force of flocculation is desired, it could be either an increase or a decrease, according to choice.

Tests with various dispersing agents for portland cement show that they produce a condition similar to that found with cement alone in pure alcohol, but only in very dilute suspensions. Used in concretes or pastes in proportions recommended for field use, they do not cause such dispersion; the pastes clearly show the effects of interparticle attraction. There is evidence, however, that these agents weaken the forces of attraction.

Effect of flocculation on rate of sedimentation

The effect of flocculation on the rate of sedimentation is illustrated in Fig. 2. The curve illustrates the following general rule:

> In *dilute* suspensions, flocculation increases the rate of sedimentation over the average for the dispersed material; in *concentrated* suspensions flocculation *decreases* the rate.

Portland cement pastes as used in practice may be classed as concentrated suspensions; accordingly, *they settle* (*bleed*) *more slowly than they would if the particles were dispersed.*

At high dilution, flocculation increases the rate of sedimentation because the displaced water is able to flow mostly around the flocs, each floc acting much like a single large particle. But at some sufficiently high particle-concentration the particles form a floc-structure so continuous that the displaced water can flow only through the floc-structure itself. In an intermediate range, the flow is partly around and partly through the flocs, a condition giving rise to "channeled bleeding."[13]

The actual conditions in a normal cement paste seem to be about as follows: From considerations already discussed, it seems that before or during the process of flocculation the cement particles become coated with hydrates and the coating acquires a layer of adsorbed water and ions. The particles are so concentrated that when flocculation occurs *they do not draw together into discrete groups* but they form a continuous network. The bonds of the network are, it would be imagined, at points

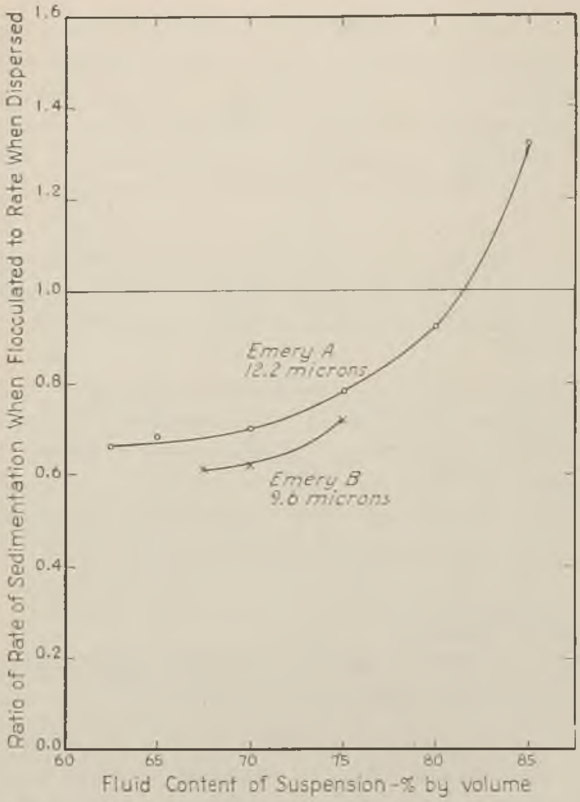


Fig. 2– Effect of flocculation on rate of settlement as influenced by concentration of solids.
(From Ref. 2b)

on adjacent particles that would be in contact were it not for the strongly attached adsorbed layer.

This conclusion is required by at least three conditions:

(1) The nature of the relationship between changes in water content and corresponding changes in the rate of bleeding is such as to show that a change in water content causes a change in the spacing of the network; that is, the effect is *not* that of changing the spacing between clusters of particles.

(2) Steinour's experiments showed that the viscous resistance to sedimentation involves the same amount of particle surface when the particles are flocculated as when they are not flocculated.

(3) Measurements of hydrostatic pressure in cement pastes showed that the particles in a fresh paste are supported entirely by the liquid; that is, none of their weight is transmitted to the bottom of the vessel through point-to-point contacts.

Points (1) and (2) show that the particles cannot exist as separate flocs. Point (3), supported by (1), shows that the particles are normally surrounded with water.

Thus, the evidence is so strong as to be little short of proof that a cement paste is to be regarded as a continuous network of particles in water, the bonds of the network being the forces of flocculation acting across small distances at points of near-contact. In other words, a cement paste may be considered as one large floc; hence, all the water in the paste is within the floc.

So far as the effects of bleeding are concerned the results obtained when the particles are subject to the force of flocculation are clearly preferable to what they are when the particles are free from that force. This becomes doubly evident when it is recalled that during the bleeding of concrete the aggregate soon forms a static framework within the cells of which the paste continues to settle. Even with a normally flocculated paste the settlement is enough to weaken the bond with the undersurfaces of the aggregate particles. Dispersion would not only increase this effect but also would tend to destroy the uniformity of the hardened paste by promoting stratification, as has already been pointed out. If cement pastes were not normally flocculated, it would seem advisable to add a flocculating agent.

From the facts just reviewed it is plain that any claim that *dispersion* is a means of reducing bleeding or "shrinkage before hardening" is based on knowledge of the effect of dispersion on the settlement of *dilute* suspensions and not of the effect on pastes. Also, any deductions based on the assumption that the cement particles in a normal paste exist in discrete flocs from within which water for hydration is excluded are bound to lead to erroneous conclusions, for the evidence is overwhelming that no such condition exists.

Effect of flocculation on plasticity

Flocculation is essential to the plasticity of granular suspensions. When the particles are not flocculated, the mixture behaves like a fluid; it flows under any force, however small, and it can have only the cohesiveness of the liquid itself. When the particles are flocculated, but the dilution is such that individual flocs exist, the mixture probably partakes both of fluidity and plasticity—plasticity increasing with increase in particle concentration. In normal cement pastes, the particles are so concentrated that the behavior of the paste is almost wholly that of a solid; in fact, it meets the definition of a gel, except that it is not colloidal. Under small stresses of short duration it responds both elastically and plastically, by far the greater part of the deformation being plastic.[7] This is true even of pastes thin enough to be poured.

When the strain produced by a sufficiently large force exceeds a limit, the material fails in shear, either with or without dilation, depending on the size and concentration of the particles. Thus when a paste (or a concrete) is forced through a pipe, the deformation is laminar at first (as it is in fluid flow) until a limiting strain is reached, whereupon the material fails in shear at the pipe-wall, the flow from then on being "plug-flow."[13] With dilute pastes, plug-flow can be converted to viscous flow by producing high rates of flow. With concentrated pastes (or concretes) continuous viscous flow cannot be produced because of dilation due to particle interference.

The fact that a normal paste meets the definition of a solid, even exhibiting some elasticity, seems at first to constitute an anomaly, for already it has been shown that the particles are not in direct contact with each other. However, these facts, put together, only support the conclusion that interparticle attractions are effective across intervening layers of water. It seems self-evident that the cohesiveness, or stickiness, of the paste arises largely from these interparticle forces that give the paste its rigidity.

In contrast to the behavior just described, silica in water, cement in alcohol, or any other suspension in which the particles are not at all flocculated—such mixtures act like fluids; they may be mobile, but they are not plastic, and in the absence of entrained air have only the cohesiveness of the suspending medium. Thus, the question as to whether dispersion would be advantageous raises the question as to whether a fluid paste would be preferable to a plastic one.

It has not been demonstrated that concrete made with a *fluid* paste is more workable than one made with a *plastic* paste. On the face of it, the plastic paste would seem much to be preferred. This is indicated by the undesirable results obtained when pulverized silica, in the absence of a flocculating material such as lime, is used instead of cement. It is indicated also, and very strongly, by the nature of the sediment formed from dispersed suspensions. As said before, the dispersed suspension tends to stratify, but in concrete this is probably less serious than the fact that the sediment formed is very compact and rigid, difficult to redisperse, in contrast to a flocculated sediment which may be only a little less plastic than the original suspension. It is plain that during any delay in transportation or placing, a dispersed paste would exhibit very undesirable characteristics.

EFFECT OF WEAKENING INTERPARTICLE ATTRACTION

The discussion up to this point has dealt mostly with comparisons of the dispersed and flocculated states with respect to the properties of fresh concrete. There remains to be considered the effect of varying the interparticle attraction without producing a state of dispersion.

To interpret the test data that will be presented, we need first to observe that, with given materials, the consistency of fresh concrete depends on two factors:

(1) The quantity of paste
(2) The consistency of the paste

With respect to the properties of fresh concrete, entrained air is considered to be a part of the paste.

Among the materials sold as dispersing agents that we have tested, all changed both the quantity and consistency of the paste in a given mix. One material apparently softened the paste but had little effect on paste volume.* Among those agents that influenced both volume and consistency of paste, all caused air entrainment and some apparently reduced interparticle attraction as did the one that had no effect on paste volume. None of these materials when used in concrete in recommended quantities produced dispersion as defined above. That is, they all left the pastes in the flocculated state, but some of them seemed to reduce the intensity of interparticle attraction. (The statements as to interparticle attraction are made tentative because the evidence concerning interparticle attraction is not quantitative, and is somewhat indirect.) The effects of these different types of agents on the composition and properties of fresh concrete will now be discussed.

Effect of an agent that reduces interparticle attraction without entraining air

The influence of interparticle attraction (force of flocculation) on paste consistency is well illustrated by the effect of oleic acid on a mixture of cement and kerosene. When a paste of cement and kerosene is made, we find that interparticle attraction is so strong that the cement particles form a relatively rigid structure, capable of supporting a small weight. Addition of a very small amount of oleic acid during mixing will produce a noticeable change in appearance and a softening of the paste. This change continues as the dispersing agent is added drop by drop, with constant stirring, the paste becoming much like cement-water paste in consistency and texture. In this state it will bleed like cement paste and have similar plasticity. Finally, a single, last drop of oleic acid (in about 500 cc of paste) will cancel interparticle attraction, i.e., it will produce a state of dispersion.

Thus, we see that varying the concentration of a dispersing agent will change the consistency, cohesiveness and bleeding characteristics of a paste even though actual dispersion is not produced.

The effect of the one agent mentioned above that apparently reduced interparticle attraction without entraining air, here called Agent A, is

*This material is not sold as a dispersing agent. Nevertheless, tests show that when used in sufficient amount, it reduces interparticle attraction.

shown in Table 3. These results indicate that when this agent was present, slightly less paste was required for a given slump.* As a matter of fact, this is the only evidence at hand, other than observations of the behavior of very dilute suspensions, that the interparticle attraction was reduced. We reason that since the average aggregate-particle spacing was less when the agent was present, and since the slump was the same, the paste containing the agent must have been softer. This reasoning arises directly from the observation that the higher the water-cement ratio of the paste, and hence the softer the paste, the less paste required for a given slump.

TABLE 3—INFLUENCE OF AN AGENT WHICH REDUCES INTERPARTICLE ATTRACTION WITHOUT CAUSING AIR ENTRAINMENT—AGENT A

Ref. No. Series 305	Agent % Cem't wt.	Cement Content sks/yd³	Slump in.	Absolute Volume Composition of Concrete				
				Aggreg.	Cement	Water	Air	Water + Air
5	0	4.99	6.6	.748	.088	.150	.014	.164
53	1	4.87	5.6	.756	.086	.139	.019	.158
55	2	4.98	5.9	.762	.088	.138	.013	.151
*	0	4.99	5.6*	.752	.088	.146	.014	.160
2	0	6.00	2.2	.738	.106	.138	.018	.156
54	1	5.98	3.0	.740	.106	.136	.018	.154
56	2	5.94	3.5	.744	.105	.134	.017	.151
*	0	6.00	3.2*	.734	.106	.142	.018	.160

*The figures in this line are estimated from Ref. 5 (or 2) on the basis of a 3 per cent change in water content for a 1-in. change in slump.

From the data in Table 3 it may be deduced that when using this particular agent with these materials, a given water-cement ratio can be obtained with about 5 per cent less cement than the amount required when not using the agent.

From the mere fact that Agent A reduced the water requirement for a given slump, we cannot conclude that workability and other properties were benefited. The net effect of using the agent was to replace a given paste with a smaller quantity of a softer paste. To conclude that this constitutes an improvement in workability would require also the conclusion that lean mixes are more workable than richer ones at the same slump, whereas the fact is that richer mixes are preferable under most conditions because of their greater cohesiveness, greater capacity for plastic deformation,† and greater ability to keep the aggregate from settling while the fresh concrete is in transit. A softening of the paste by reducing interparticle attraction would, in concrete of a given slump, give

*With respect to plasticity, air is considered to be part of the paste as it obviously influences the average spacing of the aggregate particles.
†See Ref. 7b.

relatively less of the desirable characteristics just mentioned. Except in very rich mixes where the paste volume is higher than is necessary for satisfactory workability, the general desire seems to be to enhance these properties by stiffening the paste and increasing its volume. Such is the effect of adding mineral powders or increasing the cement content.

In this connection it may be recalled that a cement of high specific surface makes a stiffer, more cohesive paste than does a coarser one at the same water-cement ratio. Yet experience shows that only in very rich mixes is it necessary to use more water (or more paste) when a cement of high specific surface is substituted for a coarser one. This indicates that under some circumstances a stiffening, rather than a softening, of the paste is advantageous. An extreme example of this was reported by Kennedy.[14] He found that although no amount of water would make a sand-and-gravel mixture plastic, air entrained in the mixing water with a suitable agent made the mix appear as if it had been made with cement paste. As will be shown below, entrained air has a stiffening effect.

In general, it seems that increasing the stiffness of the paste, that is, giving it more "body," appears to be advantageous under ordinary conditions in mixes containing less than about 5½ sacks of cement per cu. yd. (1½-in. maximum size aggregate) or, in other terms, it appears advantageous if w/c exceeds about 0.5 by weight.

The lack of benefit from weakening the force of flocculation is probably due to the fact that the cement particles are normally not very strongly flocculated. This was pointed out before in connection with Fig. 1. It seems likely that if cement in water were flocculated as strongly as is cement in kerosene, a reduction in interparticle attraction might be beneficial under most, if not all, circumstances. But, in view of the relative weakness of the flocculating forces in cement-water paste, the value of further reductions in interparticle attraction is debatable.

This point may be emphasized by considering again the effect of reducing interparticle attraction on bleeding characteristics. The effect is to *increase* the amount of settlement (bleeding). However, the *rate* of settlement is affected very little. (See Table 2). The fact that the rate of settlement is affected very little shows that the *initial* texture of the paste is virtually unaffected by reducing the intensity of interparticle attraction. However, the *final* texture, after settlement, would be affected, the pastes having the lowest interparticle attraction forming the densest sediment. This might be advantageous were it not for the fact that any increase in the amount of paste settlement is accompanied by an increase in the depth of the under-aggregate fissures, which fissures weaken the concrete and make it more permeable. To be considered also is the probability that weakening the force of interparticle attraction increases the tendency toward channeled bleeding with its attendant undesirable "sand boils"* and it decreases the ability of the paste to hold the concrete in a plastic state while the concrete is standing or in transit. All effects considered, it seems very doubtful that a weakening of the force of flocculation improves the properties of fresh concrete, even though the slump may thereby be increased, except perhaps in mixes unusually rich in cement.

Effect of an air-entraining agent having little or no effect on interparticle attraction—Agent B

The use of some agents causes a pronounced increase in the air content of cement paste or concrete without appreciable effect on interparticle attraction. The manner in which this is brought about may be explained in simplified version as follows:[15] Any mechanical process that mixes a liquid with a gas tends to form a foam, although the foam may be scant and its life extremely short. If an agent is added that lowers the surface tension of the liquid (a "capillary-active" material), a foam forms more easily and it usually lasts longer than it does without the agent, especially if the agent, by reason of its adsorption at the air-liquid interface, enables the films to withstand shocks. Many organic compounds have this effect when used with water, the soaps being perhaps the most common class. A molecule of soap is relatively large, of the long-chain variety, having a hydrophilic "head" and a hydrophobic "tail." Such molecules tend to collect at the boundary between water and air (or water and oil) and, according to theory, array themselves in such a way that the hydrophilic part remains in the water and the less wettable (hydrophobic) remains in the air. These molecules thus create a boundary film which lowers the surface tension of the water and stabilizes any foam that is formed by mechanical action.

When such materials are used with portland cement paste, the entrained air is probably not to be regarded as a foam, strictly speaking. But the individual bubbles scattered through the paste are no doubt stabilized by the same mechanism that stabilizes the foam; therefore, it is appropriate to speak of such materials as foam stabilizers or air-entraining agents.

Fig. 3 and 4 show the effect of an air-entraining agent. In each diagram the lower line shows how the amount of water decreases as the air content increases. The upper line represents the sum of the volumes of air and water. Since all the mixes represented in a given diagram had the same cement content, the rise of the upper line represents the increase in paste content and the corresponding decrease in aggregate

*In normal bleeding, water flows to the surface uniformly through all the interparticle spaces. In "channeled" bleeding, some of the flow occurs through randomly spaced channels which are the result of ruptures in the normally continuous "mesh" of the flocculated paste (see Ref. 12). The probability of the occurrence of such ruptures increases as the floc-structure is weakened by dilution. Presumably, the probability would be increased also by weakening interparticle attraction by means of a dispersing agent.

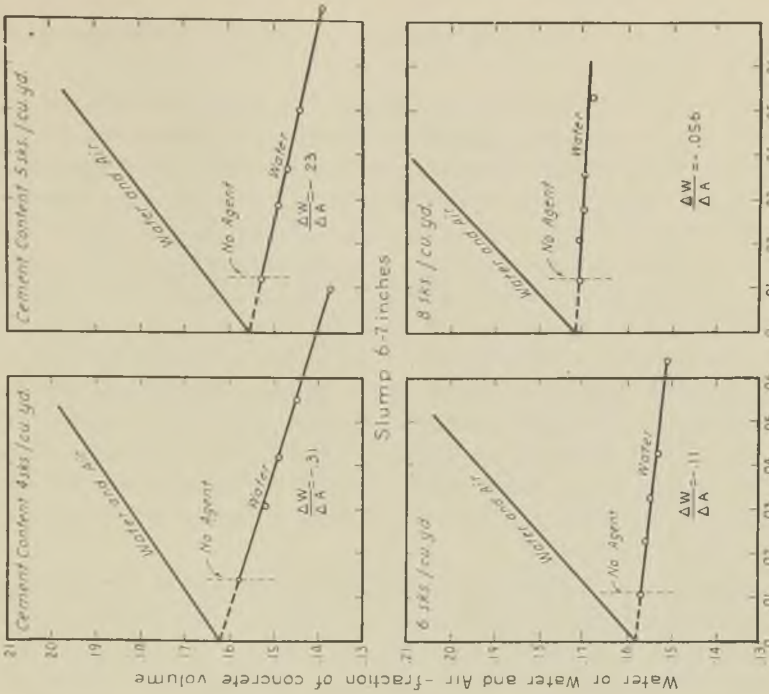


Fig. 4—Effect of an air-entraining agent on water-requirement
Aggregate: Elgin sand and gravel, max. size 1 in.

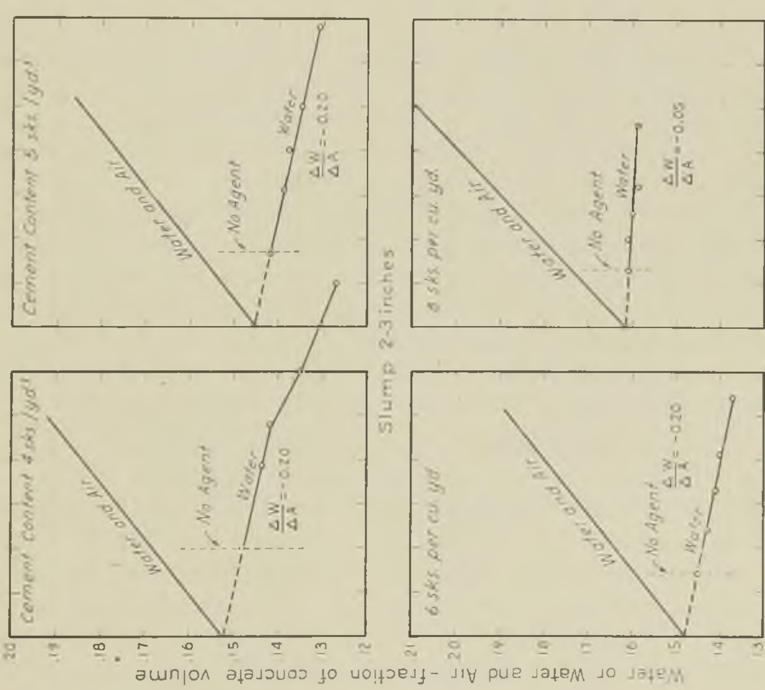


Fig. 3—Effect of an air-entraining agent on water-requirement
Cement: a mixture of four Type I commercial cements.

content. Figs. 3 and 4 represent concretes of different consistencies, as stated in the titles.

In every case the increase in air content was accompanied by a decrease in water content, the decrease being directly proportional to the increase in air content. But since the decrease in water was not as great as the increase in air, the paste content also increased in direct proportion to the increase in air content. If air and water had the same effect on slump, the slope of the water curve in each diagram would have been -1.0; i.e., each per cent of air would have replaced exactly 1 per cent of water and the paste content would have remained constant. But, as shown in the diagrams, each 1 per cent of air replaced only 0.3 per cent or less, and the paste content increased. Since the paste required for a given slump increases in direct proportion to the increase in air content, we must conclude that entrained air stiffens the paste. It follows that when the water content is not reduced, the increase in slump caused by entraining air in a given mix is due to the increase in paste content and not to a softening of the paste.

The conclusion that the paste is stiffened by entrained air is in line with the observed effect of whipping air into cream, or the stiffening that is produced when liquid-liquid emulsions are formed. Indeed, the stiffening effect of entrained air on cement paste was observed directly by Kennedy.[14]

Some idea as to the degree to which entrained air stiffens the paste was deduced from the data in Fig. 3. As a first step in this deduction, Fig. 5 was prepared. The lower diagram shows the relationship between water or water + air content and the cement content for mixes containing definite amounts of air, the curves being obtained by interpolation in the diagrams of Fig. 3. The positions of the lines in the upper diagram in Fig. 5 were then obtained from the lower diagram by adding the volume of the cement to that of water + air. The next step was to assume, as before, that at equal paste volumes and equal concrete slumps the pastes must have the same consistency, and then, by means of Fig. 5, to compare the composition of pastes of the same consistency with and without entrained air. The results for two paste contents are given in Table 4.

These figures show, for example, that a paste for which $w/c = 0.583$ and $air/c = 0.12$ had the same consistency as one for which $w/c = 0.447$ and $air/c = 0$. Thus, introducing 0.12 cc of air per g of cement stiffened the paste as much as would reducing w/c from 0.583 to 0.447, i.e., reducing the water 0.136 cc per g of cement. It is not likely that pastes of these compositions would show exactly the same consistency if tested separately from the concrete, for, as indicated by Kennedy's experiments, effectiveness of the air is probably influenced by the aggregate. Never-

TABLE 4

Air Content of Concrete	w/c Abs. Vol.	w/c Wt.	$\frac{\text{Air} + \text{w}}{\text{c}}$ cc per g	$\frac{\text{Air}}{\text{c}}$ cc per g
Paste = 0.25				
0.00	1.42	.447	.447	0
0.01	1.46	.460	.495	.035
0.02	1.54	.485	.555	.070
0.03	1.85	.583	.703	.120
Paste = 0.27				
0.00	1.31	.413	.413	0
0.01	1.33	.419	.448	.019
0.02	1.36	.429	.488	.059
0.03	1.38	.434	.531	.107
0.04	1.40	.441	.575	.134
0.05	1.66	.523	.718	.195

theless, the data show conclusively that entrained air has a stiffening effect even though it is more fluid (i.e., it has a lower viscosity) than the water it displaces. A correct explanation of the effect would undoubtedly involve the surface tension at the numerous air-water interfaces.

Returning to Fig. 3 and 4, we may note that the point for the plain mix in each diagram falls in line with the other points. This indicates that the agent had no softening effect on the paste, that is, it did not reduce interparticle attraction. Had there been a reduction in interparticle attraction the line would have passed below the point for the plain mix.

In general it appears that the lower the cement content the greater the reduction in water requirement per unit increase in air content. This is shown as a steady trend in Fig. 4, but in Fig. 3, representing the stiffer consistency, the effect is equal in the 4-, 5-, and 6-sack mixes.

So far as workability and bleeding characteristics are concerned, the effect of entrained air may be regarded as highly beneficial. It increases cohesiveness, it aids in holding the aggregate from settling while the concrete is being transported, and the increase in paste volume increases the capacity for plastic deformation.

Entrained air reduces strength, but greatly increases resistance to frost action.[16]

Effect of an agent that both reduces interparticle attraction and entrains air—Agent C

The effect of an agent of this type on water requirement and air content is shown in Fig. 6. The data are fewer than might be desired,* but

*Although many tests have been made on various agents in this laboratory, only a few have included several proportions of a given agent.

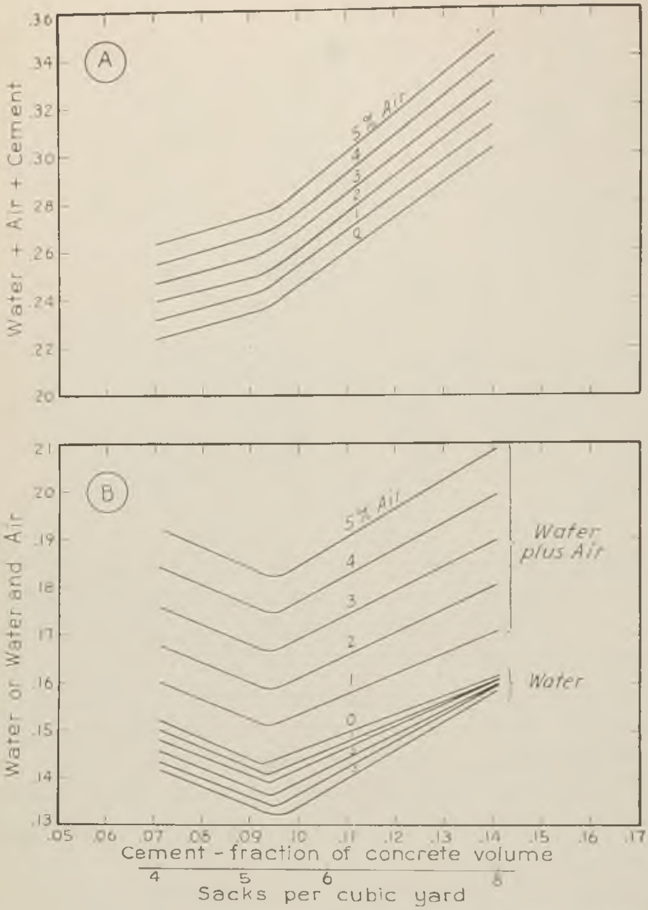


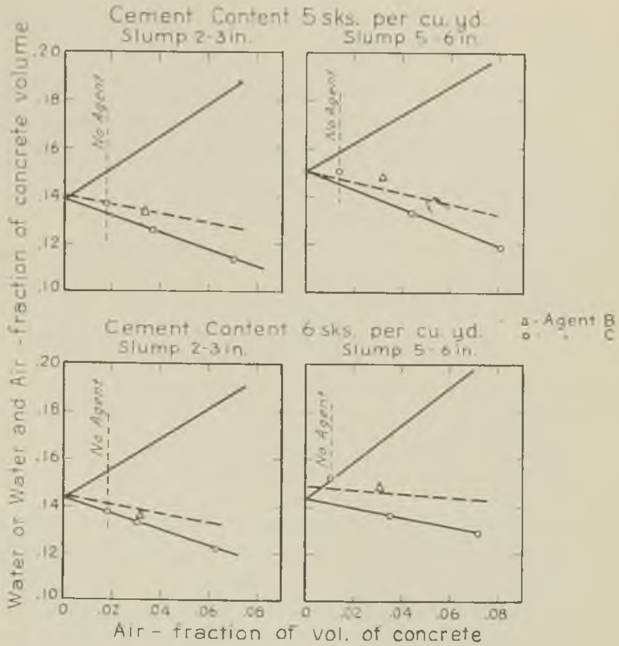
Fig. 5—Effect of an air-entraining agent on water, air, and paste content—slump 2-3 in.

their indications are such as would be expected from the discussion of the two other types presented above. In Fig. 6 the solid lines represent the water contents and water + air contents as in Fig. 3 and 4. The broken line is drawn to have the same slope as the lines in Fig. 3 or 4, for corresponding slumps and cement contents, and to pass through the point representing the plain mix, except where adjustments were made to compensate for inequalities in slump. Thus the broken line represents the effect of an air-entraining agent, Agent B, and the solid lines, the effect of the agent which both reduces interparticle attraction and entrains air, Agent C. The triangular point in each diagram represents the one mix of each kind in this series that was made with Agent B.

Although the four diagrams are not wholly consistent with each other in all respects, they show that Agent C reduced the water requirement more than did Agent B. Presumably, this is the effect of a reduction in

Fig. 6—Effect of an agent that entrains air and reduces interparticle attraction.

Cement: a mixture of four Type I commercial cements. Aggregate: Elgin sand and gravel, max. size 1½ in.



interparticle attraction caused by Agent C, as indicated by points for the plain mixes falling above the line for Agent C.

It is clear, however, that even if Agent C did reduce the interparticle attraction, the net effect of this and the air entrainment was a stiffening of the paste. This is shown by the fact that with Agent C, as with Agent B, the paste content required for a given slump increased in direct proportion to the increase in air content. In view of this, it may be concluded that whatever undesirable effects on plasticity and bleeding characteristics the reduction in interparticle attraction might have, they are offset by the effect of the entrained air.

On the whole, it appears that *with respect to the properties of fresh concrete,* agents like B and C are beneficial, C being somewhat more so, for with an agent of this type, concrete of a given slump and air content can be obtained with less water in the concrete.

Whether or not the properties of the *hardened* concrete are benefited equally, or at all, by agents of this type is a question that cannot be dealt with in general terms. Experience indicates that some agents have no effect on the chemical processes of hardening and therefore the effects on strength can be predicted fairly well from the effect on the voids-cement ratio. Other agents, especially those that influence interparticle attraction, are liable to retard hydration and, apparently for that reason, some are sold with an added accelerator. Agents of this type have different effects on different cements.

With respect to durability, entrained air is decidedly beneficial, and therefore the use of agents like B and C can be recommended where special protection against frost action is necessary. The beneficial effect of entrained air on durability is probably the result of providing room for expansion of water during the process of freezing.[16b] It is difficult to see how either dispersion, per se, or a weakening of the forces of flocculation in the absence of air entrainment, could have much influence on frost resistance.

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An Investigation of the Strength of Welded Stirrups in Reinforced Concrete Beams*

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Member American Concrete Institute

SYNOPSIS

The results of the tests of 44 beams of reinforced concrete with stirrups welded to the longitudinal reinforcement are presented. The beams were designed in such a way as to produce failure by diagonal tension. Variables including the size and inclination of the stirrups, type of concrete and ratio of longitudinal reinforcement are studied. A comparison of the strength of welded stirrups with that of loose stirrups, as reported from former tests on web reinforcement, is attempted.

1—INTRODUCTION

The use of welded stirrups as web reinforcement for concrete beams, instead of the conventional loose U-stirrups, should provide improved construction, since it gives perfect continuity to the reinforcing steel embedded in concrete and facilitates accurate steel-setting. Its practical use in units formed by longitudinal bars and stirrups welded together has been suggested as economically possible.‡

Two main questions arise in connection with the problem: (a) Will beams containing stirrups rigidly connected to the main reinforcement develop higher shearing strengths than those using common loose stirrups? (b) Can the higher theoretical efficiency of certain inclined stirrups be utilized through the use of welded units?

A favorable answer to these questions is important, since the economical use of welded stirrups will depend to a great extent upon the advantages to be secured by the use of such web reinforcement.

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‡D. M. McCain, "Welded Shear Reinforcing for Concrete Beams," *Civil Engineering*, July, 1939.

Probable advantages in strength due to welded stirrups lie in improved anchorage of the stirrups, prevention of slipping of inclined stirrups along the main bars and improved anchorage of the main bars.

The advantage to be secured by the use of certain inclined stirrups is indicated theoretically, as shown by the following summary of the usual analysis of web stresses. In such analysis, it is common to assume that the action of a reinforced concrete beam may be likened to that of a truss in which the top chord is formed by the compression zone of the concrete, the bottom chord by the longitudinal reinforcement, the tension web members by the stirrups, and the compression web members by portions of the concrete web of the beam. The assumption is also made that the compression web members are inclined at 45 deg. to the axis of the beam. The analysis shows that the strength of stirrups having a given volume of metal per unit volume of concrete depends on the inclination of the stirrups with respect to the axis of the beam.* Also according to the analysis, vertical stirrups and stirrups at 45 deg. are equally strong. The maximum strength is given by stirrups inclined at 67.5 deg. If the strength of the beam with vertical stirrups or stirrups inclined at 45 deg. is taken as unity, the analysis shows for stirrups inclined at 67.5 deg. a beam strength about 20 per cent greater. The inclination to be given to the welded stirrups should be the one that gives the maximum strength.

This paper contains the results of the tests of 44 beams of reinforced concrete, loaded at the third points with a span of 8 ft., and designed to produce diagonal tension failure. The tests were intended to give results showing the strength of welded stirrups and to point out which is the best inclination, with respect to the axis of the beam, to be given to those stirrups. The following variables were introduced in the investigation: (a) type of concrete, (b) ratio of web reinforcement, (c) inclination of the stirrups, and (d) ratio of main reinforcement, which was varied in a supplementary series of tests on four beams. These four beams are treated separately throughout this paper and are referred to as Series 1a. A comparison is also made with former tests on loose stirrups.

Notation

The following notation is used, particularly in the headings of Tables 1 to 3:

f'_c = average compressive strength of 6 by 12-in. concrete control cylinders (3 for each beam)

f_y = yield point stress of web reinforcement

$p = \frac{A_s}{bd}$ = steel ratio of longitudinal tension reinforcement

*F. E. Richart, "An Investigation of Web Stresses in Reinforced Concrete Beams," University of Illinois, Bulletin No. 166.

$r = \frac{A_v}{ab}$ = ratio of web reinforcement, where A_v is the area of the two single stirrups at a cross-section, a is the stirrup spacing, measured normal to the direction of the stirrup, and b is the width of the beam

k = ratio of the distance from the neutral axis to the top of the beam, to the effective depth d

K = $(\sin \alpha + \cos \alpha) \sin \alpha = 1.0$ when $\alpha = 45$ deg. or 90 deg., and 1.20 when $\alpha = 67\frac{1}{2}$ deg.

α = the inclination of the stirrups to the axis of the beam

P_u = ultimate load on beam

V_c = vertical shear (one-half the load on the beam) at which diagonal cracks were first noted

V_u = vertical shear (one-half the ultimate load) at ultimate load

V_y = vertical shear (one-half of the load on the beam) at which yield point stress was reached in web reinforcement

$v_c = \frac{V_c}{bjd}$ = shearing stress at which diagonal cracks were first noted.

$v_u = \frac{V_u}{bjd}$ = shearing stress at ultimate load

$v_y = \frac{V_y}{bjd}$ = shearing stress corresponding to vertical shear, V_y

v_u *calc* = shearing stress v_u, calculated from Eq. 8.

v_y *calc* = shearing stress v_y, calculated from Eq. 6.

2—DESCRIPTION OF TESTS

Relatively few of the specimens heretofore tested to determine the strength of web reinforcement in beams of normal dimensions have had a definite diagonal tension failure. Initial failure by bond has played an important part in the behavior of those beams, resulting in a failure due to the simultaneous effect of bond and diagonal tension. Many of them have failed in tension, pointing out the necessity of using a fairly large amount of tension steel, when the web reinforcement is of some importance, in order to avoid this kind of failure.

The specimens used in this investigation were designed to produce a true diagonal tension or shearing failure and to avoid as far as possible all other influences. Care was taken to eliminate all variables except those being studied. The beams were all 10 ft. long, 5.5 in. wide, and 21 in. deep. Forty of the beams were reinforced longitudinally with four 1-in. square bars placed in two layers, with an effective depth of 18.25 in. The other four beams were reinforced with two 1-in. square bars, with an effective depth of 19.5 in.

Three series of beams were tested: two groups, each of twenty beams, were identical except for the type of concrete. In Series 1, Mix No. 1 (1:3.34:5.0, by weight, with a water-cement ratio of 0.75 by weight) was

TABLE 1—OUTLINE OF BEAM TESTS

Two companion beams of each type shown. Total 44 beams. Stirrup spacing 6½ in. normal to direction of stirrup. All bars of intermediate grade, deformed, except ¼-in. bars which were plain. Portland cement concrete; aggregates, sand, and gravel from Wabash river. Tension reinforcement, 1-in. square bars, having average yield point stress of 48,000 psi.

Series No.	Designed Concrete Strength psi.	p, per cent	r, per cent									
			None	45 deg. Incl.			67.5 deg. Incl.			90 deg. Incl.		
				¼-Ø	⅜-Ø	½-Ø	¼-Ø	⅜-Ø	½-Ø	¼-Ø	⅜-Ø	½-Ø
1	3500	4.00	0	0.28	0.615	1.12	0.28	0.615	1.12	0.28	0.615	1.12
2	4500	4.00	0	0.28	0.615	1.12	0.28	0.615	1.12	0.28	0.615	1.12
1a	3500	1.88	.	....	.....	....	....	.....	....	0.28	0.615	

used. In Series 2, Mix No. 2 (1:2.79:4.18, by weight, with a water-cement ratio of 0.64) was employed. In the third group, Series 1a, consisting of four beams, Mix No. 1 was used. Table 1 lists the design features of the test beams, and Fig. 1 shows the general dimensions, details of reinforcement, and location of strain gage lines for these beams.

The longitudinal and web bars were first welded together in units, as shown in Fig. 2a and b. Two units, forming the complete reinforcement for a beam, were then joined at the proper distance by welding on small transverse spacing bars, as shown in Fig. 2c.

The beams and control cylinders were cured for 21 days in the moist room, then were stored in the air of the laboratory until they were 28 days old, when they were tested.

Strain measurements were taken on 21 gage lines (19 in Series 1a), located as shown in Fig. 1. Sixteen of these gage lines were on the stirrups, eight on each side of the beam, located symmetrically. The rest of the gage lines were used to measure deformations in the main reinforcement and in the concrete at midspan. A Berry strain gage, of 5-in. gage length, was used for this purpose. The ultimate load was reached in six to ten increments, a complete set of readings being taken at each increment. The deflections of the beams, and the load at which the diagonal cracks started, were also recorded during the progress of the tests.

In designating the beams, the following notation was used: The first number (1, 2, or 1a) indicates the series. The following letter specifies the type of web reinforcement: N, no stirrups; V, vertical; I, 67.5 deg.

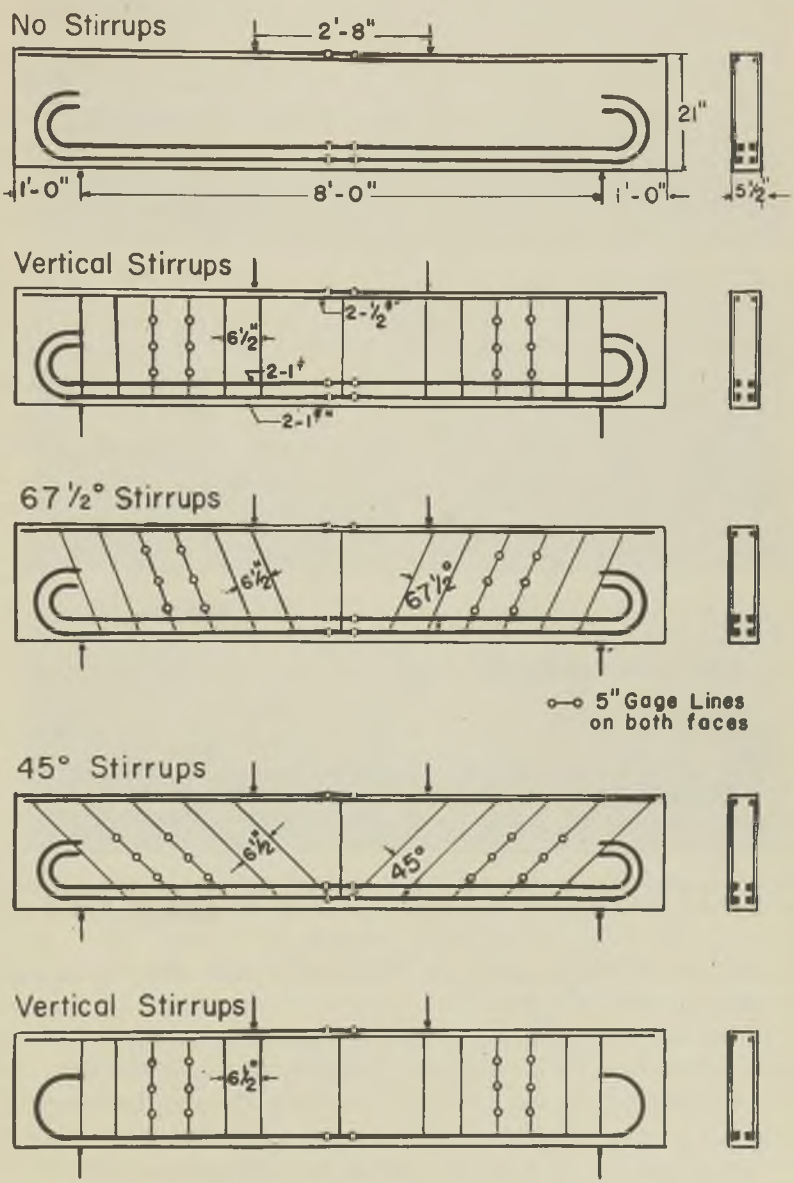

Fig. 1—General dimensions, details of reinforcement and location of strain gage lines of beams.

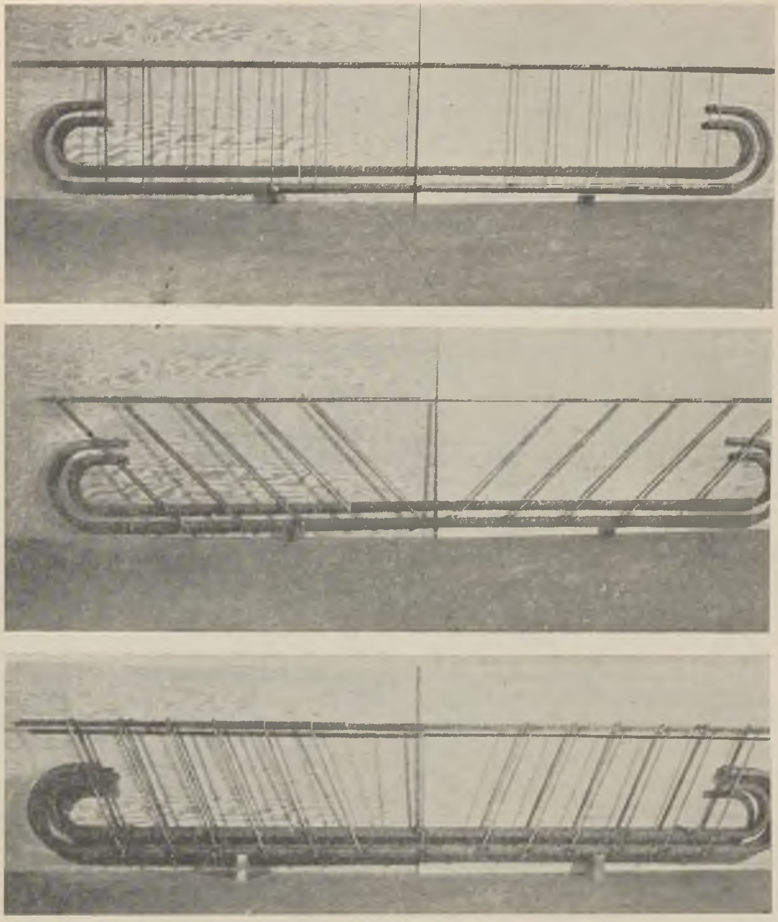

Fig. 2—View of the reinforcement of beams. (a top) Unit with vertical stirrups, (b center) Unit with 45 deg. stirrups, (c bottom) Complete reinforcement of a beam with 67.5 deg. stirrups.

inclination; D, 45 deg. inclination. The fraction represents the diameter of the stirrups used. For example.

1-N indicates: Series 1, beam without web reinforcement.

2-D ½ indicates: Series 2, ½-in. stirrups at 45 deg. inclination.

1-I 3/8 indicates: Series 1, 3/8-in. stirrups at 67.5 deg. inclination.

1a-V ¼ indicates: Series 1a, ¼-in. vertical stirrups.

The phenomena of the tests showed the following general picture: Up to a load of about 50,000 lb., corresponding to a computed shearing stress of 300 psi, no diagonal cracks were developed. The stresses in the stirrups were very small, the diagonal tension being taken up by the concrete, which was still uncracked.

For the beams without web reinforcement, after the diagonal cracks started, a main crack developed on each side of the beam. These cracks, which were inclined at about 45 deg. to the horizontal, started in the lower half of the beam and progressed upward as the rate of deflection of the beam increased. At ultimate load, the upper portion of the beam sheared off in the compression zone.

For the beams with web reinforcement, the diagonal cracks started in the lower half of the beam as before. As the load increased, these cracks extended upward, while new cracks that started at the bottom of the beams extended vertically for a while, then turned at about 45 deg. and ran approximately parallel to the other diagonal cracks. The number of cracks developed and the distance between them was, on the average, dependent on the ratio of web reinforcement. At ultimate load, the beams with ¼-in. stirrups had a main diagonal crack on each side, at each end, with relatively few cracks parallel to these. In the beams with 3/8-in. stirrups, the diagonal cracks were more numerous and the distance between them was, on the average, smaller. All beams with ½-in. stirrups failed in compression when the stress in the web reinforcement had barely reached the yield point, so that the diagonal cracks were not fully developed; however, at ultimate load, the number of cracks was generally greater than for the other beams.

In general, almost all of the diagonal cracks were well started at the load at which the web reinforcement was stressed to the yield point. Increasing the load merely opened up these cracks, extending them upward untii the ultimate load was reached by shearing off the upper portion of the beam.

All the beams failed by diagonal tension except those with ½-in. stirrups, in which failure was due to crushing of the concrete in compression, and those of Series 1a with 3/8-in. stirrups, in which failure was due to the simultaneous effect of tension in the main steel and diagonal tension. For all beams that failed by diagonal tension, failure occurred in about the same way. One of the two main cracks at each side of the specimen, which had extended over about 7/8 of the depth, started to open up rapidly at mid height as the stirrups yielded, developing at both ends until the remaining uncracked concrete at the top of the diagonal crack failed by shear, causing a sliding of the two parts of the beam along the crack.

3—SERIES 1 AND 2, ANALYSIS OF TESTS

Experimental data

Table 2 gives properties of the materials used in the beams, together with the principal results of the tests. Fig. 3 gives average curves show-

TABLE 2—RESULTS AND ANALYSIS OF TESTS

(Two companion beams of each type shown)

Beam No.	f'_c psi.	f_y* psi.	k	v_c psi.	P_u lb.	v_u psi.	V_y lb.	v_y' psi.	v_u-v_y psi.	v_y-Krf_y psi.	v_y calc. psi.	Ratio $\frac{v_y \text{ calc.}}{v_y}$	v_u calc. psi.	Ratio $\frac{v_u \text{ calc.}}{v_u}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
							Series 1	and 2						
1-N	3585			261	79,000	474								
2-N	4490			321	84,500	507								
1-V ¼	3460	55,000		300	116,600	699	44,705	536	163	382	492	0.914	700	1.001
2-V ¼	4805	55,000	0.507	339	135,100	810	45,100	541	269	387	546	1.012	834	1.065
1-I ¼	3795	55,000	0.555	300	120,950	725	44,555	534	191	349	537	1.006	765	1.054
2-I ¼	4700	55,000	0.516	312	132,000	792	46,450	557	234	372	572	1.027	854	1.086
1-D ¼	3475	55,000	0.541	282	118,550	711	42,180	506	205	352	492	0.972	701	0.985
2-D ¼	3455	55,000	0.578	318	140,550	843	44,520	534	309	380	492	0.924	699	0.830
1-V ⅜	3115	47,900	0.565	345	146,850	881	54,450	653	228	358	624	0.955	810	0.919
2-V ⅜	4120	47,900	0.535	300	143,650	862	53,860	646	215	351	660	1.022	907	1.054
1-I ⅜	3575	47,900	0.559	378	159,000	954	60,855	730	223	376	697	0.949	911	0.955
2-I ⅜	4185	47,900	0.539	324	168,000	1008	59,915	719	289	364	721	0.992	972	0.969
1-D ⅜	2855	47,900	0.552	300	129,750	778	52,155	626	152	331	610	0.971	780	1.002
2-D ⅜	3685	47,900	0.578	300	143,650	862	53,185	638	223	338	642	1.007	863	1.002
1-V ½	3665	50,700	0.508	330	157,000	942	74,240	891	Compression Failure	323	915	1.027		
2-V ½	4815	50,700	0.535	267	186,400	1118	82,125	985		417	960	0.977		
1-I ½	3090	50,700	0.578	360	177,750	1066	83,080	997		315	1005	1.009		
2-I ½	4210	50,700	0.560	360	196,200	1177	89,280	1071		389	1050	0.980		
1-D ½	2705	50,700	0.569	315	155,000	930	74,850	900		332	876	0.974		
2-D ½	3495	50,700	0.549	360	175,825	1055	73,880	886		318	907	1.023		
							Series 1a							
1a-V ¼	3540	46,000	0.383	195	106,100	558	35,425	372	182	243	364	0.978	576	1.031
1a-V ⅜	3335	52,000	0.397	179	117,350	617	50,395	530		210	547	1.034		

*Determined by extensometer tests on samples of bars.

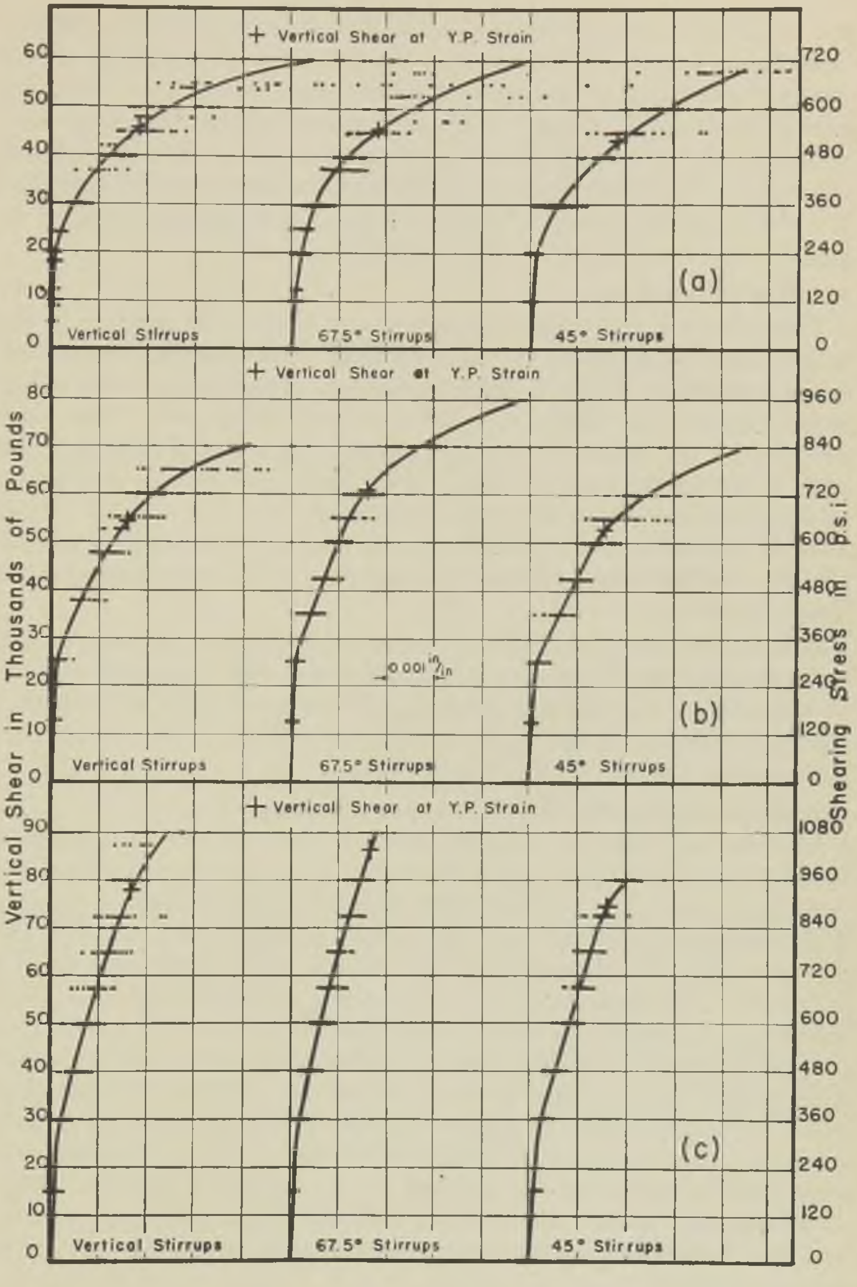

Fig. 3—Average 'shear-strain' curves for the web reinforcement. Series 1 and 2. a. ¼ in stirrups, (b) ⅜ in. stirrups, (c) ½ in. stirrups.

ing the relation between vertical shear and strain in the web reinforcement for each type of beams tested. In the right hand margin of each figure, the shearing stress corresponding to the vertical shear (computed as explained later) is shown. The plotted points in those figures represent actual values from the tests. In plotting these curves, and in analyzing the results, only about half of the measured strains (those which reached the yield point first) were considered. These were generally located on gage lines across or near the principal diagonal cracks.

Position of the neutral axis

From the flexural strains determined at midspan, the position of the neutral axis was computed for each loading. The values so determined, and corresponding to the last five increments of loading at which strain measurements were taken, were averaged and are given in Table 2 for each beam. As can be seen from this table, k ranges from 0.507 to 0.578 with a general average of 0.549. This figure corresponds practically to the value of k computed for the cracked section with $n = 10$. From $n = 10$, the value of $j = 0.833$ was determined for the cracked section and this constant value was used to compute the shearing stresses for the beams of these two series.

It is realized that this value of j may not be the actual value at the outer thirds of the beams, but it is in agreement with the usual practice in determining the shearing stresses. Furthermore, it is not believed that the lever arm at the outer thirds is very different from that at the center when diagonal cracks have developed.

Average shear at the yield point stress in the web reinforcement

The vertical shear at which the strain in the web reinforcement was equal to the known yield point strain for the steel was determined for each gage line. The smallest eight shears so selected were averaged. This average is arbitrarily defined as the "average shear at which the stress in the web reinforcement reached the yield point" and will be called V_y. This shear and the corresponding shearing stress have been selected as the basic values for the analysis of the tests. This selection and the consideration of that shearing stress as the basic value on which a permissible stress should be based is just one criterion for that purpose. Although all of the beams that failed by diagonal tension took more load than necessary to stress the stirrups to the yield point, this extra load was taken at the expense of an excessive increase in width of the diagonal cracks. It remains to be determined whether a shear V_y sustained indefinitely would not produce failure of the beam.

Method of analyzing results

In Table 2, the analysis of the results of the tests for these two series is presented. Columns 1 to 10 of this table give various quantities that

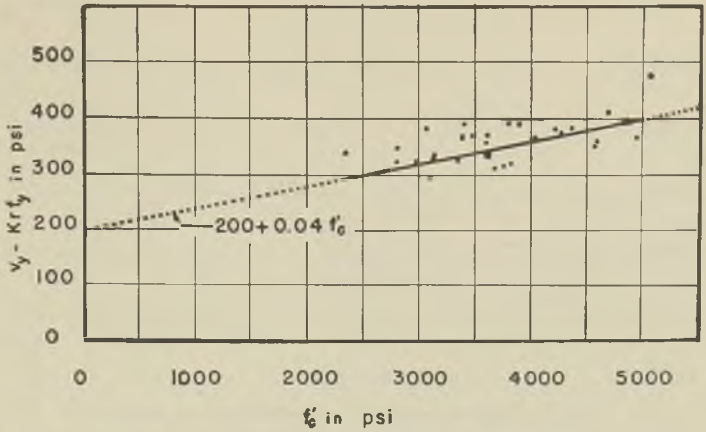

Fig. 4—Relation between v_y—Krf_y and concrete strength.

have been determined by the tests. They are self-explanatory or have been explained previously. In column 11 are given the values of:

$$v_y - Krf_y$$

Krf_y is the theoretical shearing stress at the yield point strength of the web reinforcement when the analogy with a truss is made in the analysis of the action of stirrups in reinforced concrete beams.*

The values of $v_y - Krf_y$ have been plotted in Fig. 4 against the concrete strength as determined from standard test cylinders. Through the points so determined, a straight line has been passed, for which the equation is

$$v_y - Krf_y = 200 + 0.04f'_c \text{ (in psi)}$$

The shearing stress at which the stirrups are stressed to the yield point, as defined previously, is thus represented fairly well, for these series of tests by the following equation:

$$v_y = Krf_y + 0.04f'_c + 200 \text{ (in psi)} \ldots\ldots\ldots\ldots\ldots (1)$$

The shearing stresses determined by this formula are shown in Column 12 of Table 2 and are denoted as "v_y *calc.*" Column 13 of the same table gives the ratio between the shearing stresses calculated by formula (1) and those derived from the experimental value V_y.

The calculated and experimental values of v_y agree more closely than could generally be expected for tests on reinforced concrete beams. When individual beams are considered, the ratio between calculated and experimental values ranges between 0.914 and 1.027. The general average for the beams with ¼-in. stirrups is 0.976, for those with ⅜-in. stirrups,

*See, for example, Bulletin 166, University of Illinois, loc. cit.

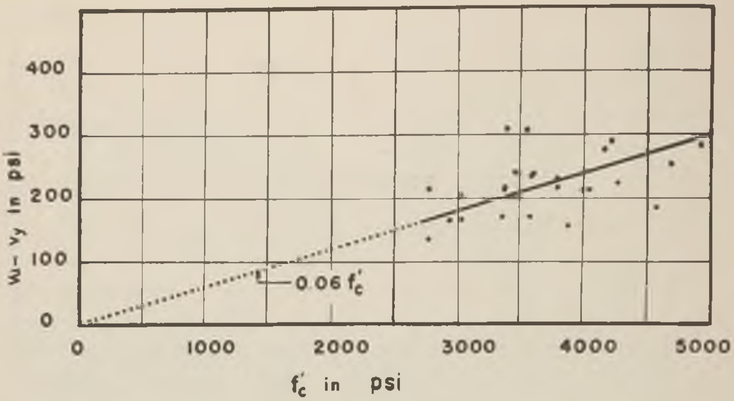

Fig. 5—Relation between v_u—v_y and concrete strength.

0.983, and for those with ½-in. stirrups, 0.998. The general average for all the specimens is 0.986.

Column 10 of Table 2 shows the difference between the experimental shearing stresses, $\frac{V}{bjd}$, at ultimate load and at V_y. This difference, $v_u - v_y$, has been plotted against concrete strength as abscissas in Fig. 5. A straight line has been passed through the points so determined. The equation of this straight line is:

$$v_u - v_y = 0.06f'_c$$

substituting v_y as given in equation (1):

$$v_u = Krf_y + 0.10f'_c + 200 \quad \text{.....................(2)}$$

With this formula, the shearing stresses at ultimate load have been computed for each beam, are given in Column 14 of Table 2 and are denoted as "v_u *calc.*" Column 15 gives the ratio between the shearing stresses computed by formula (2) and those determined experimentally. The agreement between calculated and experimental values is again very good, with ratios that range from 0.830 to 1.086 for individual beams. The general average for the beams with ¼-in. stirrups is 0.998, for those with ⅜-in. stirrups, 0.983, and the general average for all the beams that failed by diagonal tension, 0.99.

The measured deflections at the center of the beams were compared with the theoretical deflections computed by Maney's equation.* For a beam loaded at the third points, this equation is

$$D = \frac{23}{216}\frac{l^2}{d}(c + t)$$

*Maney, G. A., "Relation Between Deformation and Deflection in Reinforced Concrete Beams," Proc. A.S.T.M. Technical Papers, Vol. 14, p. 310, 1914.

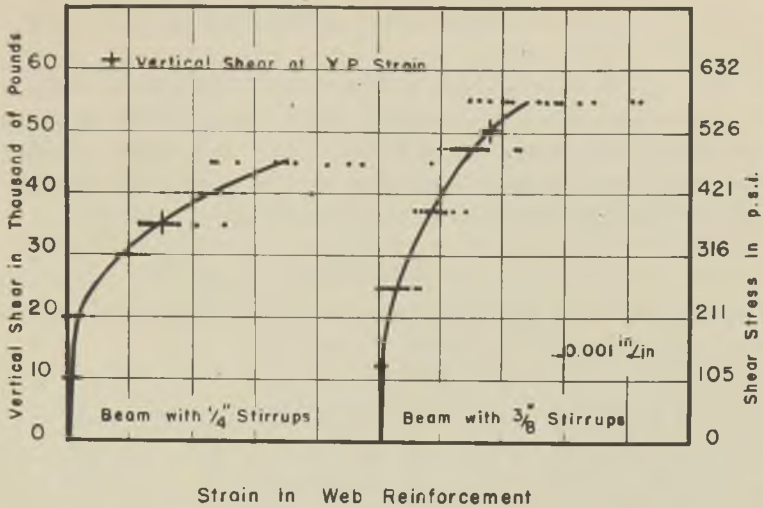

Fig. 6—Average "shear-strain" curves for the web reinforcement. Series 1a.

in which l is the span, d the effective depth, c the strain at the top fiber, and t the average strain in the tension steel.

The measured deflections at ultimate load were in certain cases as much as 40 per cent greater than the computed. At the load corresponding to V_y, the measured deflections were on the average 10 to 20 per cent greater than the calculated. Part of this disagreement between measured and calculated deflections is due to the effect of the shearing deflections, and the other part is believed to be due to the opening of the diagonal cracks, which Maney's equation does not take into account.

4—SERIES 1a, ANALYSIS OF TESTS

This series of tests was made with the purpose of studying the effect of varying the ratio of main reinforcement. The four beams of this series were entirely similar to those of Series 1 with vertical stirrups of ¼ and ⅜-in. diameter, the only difference being in the main steel. Table 2 gives the principal properties of the materials used in the beams, together with the results of the tests. In Fig. 6 are given average shear-strain curves for the two types of beams tested.

The analysis of tests was made in the same way as for Series 1 and 2. The lever arm of the resisting couple was determined for the cracked section with a value of $n = 8$ corresponding to the average experimental $k = 0.39$. With $n = 8$, $j = 0.886$. From an analysis similar to that explained for Series 1 and 2, the following general equations were fitted to the test results:

$$v_y = Krf_y + 0.04f'_c + 94 \text{ (in psi)} \quad \text{(3)}$$

and

$$v_u = Krf_y + 0.10f'_c + 94 \text{ (in psi)} \quad \text{(4)}$$

Table 2 shows the relation between the values calculated by the foregoing formulas and the experimental values. The agreement between calculated and experimental values is again very good with averages that for all practical purposes can be considered as 1.0.

5—DISCUSSION

The analysis of these tests indicates that the strength of welded stirrups can be given by a formula of this type:

$$v = Krf_y + qf'_c + sp \quad \text{(5)}$$

in which v is the shearing stress, Krf_y is the shearing stress that the beam would be able to resist if the truss analogy that is usually made in analyzing the action of the web reinforcement in reinforced concrete beams held absolutely true in an actual beam. The quantity qf'_c, in which q is a coefficient and f'_c the concrete strength, gives the effect of the concrete in resisting diagonal tension. The coefficient q for the beams tested was 0.04 when the shear V_y was considered, and 0.10 when the ultimate shear was analyzed. The quantity sp, in which s is a coefficient and p the ratio of tension steel, gives the effect of the main reinforcement on the resistance to diagonal tension. The factor sp was 200 in formulas (1) and (2), and 94 in formulas (3) and (4). With these values, the coefficient s is 5000.

Although the quantity sp has been taken as proportional to the ratio of main reinforcement, there is no definite proof from the tests for this supposition, which has been based mainly on the results of Series 1a. Should this assumption be true, it seems likely that the factor sp is not only a function of the ratio of reinforcement but also of the size of the bars used. No definite answers can be given to these questions, as only two ratios of main reinforcement and one size of bars were used in these tests.

The action of a beam with welded stirrups in resisting diagonal tension when it is subjected to a shear in the neighborhood of V_y may be likened to that of a truss in which the top chord is formed by the compression zone of the concrete, the bottom chord by the longitudinal reinforcement, the tension web members by the stirrups, and the compression web members by portions of the concrete web of the beam. The members of this truss are connected rigidly, but the rigidity of the tension web members is so small that they act practically as if they were pin-connected. The top chord, the bottom chord, and the compression web members have enough rigidity so that the truss can be thought of as a truss whose behavior lies somewhere between that of a Vierendeel truss with special diagonal members and that of a pin-connected truss. The stresses that would exist in the tension web members if the truss were pin-connected are relieved by the rigidity of the other members. The factor qf'_c in formula (5) would account for the rigidity of the top chord and that of the compression web members, and the factor sp for the rigidity of the main reinforcement. The scattering of the points in Fig. 4 from the straight line would depend on variations of the rigidity of the top chord and compression web members due to differences in the extension and location of the diagonal cracks and in the distance between them.

After the shearing stress V_y is reached, the deformation of the tension web members increases so rapidly that the truss can take more load only at the expense of the compression chord whose flexural resistance is rapidly decreased by the extension of the main diagonal crack until the remaining part fails by shear along this crack. This would explain why the values of $v_u - v_y$ are a function of the concrete strength only.

The analogy usually made, in which the action of the beam is likened to that of a pin-connected truss, appears to be substantially correct in determining the value of the coefficient K in formula (5). The agreement between calculated and experimental values in Table 2 indicates that for stirrups inclined at 67.5 degrees, the factor Krf_y is about 20 per cent greater than for stirrups at 45 or 90 deg. This factor Krf_y can be thought of as representing the effect of primary stresses in the truss with rigid connections. The other two factors in formula (5) would represent the effect of secondary stresses in that truss due to the rigidity of the connections. The effect of these secondary stresses decreases somewhat the advantage given by the stirrups inclined at 67.5 deg. over those inclined at 45 or 90 deg. In Fig. 7, the theoretical ratio between the shearing stresses at V_y for stirrups at 67.5 deg. and those for stirrups at 45 and 90 deg. have been represented. The calculations have been made for a beam with one per cent of longitudinal steel and assuming the coefficient s equal to 5000. The value of f_y has been taken as 40,000 psi. With these values, v_y is given by:

For stirrups at 67.5 deg. $v_y = 1.20r \times 40{,}000 + 0.04f'_c + 5000 \times 0.01$

For stirrups at 45 or 90 deg. $v_y = r \times 40{,}000 + 0.04f'_c + 5000 \times 0.01$

From Fig. 7, it can be seen that the efficiency of the stirrups at 67.5 deg. compared to those at 45 or 90 deg. depends on the ratio of web reinforcement as well as on the concrete strength. For a concrete strength of 3000 psi., and varying the ratio of web reinforcement from 0.2 to 1.00 per cent, the ratio represented ranges between 1.064 and 1.14. If these calculations are made for 2 per cent of tension steel, the range of the ratio for the same conditions is from 1.053 to 1.129.

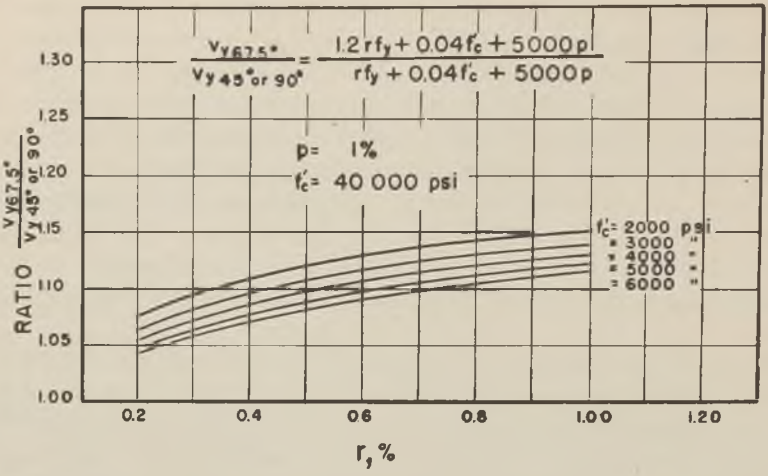

Fig. 7—Ratio between shearing stresses at V_y for stirrups at 67.5 deg. to those for stirrups at 45 deg. or 90 deg. as a function of the ratio of web reinforcement.

The current use of welded stirrups in reinforced concrete beams seems economically possible only if a scheme is worked out so that welded units similar to those shown in the photographs of this paper are prepared in advance in the shop, so that when they arrive at the construction where they are to be placed, they will require a minimum of workmanship and handling. In this case, advantage should be taken of the better efficiency given by stirrups placed at 67.5 deg. to the axis of the beams.

The safe shearing stress that the stirrups will be able to resist should be based on the value of v_y, which for these tests was equal to:

$$v_y = Krf_y + 0.04f'_c + 5000p \text{ (in psi)} \quad (6)$$

K being 1.00 for stirrups placed at 45 or 90 deg. and 1.20 for stirrups at 67.5 deg. The factor of safety by which v_y should be divided is a matter of judgment. It is believed that in selecting the factor of safety, some extra allowance should be made in the factor $5000p$ due to the unknown effect of the longitudinal bars.

The following formula for the safe shearing stress is proposed:

$$v_s = Krf_s + 0.02f'_c + 2000p \text{ (in psi)} \quad (7)$$

Fig. 8 gives the factor of safety, $\frac{v_y}{v_s}$, as a function of the ratio of web reinforcement for several concrete strengths. The computations have been made for a beam with one per cent of longitudinal reinforcement and stirrups at 67.5 deg. The value f_y has been assumed to be 40,000 psi and that of f_s to be 20,000 psi.

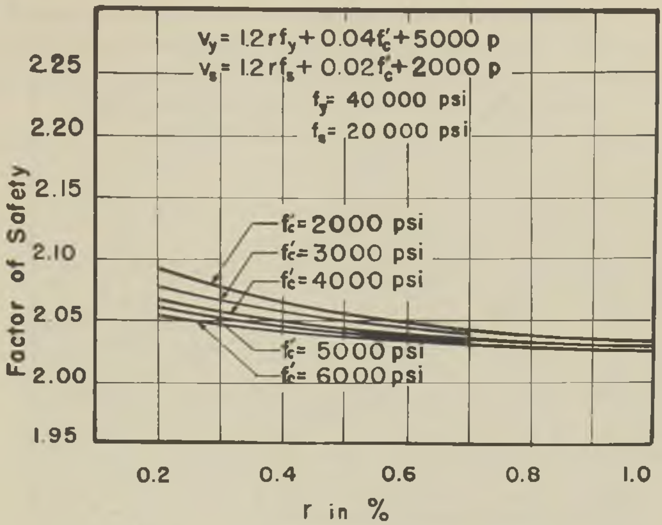

Fig. 8—Factor of safety, with reference to yielding of web reinforcement $\frac{v_y}{v_s}$, as a function of the ratio of web reinforcement for p = 0.01 and stirrups at 67.5 deg.

Due to the fact that the factor Krf is more important for stirrups at 67.5 deg. than for those at 45 and 90 deg., the factor of safety, computed as before, is a little greater for these latter types of stirrups.

Fig. 9 gives the relation between the shearing stresses at ultimate load determined by the formula

$$v_u = Krf_y + 0.10f'_c + 5000p \ldots\ldots\ldots\ldots 8)$$

and the safe shearing stresses computed from formula (7). These values, given for various concrete strengths, are plotted as ordinates against abscissas representing the ratio of web reinforcement. In computing these values, a beam with one per cent of longitudinal reinforcement and stirrups at 67.5 deg. was assumed. The value of f_y was taken as 40,000 psi. and that of f_s as 20,000 psi.

This factor of safety depends greatly on both the concrete strength and the ratio of web reinforcement. It is more than 3 for small ratios of web reinforcement and as low as 2.43 if one per cent of web reinforcement and a weak concrete (2000 psi.) is used, the ratio of longitudinal reinforcement being also one per cent.

6—COMPARISON WITH FORMER TESTS ON LOOSE STIRRUPS

A survey was made of former tests on web reinforcement in order to compare the behavior of welded stirrups with the conventional loose stirrups. It was found that practically all of the beams (of sizes comparable to those commonly used in buildings and other structures) that

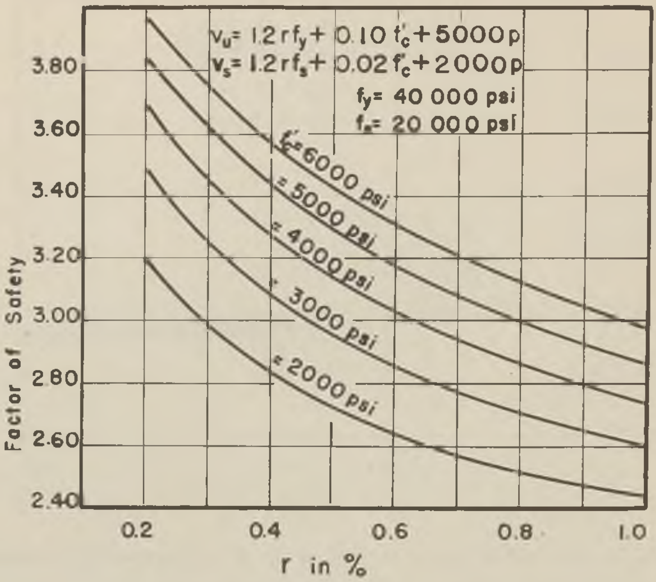

Fig. 9—Factor of safety, based on ultimate strength $\frac{v_u}{v_s}$, as a function of the ratio of web reinforcement for p = 0.01 and stirrups at 67.5 deg.

were previously tested had failed by causes other than diagonal tension. The only tests that could be used for this purpose are those reported by Slater, Lord, and Zipprodt.*

The specimens used in that investigation were short, deep, reinforced concrete I-beams loaded at the center and provided with stiffeners at the support and the center. Because of the heavily reinforced flanges and the presence of vertical stiffeners, the beams resembled Vierendeel girders with a thin web between the main members. Due to this, the action of those beams was probably different from that of the beams reported herein. For this reason, although a comparison has been attempted, it is believed that it does not settle the problem, but gives only a general idea of what might be expected.

Table 3 shows by reference number, as given in the report cited, the specimens selected for this comparison. No specific yield point stress is given for the steel; it is only stated that "the yield point stress of the shrapnel steel bars varied with the size of the bar, with some degree of regularity, from about 55,000 psi. for 1¼ in. to about 70,000 psi. for ⅜-in. bars." The values shown in Table 3 have been determined by assuming a straight line variation between the extreme values noted.

*W. A. Slater, A. R. Lord, and R. R. Zipprodt, "Shear Tests of Reinforced Concrete Beams," Tech. Paper No. 314 of the Bureau of Standards.

TABLE 3— COMPARISON OF THE EFFICIENCY OF WELDED AND LOOSE STIRRUPS

Based on test values for loose stirrups (Technologic Paper 314, U. S. Bureau of Standards) and values calculated from Equation 8.

Ref. No.	$f'c$ psi.	p, %	r, %	f_y psi.	v_u psi.	v_u corrt'd, psi.	v_u calc.	Ratio $\frac{v_u \text{ calc.}}{v_u}$	Ratio $\frac{v_u \text{ calc.}}{v_u \text{ corrt'd}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
13	5970	2.3	1.30	63,500	1030	950	1540	1.49	1.62
18	3510	2.3	1.30	63,500	970	900	1290	1.33	1.43
23	5580	1.73	2.60	65,500	1700	1470	2345	1.38	1.59
28	3690	2.33	0.92	65,500	980	910	1085	1.11	1.19
34	5550	2.02	1.55	68,000	1550	1360	1710	1.10	1.25
35	5160	1.41	1.66	68,000	1430	1220	1715	1.20	1.40
38	5750	2.04	1.20	68,000	1310	1150	1495	1.14	1.30
40	5720	2.26	0.83	68,000	1120	1010	1250	1.11	1.24
41	5700	2.34	0.59	68,000	880	805	1090	1.24	1.35
44	4720	2.04	1.77	68,000	1580	1390	1775	1.12	1.28
46	4150	2.29	0.86	68,000	1060	930	1115	1.05	1.20
52	5920	1.98	0.81	70,000	1290	1090	1260	0.98	1.15
53	5170	2.03	0.95	70,000	1410	1200	1285	0.91	1.07
56	4170	2.28	1.16	70,000	880	600	1345	1.53	2.24*
69	5500	2.01	2.38	65,500	1860	1640	2210	1.19	1.35
70	5310	2.04	2.52	65,500	1840	1650	2285	1.24	1.38
71	3650	2.04	2.69	65,500	1960	1820	2225	1.14	1.22
72	5400	2.02	1.58	68,000	1790	1600	1815	1.01	1.13
73	5190	1.99	1.49	68,000	1730	1530	1630	0.94	1.07
74	4380	2.04	1.16	68,000	1220	1075	1330	1.09	1.24
76	5950	2.24	0.83	68,000	1290	1210	1270	0.99	1.05
77	4990	2.34	0.58	68,000	970	910	1010	1.04	1.11
78	5160	2.02	1.67	68,000	1720	1490	1755	1.02	1.18
81	5230	2.02	0.87	70,000	1400	1210	1235	0.88	1.02
82	4380	2.01	0.84	70,000	1300	1120	1125	0.87	1.00
83	3240	2.04	0.94	70,000	1310	1120	1085	0.83	0.97
Average								1.11	1.23

*Not considered in the average.

This study has been limited to a comparison of the probable shearing stresses that the beams would resist at ultimate load were they reinforced with welded stirrups as calculated by formula (8); and the actual ultimate shearing stress given by the tests. Of the 114 beams reported in Tables 7 and 8 of the Technological Paper No. 314, 26 were selected for this purpose. The selection was made by considering only those with web reinforcement similar to that used in this investigation reported to have failed in diagonal tension and in which the stress in the web reinforcement at failure had reached the yield point. Column 6 shows the shearing stresses at ultimate load, based on gross section, as given in Tables 7 and 8. These stresses, reported to have been corrected to allow for the stiffness of the heavy flanges and stiffeners of the test beams, appear, from the maximum loads listed, to be uncorrected. The writer has therefore applied a correction to the shearing stresses listed in Technological

Paper No. 314, determined as indicated on page 414 and based on the maximum deflections listed on page 492 of that paper. Values of the maximum shearing stresses, uncorrected and corrected, are given in Columns 6 and 7 of Table 3.

For comparison, the writer has computed values of v_u by use of Eq. (8). For this purpose, p has been taken as the ratio of the tension steel section to the rectangular section enveloping the I-beam cross section. The ratio of this calculated value to the uncorrected and corrected values of Columns 6 and 7 is shown in Columns 9 and 10 of Table 3. These ratios show considerable spread in the individual values, as might be expected. On the average, it can be said that the strength of welded stirrups may be expected to be between 10 and 25 per cent greater than that of loose stirrups.

Due to the heavy reinforcement provided in the compression flange of the reinforced concrete I-beams it is believed that the values given in Column 10 represent more nearly what could be found on similar specimens with welded and loose stirrups. From the discussion in Section 5 of this paper, it is to be expected that the strength provided by the reinforced concrete I-beams after the load corresponding to V_y had been reached, was comparatively much greater than that given by the specimens reported in this investigation. It seems reasonable to expect the welded stirrups to be, on the average, about 20 per cent stronger than similar loose stirrups. The use of welded stirrups with 67.5 degrees inclination will make these stirrups 25 to 35 per cent stronger than the loose stirrups placed at 45 or 90 deg. Furthermore, while inclined loose stirrups were used in the laboratory tests of Technological Paper No.314, it is not likely that such stirrups would be practical or satisfactory in building construction.

7—SUMMARY AND CONCLUSIONS

This investigation embodies the tests of 44 beams of reinforced concrete of a size comparable to that commonly used in buildings and other structures, and designed in such a way as to produce failure by diagonal tension. The tests were made on simple beams subjected to two-point loading, and the web reinforcement was placed in the region of constant shear. A fairly large amount of tension steel was used to avoid failure by longitudinal tension or bond. The web reinforcement was provided by straight stirrups welded to the main reinforcement to form a unit. Variables including the size and inclination of the stirrups and the type of concrete used were studied. In four of the beams the ratio of main reinforcement was varied, in which case the quantity of tension steel was more nearly comparable to that commonly used in reinforced concrete beams.

The following are general remarks based on the results and discussion of these tests:

(a) The beams without web reinforcement failed by diagonal tension at an average shearing stress of about 490 psi.

(b) For beams with web reinforcement, an analysis was made of the test results, with particular attention to two stages of loading: (1) for the load at which the yield point stress of the web reinforcement was reached, as defined in Section 3, and (2) for the ultimate load. For both cases a formula was developed for the shearing stresses. For the load at which the web reinforcement was stressed to the yield point, the shearing stress is given by:

$$v_y = Krf_y + 0.04f'_c + 5000p \text{ (in psi.)} \quad \ldots\ldots\ldots\ldots (6)$$

At ultimate load, the shearing stress is given by:

$$v_u = Krf_y + 0.10f'_c + 5000p \text{ (in psi.)} \quad \ldots\ldots\ldots\ldots (8)$$

(c) The safe shearing stress that the stirrups will be able to resist should be based on formula (6). The factor of safety by which v_y should be divided is a matter of judgment.

The following formula for the safe shearing stress is proposed:

$$v_s = Krf_s + 0.02f'_c + 2000p \text{ (in psi.)} \quad \ldots\ldots\ldots\ldots (7)$$

This formula gives a factor of safety a little greater than two with respect to v_y and a mimumum of 2.43 with respect to v_u when a 2000 psi. concrete is used in a beam that has one per cent of both web and main reinforcement, with $f_y = 40{,}000$ psi. and $f_s = 20{,}000$ psi.

(d) A study was made of the ratio of the strengths of beams made with welded and loose stirrups. For that purpose, 26 beams were selected from the series of tests reported by Slater, Lord, and Zipprodt in Technological Paper No. 314 of the Bureau of Standards.

Although relatively short beams were used in both series of tests studied, the comparison is rendered uncertain by the fact that the two series of tests were made 25 years apart, at different laboratories, and with different materials, techniques, and types of test beams. From the comparison, it seems reasonable to expect a beam made with welded stirrups to have on the average, about 20 per cent greater resistance to diagonal tension than one containing similar loose stirrups. Furthermore the use of welded stirrups with a 67.5 deg. inclination should make a beam 25 to 35 per cent more resistant than one made with loose stirrups placed at 45 or 90 deg.

(e) A comparison was made between the measured deflections at midspan and the theoretical deflections due to flexure, as calculated by Maney's equation. The measured deflections at ultimate load were sometimes as much as 40 per cent greater than the calculated ones. At the load corresponding by V_y, they were, on the average, 10 to 20 per cent greater than the calculated values. This indicates the presence of shearing deflections and deflections due to opening of diagonal cracks of appreciable importance in these test beams.

ACKNOWLEDGMENT

These tests were made in the University of Illinois Engineering Experiment Station under the supervision of Prof. Frank E. Richart. The writer is very much indebted to Professor Richart for many valuable suggestions and criticisms throughout this investigation.

ACI NEWS LETTER

Vol. 17 No. 2 JOURNAL of the AMERICAN CONCRETE INSTITUTE November 1945

42nd ANNUAL ACI CONVENTION

New York City—Feb. 18-21, 1946

The times have not yet come when hotel accommodations are "easy". Make your plans early to attend the ACI meetings in New York City and MAKE YOUR RESERVATIONS EARLY. The management of Hotel New Yorker has promised to take care of all ACI people—but that promise is based on your responsibility to make early and specific reservations. In making these reservations be sure to identify yourself with ACI.

Timely and varied subject matter will engage six general sessions of the Institute's 42nd Annual Convention at Hotel New Yorker, New York City, Tuesday, Wednesday and Thursday, February 19 to 21, 1946.

This will be ACI's first postwar get-together and the first in the nation's metropolis since 1939—not counting the pocket-size, purely administrative, meetings last February.

Some outstanding objectives in subject matter were discussed by the Board of Direction in September and referred to the Publications Committee, Robert F. Blanks, Chairman. The Committee is at work to produce a good program. To all ACI Members it might be said: If you are asked to help, try to do it. If you have an idea—a suggestion of a worth-while source for a JOURNAL paper likely to be of interest to many readers, pass it on to Mr. Blanks (Bureau of Reclamation, Denver) with a copy to the ACI Secretary. It is well to remember that not all good papers are good convention papers. The rostrum presents problems different from the printed page.

"Leaders" have been named, each of whom is to develop a subject coverage—not in one paper but in several brief, brisk convention presentations which will leave laborious detail for study in the printed page, from which such detail can be more readily absorbed. Papers offered for each session are first to be prepared in full in a version intended for publication—then be briefed by their authors for convention purposes.

Board of Direction and Advisory Committee meetings are scheduled to be held Monday, February 18, with meetings that day also of such technical committees as do not involve personnel conflicts with Board or Advisory Committee. Other technical committees are planning to meet Tuesday morning, February 19.

General convention registration will begin at 9 a.m. on the 19th. The following sessions are scheduled:

Tuesday, February 19	—2 p.m.
	8 p.m.
Wednesday, February 20	—9.30 a.m.
	2.00 p.m.
	8 p.m.
Thursday, February 21	—9.30 a.m.

Subjects scheduled tentatively for special attention are:

- Reinforced concrete design theory and practice.
- Concrete construction practice.
- Research (under the customary auspices of ACI Committee 115, Morton O. Withey, Chairman, and S. J. Chamberlin, Secretary).
- Maintenance and repair of concrete structures.
- Use of air-entraining concrete.
- In addition to these, a sprinkling of diversified papers.

WHO'S WHO in this JOURNAL

Glenn S. Paxson

ACI member since 1930, is Bridge Engineer for the Oregon State Highway Commission. He received his B.S. degree from Oregon State College in 1912 and a C.E. degree in 1937 from the same school. Following his graduation he was for two years assayer and surveyor for the Ben Harrison Mines Co. in Eastern Oregon, and then for three years was superintendent of quarry operations producing surfacing material for highway construction. He served as 1st Lieut. during the First World War with eighteen months' service in France. Upon returning to civil life, he entered the employ of the Oregon State Highway Commission and has remained continuously with this organization successively as transitman, resident bridge engineer, assistant bridge engineer in charge of construction, and bridge engineer, serving in the latter position since 1935. See his contribution on maintenance and repair of concrete bridges p. 105.

T. C. Powers

ACI member since 1927—twice a Wason Research Medalist, is too well known as a contributor to this JOURNAL and as Manager of Basic Research, Portland Cement Association, Chicago, to need an introduction in this column on the occasion of his paper "Should Portland Cement Be Dispersed?" p. 117.

Oreste Moretto

ACI member only since 1944, is a graduate of the University of Litoral, Rosario, Argentina, where he received the degree of Civil Engineer in 1940. From 1940 to 1943 he was engaged on bridge design for the Public Roads Administration of Argentina. Since September, 1943, he has pursued graduate studies in Civil Engineering at the University of Illinois, working under fellowships from the University of Litoral and the Institute of International Education in New York City.

Mr. Moretto received the degree of M.S. in Civil Engineering in 1944 and is now completing his work for the Ph. D. degree. He expects to return to his native country, Argentina, in 1946. His contribution to this JOURNAL p. 141 grew out of work with Prof. F. E. Richart at Talbot Laboratory, University of Illinois.

Phaon H. Bates Retires

Phaon H. Bates, chief of the Division of Clay and Silicate Products, National Bureau of Standards, and a member of the Bureau's staff since 1910, exercised his optional retirement privilege Sept. 15, 1945. Mr. Bates was born August 1, 1879, in Sipisville, Somerset County, Pa. and received his formal education in the public schools of Philadelphia and at the University of Pennsylvania, choosing chemistry as his profession. He served in the chemical laboratory of the Pennsylvania Railroad where he analyzed miscellaneous supplies and later joined the staff of their locomotive testing plant where he worked on coals, ashes, and combustion products. From 1906 to 1910 he served as a chemist in the Pittsburgh laboratory of the U. S. Geological Survey.

When on July 1, 1910, the functions of that laboratory were divided between the newly established Bureau of Mines and the National Bureau of Standards, he was transferred to the latter. Continued as the Pittsburgh branch of the Bureau, a laboratory was maintained on the grounds of the old U. S. Arsenal in that city for ceramic research and the testing of structural materials; he became principal chemist of this laboratory, and was placed in entire charge of the work. Because of unsettled conditions prior to World War I, it appeared that the United States would be cut off from its supply of optical glass, and facilities for its production were not available in this country. Mr. Bates organized an experimental program that ultimately led to the manufacture of high grade optical glass in the Pittsburgh laboratory.

On the completion of the Bureau's Industrial Building in 1919, the major portion of the work of the Pittsburgh branch was transferred to Washington where it was consolidated with other projects to form a new division concerned with engineering and structural materials. This eventually became the present Division of Clay and Silicate Products, covering the fields of whiteware, optical glass, refractories, ceramic coatings, cement and concrete, lime and gypsum, stone, and masonry construction. This division played a leading part in the extensive test program on building materials and structures conducted during the years just preceding the war, the results of which have been published in a series of more than one hundred reports.

During World War II, the optical glass laboratory assumed the proportions of a fair-sized industrial plant with all the usual difficulties of recruiting personnel and meeting exacting production schedules. That these difficulties were overcome is due in no small measure to the work of Mr. Bates. Another important contribution of the Division was the development of a new ceramic coating for the protection of metal parts against corrosion at high temperatures. This was used on the exhaust system of aircraft and amphibious vehicles.

Mr. Bates has always taken a leading part in the development of testing methods and likewise in the preparation of Federal and other nationally-used specifications. In recognition of his outstanding reputation in this field, he was elected president of the American Society for Testing Materials at the 47th annual meeting in June 1944, serving until June 30, 1945. He delivered the 1940 Edgar Marburg Lecture to this Society, a lecture given annually by an outstanding technical authority as a means of emphasizing the promotion of knowledge of engineering materials.

He has been closely identified for many years with the work of the American Ceramic Society of which he is a fellow, and of the American Concrete Institute. President of the Institute from 1934 to 1936, he was awarded its Turner Gold Medal in 1939 "for contributions to science, direction of research and outstanding leadership in advancing the intelligent utilization of cement and concrete." A member of ACI continuously from 1926 and elected a director in the same year, he had been an active member in the Institute's earlier years. He had been member and chairman of its Publications Committee, member of Advisory Committee, member and chairman of its former Program Committee. He was also a member of the ACI Standards Committee, from which he resigned at the time of his retirement. He had always been an active advocate of increased work on the development of standards. He is a fellow of the American Association for the Advancement of Science and a member of the American Chemical Society, the Washington Academy of Sciences, and the Cosmos Club.

Author or co-author of several Bureau papers dealing with portland cement and related subjects, he has also contributed numerous articles to the technical journals in his special fields of activity.

Douglas E. Parsons has been named chief of the Division of Clay and Silicate Products to succeed Mr. Bates.

Douglas E. Parsons

President of American Concrete Institute, who has been a member of the staff of the National Bureau of Standards for more than twenty years, became, in September, Chief of the Bureau's Division of Clay and Silicate Products, to succeed Phaon H. Bates, who retired from Government service on Sept. 15, 1945. Mr. Parsons, a civil engineer, has been associated with the Bureau's section on masonry construction.

A native of Iowa, born in Marion, Nov. 3, 1894, Mr. Parsons received his formal education in his home state, graduating in 1917 with the A.B. degree and receiving the C.E. degree in 1923, both from Cornell College, Mount Vernon, Ia. In the United States Army from 1917-19, he served as a Lieutenant with the 89th Division, American Expeditionary Forces. Upon his return to civilian life, he became Assistant Chief Engineer of the Iowa Railway and Light Co., Cedar Rapids.

In 1923 he joined the staff of the National Bureau of Standards, was placed in charge of the Masonry Construction Section in 1929, and early in 1944 was designated assistant chief of Division IX which he now heads; his entire service with the Bureau has been in that division.

Following many years of active participation in the affairs of the American Concrete Institute, Mr. Parsons was elected its President in February of this year, for a one-year term. In 1936, he was awarded the Institute's Wason Medal in recognition of noteworthy research; he shared this honor jointly with Ambrose H. Stang of the National Bureau of Standards and Vice-Admiral Ben. Moreell, U. S. Navy, Bureau of Yards and Docks, for work involved in their studies of Mesnager Hinges used in concrete construction. Through a coincidence, the medal was presented to him by P. H. Bates, then President of the Institute, whom he has now succeeded as division chief.

Mr. Parsons has been an Institute member since 1926, a member of its Board of Direction since 1941 and served as member of the Advisory Committee and of its Publications Committee, having been chairman of the latter 1941 to 1945.

He is a member of the American Society of Civil Engineers, the American Society for Testing Materials, the American Association for the Advancement of Science, the Society for Experimental Stress Analysis, and the Washington Academy of Sciences; and is Chairman of American Standards Association Committee A-41 of Masonry, and the Federal Specifications Technical Committee on Precast Reinforced Concrete Products. Mr. Parsons is author or co-author of a large number of papers in the Bureau's series of publications, including more than a dozen of the Building Materials and Structures Reports on structural properties of various types of masonry construction. He has also contributed numerous articles and papers to technical journals in his field.

New Members

The Board of Direction approved 47 applications for Membership (35 Individual, 7 Corporation, 2 Junior, 3 Student) received in August and September as follows:

Billig, K., 167 Victoria St., London, S. W. 1, England

Bohn, Richard A., Nat'l Bureau of Standards, 209 Old Mint Bldg., San Francisco 3, Calif.

British Reinforced Concrete Engineering Co. Ltd., Stafford, England (C. H. Griffin)

Benavant y Campama, Jaime P., Edificia Banco, Nova Scotia 224, Havana, Cuba

Carroll, P. J., "Wave Crest", Ennis Crone, 60 Sligo, Eire

Chambers, J. W., Alabama State Highway Dept., Montgomery, Ala.

Cohen, George N., c/o Euclid Construction Co., 101 Park Ave., New York 17, N. Y.

Diaz, Agapito Leon, Princesa 119 Jesus del Monte, Havana, Cuba

Forsman, Otto, Statens Provningsanstalt, Stockholm 26, Sweden

Funk, Charles R., 6a Avenida Sur, Numero 54, Guatemala City, Guatemala, C. A.

Guillo, Carlos Maruri y, 14 entre 3a y 5a, Miramar, Havana, Cuba

Habashi, Fauzi, Faculty of Engineering, Fouad 1 st University, Giza, Egypt

Hatch, George E., c/o U. S. Engineer Office, Honolulu, T. H.

Hawaiian Cement Co. Ltd., P. O. Box 2454, Honolulu, Hawaii (L. N. Bryant)

Hebold, Denis, 217 W. Sparks St., Philadelphia 20, Pa.

Hong, Li Ching, c/o The Librarian, Bureau of Reclamation, Denver 2, Colo.

Huckleberry, Major B. C., New Belvedere Hotel, Columbus, Indiana

Highway State Comm. of Kansas, Masonic Temple Bldg., Topeka, Kansas (R. D. Finney)

Kendall, Edgar R., 2360 Glenwood Ave., Toledo, Ohio

Krueger, Arthur, Box 77, R. R. 3, Cincinnati 11, Ohio

Long, J. S., Devoe & Raynolds Co., Inc., P. O. Box 328, Louisville, Ky.

Loomis, Robert S., A/S, Navy V-12 Unit, M.I.T., Room No. 518-A, Cambridge 30, Mass.

Matthews, Homer M., Matthews & Kenan, 1616 Transit Tower, San Antonio, Texas

Mautner, Dr. K. W., 65 Ladbroke Grove W., 11 Flat 3, London, England

Painter, William D., 101 Lincoln Street, Alcoa, Tenn.

Pallot, D. P., 123 Derby Rd., Subiaco, West Australia

Pang, Dat Quon, 938 N. Vineyard St., Honolulu 7, T. H.

Panuzio, Frank L., 1385 Capitol Ave., Bridgeport, Conn.

Parva Domus, S. A., Apartado 1095, Lima, Peru, S. A.

Pennsylvania-Dixie Cement Corp., Old Colony Bldg., Des Moines 9, Iowa

Pittard, A. R., 39 Victoria St., London, S. W. 1, England

Polychrone, James A., Nichols 102, M.I.T. Dorms, Cambridge, Mass.

Roberts, C. G., Country Roads Board, Carlton, N3, Victoria, Australia

Roche, Walter W. R., 6 Haarlem Road, London W. 14, England

Rosberg, E. O., 1496 Marin Ave., Albany, Calif.

Schuetz, Clyde C., United States Gypsum Co., 1253 Diversey Parkway, Chicago 14, Ill.

Sementtiyhdistys, Kalevankatu 20, Helsinki, Finland

Shevling, Charles E., Ross Dam, Rockport, Wash.

Shoemaker, Robert R., P. O. Box 318, Long Beach, Calif.

Spindel, M., 89 Dorchester Court, Flerne Hill, London, S. E. 24, England

U. S. Engineer Office, 409 Davidson Bldg., 10 East 17th St., Kansas City 8, Mo.

Upper, R. E., 53 Fulton Ave., Toronto, Ont., Canada

Wagenett, Frank, 28 Mountain St., Rockville, Conn.

Wai, Francis K., 3425 Pahoa Ave., Honolulu, Hawaii

Waidelich, A. T., The Austin Co., 16112 Euclid Ave., Cleveland 12, Ohio

Wessels, Vincent E., Petoskey Portland Cement Co., Petoskey, Mich.

Wright, K. W., Ash Grove Lime & Portland Cement Co., P. O. Box 519, Chanute, Kansas

Honor Roll

February 1 through October 31, 1945

Rene Pulido y Morales, in Havana, Cuba heads the list with 21 new Members proposed since Feb. 1.

Rene Pulido y Morales 21
Roy Zipprodt 6
Harry B. Dickens 5
H. F. Gonnerman 4
A. Amirikian 3
J. H. Spilkin 3
Charles S. Whitney 3
Ernst Gruenwald 2½
Charles E. Wuerpel 2½
C. Blaschitz 2
H. W. Cormack 2
Francis MacLeay 2
Calvin C. Oleson 2
D. E. Parsons 2
Dean Peabody, Jr. 2
C. H. Scholer 2
J. M. Wells 2
J. W. Kelly 1½
Ben E. Nutter 1½
O. A. Aisher 1
H. Victor Carman 1
W. Fisher Cassie 1
A. R. Collins 1
R. F. Dierking 1
H. F. Faulkner 1
P. J. Freeman 1
J. K. Gannett 1
Stanley S. Haendel 1
G. H. Hodgson 1
F. B. Hornibrook 1
V. P. Jensen 1
L. I. Johnstone 1
William G. McFarland 1
Denis Matthews 1
H. W. Mundt 1
Y. G. Patel 1
A. F. Penny 1
Kenneth Powers 1
Simeon Ross 1
John A. Ruhling 1
Byram Steel 1
G. W. Stokes 1
Wm. Summers Jr. 1
M. A. Timlin 1

J. W. Tinkler........................1
Maxwell Upson........................1
Stanton Walker........................1
J. C. Witt........................1

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

Birger Arneberg	Adolph Meyer
Michel Bakhoun	A. F. Moore
E. E. Bauer	O. F. Moore
E. W. Bauman	M. D. Olver
P. G. Bowie	C. E. O'Rourke
C. H. Chubb	Jerome Raphael
J. H. Chubb	F. E. Richart
Arthur P. Clark	Chas. S. Rippon
R. R. Coghlan	Kanwar Sain
R. B. Crepps	J. L. Savage
Harmer E. Davis	Oskar Schreier
N. M. Hadley	A. L. Strong
W. C. Hanna	A. C. Trice
W. S. Hanna	J. H. de W. Waller
Carl W. Hunt	A. R. Waters
W. R. Johnson	David Watstein
Paul A. Jones	George Winter
H. J. McGillivray	Harry C. Witter
R. E. McLaughlin	K. B. Woods

Frank W. Garran

dean of the Thayer School of Engineering at Dartmouth College, ACI member since 1942, died Sept. 19 in the college infirmary at the age of 51. He was stricken with nephritis four months ago. He was born in Boston, 1894.

Dean Garran, associated with Thayer School since 1929, had been dean since 1934. Recently he was chairman of the New England section of the Society for the Promotion of Engineering Education and at his death was a national director of the society representing New England. He was also a trustee of Norwich University.

A graduate of Norwich in 1917, he was headmaster of the Atkinson Academy in New Hampshire for a year before returning to the university as Assistant Professor of Civil Engineering 1920 to 1923. He took his master's degree at Massachusetts Institute of Technology in 1924 and in that year became assistant professor of civil engineering at the University of Arizona. Two years later he was named Professor of Engineering at the College of Charleston, S. C., leaving there in 1929 to come to Dartmouth as assistant professor. Dartmouth awarded the honorary master's degree to him in 1933, the year in which he was made full professor and acting dean of the Thayer School.

During the war he was coordinator of Dartmouth's civil pilot training program and also directed the Government-sponsored engineering, science, management war training course. In the first World War he was a first lieutenant with the Army engineers and served in France and Germany. He was a former president of the Dartmouth Scientific Association, a member also of the American Society of Civil Engineers and Theta Chi Fraternity.

George T. Seabury

who died May 25, 1945, on the eve of his retirement as Secretary of the American Society of Civil Engineers, after 20 years service with the society, had been an ACI member since 1926.

Mr. Seabury went to the A.S.C.E. in 1925 with a background of practical experience. Born in Newport, R. I., in 1880 and graduated from M. I. T. in 1902 as a civil engineer, he spent four years as a contractor's engineer around New York. The next 9 years were spent on Catskill work of the New York Board of Water Supply.

In 1915 Mr. Seabury went to Providence, R. I. to become a division engineer with the water supply board at that city. Three years later he left that position to accept a commission as a major in the Construction Division of the Army, participating in the construction of Camps Devens, Upton, Meade, Mills, Merritt, Dix and Lee. In 1918 he organized the contracting firm of George T. Seabury, Inc., and was engaged in construction work until 1923 when he was named manager of the Providence Safety Council. He left that position to become secretary of the A. S. C. E. on Jan. 1, 1925.

Guy O. Gardner

ACI member since 1930, who was production manager and research director of all the Ash Grove Lime & Portland Cement plants in Kansas, Missouri, and Nebraska, died August 5 at his home in Chanute, Kan., age 57. He had been ill with a heart ailment.

Mr. Gardner served as superintendent of the Chanute, Kan. plant from 1925 until 1945 when he received the promotion to production manager and research director. Coming to the Chanute plant June 25, 1912, as a chemist, he held that position until 1917, when he was promoted to assistant superintendent,

He was an active member of the Rotary club, and served as president and also as secretary. He was a past president of the Chanute Country Club and was serving as a director at the time of his death. He was also a director of the Chamber of Commerce, and a member of the Masonic lodge, the Elks, and Presbyterian church.

PCA regional office changes

have been effective since June 15, as announced by William M. Kinney, General Manager of the Portland Cement Association.

M. J. McMillan, Manager of the Washington Office since 1936, is now Regional Manager of the Eastern Offices, 347 Madison Ave., New York 17.

James E. Dunn, District Engineer of the Richmond, Va. office since 1938, is now Manager of the Washington Office, 837 National Press Building, Washington 4, D.C.

Gordon S. Maynard, field engineer in North Carolina and Virginia for the association since 1937, is now District Engineer with headquarters at 1210 State Planters Bank Building, Richmond 19, Va.

Working with Mr. McMillan in supervising association activities in the eastern region is E. M. Fleming, District Manager, New York, in charge of field activities in Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York, Rhode Island and Vermont, and G. C. Britton, District Manager, Philadelphia, covering Delaware, Maryland and Pennsylvania.

Messrs. McMillan, Dunn, Fleming and Britton are all ACI members.

Lt. Col. E. W. Scripture

recently returned from Europe after serving three years with the Army Engineers, resumed his post October 8 as Director of Research of The Master Builders Co., Cleveland, O.

Colonel Scripture entered the service in 1942 as a captain, and was assigned to the Office of the Chief Engineer, European Theater of Operations. The first two years of his service overseas were in England where he took part in planning the invasion. In August 1944 Colonel Scripture went to France where he continued planning for engineering operations on the continent. He was awarded the Purple Heart, Bronze Star Medal, Legion of Merit and presented with the Medaille, Louis de Broglie, by the Association des Engeineurs Docteurs de France.

Colonel Scripture, also a veteran of World War I, having served in France for two years with the 26th Division, is widely known in the construction industry for his research work in the improvement of concrete and mortars.

C. D. Williams

member of the Institute since 1937, who has been head of the Department of Structural Engineering at Fenn College, Cleveland, since 1937, except for leave of absence in the war years,accepted appointment as head of the Civil Engineering Department at the University of Florida, effective September 1, Gainesville, Fla. While recently on leave from Fenn College, Professor Williams was supervisor of stress engineers at the Fisher-Cleveland Aircraft Division in charge of stress work on the B-29 and still later as assistant supervising engineer for the J. E. Greiner Co., Baltimore, Md.

The Laclede Steel Company, St. Louis, Mo., contributing member of ACI, announces the election of William M. Akin as president and treasurer of the company on September 14, 1945.

ACI publications in large current demand

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.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity).

For further information about ACI Membership and Publications (including pamphlets presenting Synopsis of recent ACI papers and reports) address:

AMERICAN CONCRETE INSTITUTE 742 New Center Building Detroit 2, Michigan

Sources of Equipment, Materials, and Services

A reference list of advertisers who participated in the Fourth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1945 issue and (when it is completed) in V. 41, ACI Proceedings. **Watch for the 5th Annual Technical Progress Section in the February 1946 JOURNAL.**

Contractors, Engineers and Special Services

Materials and Accessories

Testing Equipment

Statement of Ownership, Management, Circulation, Etc., Required by the acts of Congress of August 24, 1912, and March 3, 1933 of Journal of the American Concrete Institute published 6 issues a year at Detroit, Michigan, for September 1945.

STATE OF MICHIGAN...} ss
COUNTY OF WAYNE....}

Before me, a Notary Public in and for the State and county aforesaid, personally appeared Harvey Whipple, who, having been duly sworn according to law, deposes and says that he is the Editor of the Journal of the American Concrete Institute and that the following is, to the best of his knowledge and belief, a true statement of the ownership, management (and if a daily paper, the circulation), etc., of the aforesaid publication for the date shown in the above caption, required by the Act of August 24, 1912, as amended by the Act of March 3, 1933, embodied in Section 537, Postal Laws and Regulations, printed on the reverse of this form, to wit:

1. That the names and addresses of the publisher, editor, managing editor, and business managers are:

Publisher, American Concrete Institute, 742 New Center Bldg., Detroit 2, Mich.

Editor, Harvey Whipple, 742 New Center Bldg., Detroit 2, Mich.

Managing Editor, None.

Business Managers, None.

2. That the owner is: (If owned by a corporation, its name and address must be stated and also immediately thereunder the names and addresses of stockholders owning or holding one per cent or more of total amount of stock. If not owned by a corporation, the names and addresses of the individual owners must be given. If owned by a firm, company, or other unincorporated concern, its name and address as well as those of each individual member, must be given.)

American Concrete Institute, 742 New Center Bldg., Detroit 2, Mich.

Douglas E. Parsons, President, National Bureau of Standards, Washington 25, D. C.

H. F. Gonnerman, Vice-President, 33 W. Grand Ave., Chicago 10, Ill.

Stanton Walker, Vice-President, 951 Munsey Bldg., Washington 4, D. C.

3. That the known bondholders, mortgagees, and other security holders owning or holding 1 per cent or more of total amount of bonds, mortgages, or other securities are: (If there are none, so state.)

NONE.

4. That the two paragraphs next above, giving the names of the owners, stockholders, and security holders, if any, contain not only the list of stockholders and security holders as they appear upon the books of the company but also, in cases where the stockholder or security holder appears upon the books of the company as trustee or in any other fiduciary relation, the name of the person or corporation for whom such trustee is acting, is given; also that the said two paragraphs contain statements embracing affiant's full knowledge and belief as to the circumstances and conditions under which stockholders and security holders who do not appear upon the books of the company as trustees, hold stock and securities in a capacity other than that of a bona fide owner; and this affiant has no reason to believe that any other person, association, or corporation has any interest direct or indirect in the said stock, bonds, or other securities than as so stated by him.

5. That the average number of copies of each issue of this publication sold or distributed, through the mails or otherwise, to paid subscribers during the twelve months preceding the date shown above is ...... (This information is required from daily publications only.)

Sworn to and subscribed before me this 27th day of September, 1945.

HARVEY WHIPPLE,
(Signature of editor)

ETHEL B. WILSON, Notary Public
(My commission expires Aug. 10, 1946)

[SEAL]

THE AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete. The Institute is dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

For nearly four decades that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in annual volumes of ACI Proceedings since 1905. Beginning 1929 the Proceedings have first appeared periodically in the Journal of the American Concrete Institute and in many separate publications. (Pamphlets presenting brief synopses of Journal papers and reports of recent years, most of them available at nominal prices in separate prints, are available for the asking.)

For further information about ACI Membership and publications (including pamphlets presenting synopses of ACI Journal papers and reports of recent years), address:

Secretary, AMERICAN CONCRETE INSTITUTE

New Center Building, Detroit 2, Michigan, U. S. A.

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