

EFFECTS OF CONCRETE CONSTITUENTS, ENVIRONMENT, AND STRESS ON CREEP AND SHRINKAGE OF CONCRETE

Reported by ACI Committee 209

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This paper discusses the current state of knowledge in the area of time-dependent deformation of plain concrete. The paper is divided into three broad areas: effects of concrete constituents, effects of environment, and effects of stress.

Under the general heading of effects of concrete constituents the relation between creep and shrinkage and the structure of the gel, moisture movement, aggregate properties, internal microcracking and admixtures are discussed. Included in the second section, the effects of environment, are the subjects of temperature, humidity, maturity and time. The final section, effects of stress, considers type of stress, variation of stress, magnitude of stress and duration of loading.

Keywords: absorption; admixtures; age; aggregates; cement pastes; concretes; cracking (fracturing); creep properties; environments; gels; microcracking; modulus of elasticity; moisture content; plain concrete; reviews; shrinkage; stresses; temperature; water-cement ratio.

□ In a recent symposium concerned with the European “Unified Code for Structural Concrete” Dr. S. C. C. Bate suggested that “The limit state of excess

*Member of Subcommittee 1 which prepared this report.

ACI Committee 209 was organized to review critically available data on the factors affecting the magnitude of creep and volume changes of concrete; suggest and recommend topics of needed research; stimulate the preparation of papers; and interpret available information in a form that is useful to the designer.

The mission of Subcommittee 1 whose members wrote this paper, was to report on the current state of knowledge of time-dependent plain concrete.

deflection will form one of the three main criteria of serviceability of structures (the others being collapse and local damage).” In order to control excessive deflections the design engineer must first be able to reasonably predict the short time and long time deformational behavior of a given structure. This problem has been the subject of about 1500 research papers published since the turn of the century. In general, all of these efforts can be divided into two categories:

1. Attempts to understand the basic mechanisms and causes of creep and shrinkage of concrete.
2. Attempts to develop empirical design methods (based on the best available scientific information) that will enable the designer to control the deflection of structures, within specified limits of uncertainty.

In 1964 The American Concrete Institute released Special Publication No. 9, in which the available information on the subject of Concrete Creep and Shrinkage was summarized. Since that time the proliferation of publications in both of the aforementioned areas has continued. The paper that follows may be considered the second in what the authors hope will not be a long series of ACI Committee 209 papers concerning the state of the art of one of the major unsolved problems facing the civil engineer, the causes of creep and shrinkage of plain concrete.

As the title suggests, the paper will be divided into three broad areas. The effects of concrete constituents, including gel structure, water-cement aggregate interactions and admixtures, the effects of environment, broadly defined to include temperature, humidity, maturity and time; and the effects of stress, specifically applied stress.

THE EFFECTS OF CONCRETE CONSTITUENTS

EFFECT OF WATER-CEMENT-HYDRATE INTERACTION

General – The structure of cement paste can be described in terms of the following constituents: unhydrated cement; solid products of hydration including crystals, crystallites; and void space containing both strongly adsorbed and capillary water and air.^{1,2,3}

The porous hydrate known as the gel comprising the solid products of hydration and gel pores has a minimum porosity of about 26 percent, which corresponds to about four times the quantity of water which would form a layer 1 molecule thick on the solid phase. Although dimension changes occur mainly in gel, secondary volume changes which accompany continuing hydration of the cement cannot be ignored in some cases.

Solid phases in the gel – Crystals are defined as those particles, for example calcium hydroxide, which are large enough to be seen with an optical microscope, while crystallites may be defined as smaller particles which are beyond the resolution of the optical microscope. In recent years the structure of crystallites has been deduced from electron diffraction and x-ray diffraction studies but it has not yet been possible to obtain real images of the smallest discrete particles. Both transmission and surface scanning electron microscopes have produced images of assemblies of primary particles or crystallites.

Fig. 1-1 shows a typical surface scanning electron micrograph of calcium silicate hydrate “flowers” growing from a nucleation site on the surface of a portland cement grain.¹ Fig. 1-2 shows a transmission electron micrograph of petal shaped rolled foils of calcium silicate hydrate from ball-milled portland cement ultrasonically dispersed.¹ The second photograph was taken in a transmission electron microscope and the fibrous material comprising the “petals” was identified by electron diffraction as C_3S .

It has been suggested¹ that the strengths of the bonds between primary particles in the “petals” are orders of magnitudes greater than those between “petals” and that the strength of cement paste is determined by the weaker bonds. It is clear that displacements of these structures contribute significant components of strain.

Water – Free water is defined as water located at distances outside the range of surface forces and may, for practical purposes, be assigned the properties of bulk water. It is assumed that such water may be evaporated at 98 percent relative humidity.

Capillary or pore water – defined as water between 1 mm and 30 Å from gel surfaces. It is assumed that all such water may be evaporated at 40 to 50 percent relative humidity and may be assigned an average specific volume of 1.02 gm/cc.

Adsorbed water – water that is strongly attracted to the gel at distances between 0 and 30 Å from gel surfaces. It may be evaporated over the range 40 to 0 percent relative humidity. It is this type of water that has been described as “load bearing water” by Powers,³ as “active water” by Mills,¹ and as interlayer water by Feldman and Sereda.⁴ It appears that the active nature of such water would influence mainly the dilation and collapse of spaces with the “petal” structure. Its density can be assumed to be about 0.90 cc/gm.

Water of hydration or “non-evaporable” water – that part of the total water which cannot be evaporated at 0 percent relative humidity. It is approximately equal to the water held at 110°C but lost at 1000°C. It can be regarded as part of the solid phase in the gel.

Air – more correctly water vapor, is located in the remaining void space. Even in sealed specimens the volume of vapor filled space increases as hydration proceeds and the accompanying meniscus forces may cause shrinkage.

It will be observed that statements which assign reduced specific volume to adsorbed water imply that such water is in compression. This would seem to follow logically from the fact that adsorbed water molecules are subjected to inward directed forces of adhesion to the solid in keeping with reduction of surface energy in the solid. It should also be noted that spaces separating primary particles of the solid are unlikely to be parallel and therefore the tendency of water molecules to migrate to all vacant sorption sites on one side of a tapering void would be restricted by the repulsive impulses of water molecules subjected to similar forces on the opposite face. The net result would be an increase in the frequency of collisions between molecules as the space between adjacent solid surfaces is diminished. Hence the water may be said to be compressed and this compression would result in alternation of solid-solid binding forces. Thus strain energy would be stored in the solids and this would result in the sides being drawn together upon removal of adsorbed water.

The time dependent deformation of concrete is in part a result of the response of the above components to stimuli which alter the pressure in, and the hygral equilibrium of, the cement paste. Deformations of primary particles in the gel structure are insignificant by comparison with deformations of the gross structure and, since dry concrete does not creep at normal stress levels, it is clear that the primary particles themselves do not creep. It is clear that the bulk of time dependent deformation is related to changes in the interaction of water and cement hydrate.

Free water and part of the capillary water is relative mobile and responds rapidly to pressure changes. These changes do not significantly contribute to long term creep.

Water that is bonded to the hydrophillic solids comprising the gel (adsorbed and the remaining capillary water) responds slowly to mechanical and thermodynamic stimuli. Because of the hydrophillic nature of the gel, adsorbed water will tend to return to its original position upon removal of the mechanical or thermodynamic stimuli causing motion. Such movement may be inhibited by permanent changes in the solid.

Time-dependent deformation and moisture movement^{3,5,6,7} – Disturbance of the energy balance resulting from sustained mechanical load results in redistribution of water from areas of higher pressure to areas of lower pressure and/or relative humidity, or to the surrounding atmosphere.

In the early stages of loading the rapid responses of free water results in a minor volume change which is often regarded as stress induced shrinkage. This component of volume change may be recovered upon removal of the sustained load and re-saturation of the concrete.

The water more strongly attracted to the crystal and crystallite surfaces (capillary and adsorbed water) diffuses from its original position very slowly. The gel particles are brought closer together resulting in the major component of the

time-dependent deformation of concrete (creep). This deformation takes place rather slowly and probably never completely reduces the adsorbed water layers to zero. Providing that there is no change in internal gel structure and the condition of the water, the associated deformation should be recoverable. However, measurements have shown that a large percentage of this deformation is not recoverable under ordinary circumstances. The origin of this irrecoverable component has been examined in relation to evidence of a permanent change of structure.

Mills¹ has described as “collapse of structure” the mechanism whereby the surfaces of the solids originally containing load bearing adsorbed water are drawn close enough together to form inter-molecular bonds which can be numerous and strong enough to resist penetration of water upon re-saturation and/or removal of load. The suspected collapsed structure of a calcium silicate flower is illustrated in Fig. 1-3. Fig. 1-3B is typical of flower structures in shrinkage specimens, while Fig. 1-3A illustrates the amorphous nature of collapsed structure found in creep specimens. A comparison of the two photographs suggests that sustained load results in a permanent sintering of the crystallites in the flower structure, while shrinkage does not.

The concept of collapse of structure suggests methods of alleviating creep. It is well known that high pressure steam curing reduces the surface area, shrinkage, and creep of the cement gel. Conversion of colloidal to microcrystalline material, which occurs during high pressure steam curing, can be considered collapse of structure of one type. Ishai⁸ has shown that mechanically induced collapse of structure before loading materially reduces creep. It might also be possible to reduce the effect of collapse while the specimen is under load, by impregnating the material with a colloidal silicate or polymer which maintains the gel structure in a dilated state while evaporation of water takes place. The effective use of concrete made with low water cement ratios certainly merits investigation.

From the above it can be seen that the main variables associated with cement paste, that effect the creep of concrete, are those variables that depend ultimately on the quantity and equilibrium state of the active water in the system. The quantity of active water depends on the volume concentration of hydrated cement and this depends on the water-cement ratio and degree of hydration. Strength is a measure of both volume concentration of hydrate and volume of active water in a saturated paste. It is therefore not surprising that as an initial engineering approximation this component of creep can be evaluated in terms of the water-cement ratio, the age at time of loading, and the degree of hydration. Fig. 1-4^{9,10} illustrates the influence of water cement ratio. Table 1-1¹¹ shows the relationship between strength and specific creep (creep per psi) and the effect of age at time of loading is illustrated in Fig. 1-5⁵ where the creep of three specimen groups, moist cured an equal number of days but loaded at various times after curing, are plotted.

EFFECT OF AGGREGATE—CEMENT PASTE INTERACTION

Effect of aggregate — Since aggregates can be assumed to be volumetrically stable, it is to be expected that creep in concrete subjected to working loads

reduces with an increase in the volume proportions of aggregate. Typical results are shown in Fig. 1-6.¹² The data shown in the figure may be represented mathematically by $\log \frac{C_p}{C} = \alpha \log \frac{1}{1-g}$ where C_p is the creep of neat cement paste of the same quality used in the concrete, C is the creep of the concrete, g is the volumetric content of aggregate and α is a function of Poisson's ratio and the moduli of elasticity of the aggregate and paste.

Many investigators have shown that concretes made with different aggregates exhibit widely varying creep strains. Tests by Troxell et al¹³ and by Rusch¹⁴ suggest that variations associated with different petrological type of aggregate arise because of differences in elastic moduli of the aggregate. In addition, modulus of elasticity is related to the porosity or water sorption of the aggregate, as shown in Fig. 1-7a.¹⁴ Although aggregates with higher absorptions will tend to remove water from the cement paste, thereby decreasing the creep potential, the decrease is in most cases offset by the lower modulus of elasticity of the aggregate. See Fig. 1-7b.¹⁴

Other aggregate variables such as grading, maximum size and shape of the aggregate particles have been suggested as factors affecting the creep of concrete. These variables most likely have their main influence in the effect they have on the aggregate content and the interaction of the aggregate paste interface.

Microcracking — Cracking at the aggregate cement paste (or mortar) interface strongly affects the deformational behavior of concrete subjected to both short time and long time loads.¹⁵

Two types of microcracks exist in the concrete system:

1. Bond or interfacial cracks observed at the aggregate paste interface, and
2. Mortar or paste cracks within the mortar or paste matrix.

Interfacial cracks exist in concrete before it is subjected to external load and are caused by settlement of aggregate, bleeding of the mixing water, and shrinkage stresses induced by the drying process.¹⁶ They increase in number, width, and length as the magnitude of the external load is increased. The onset and continued propagation of microcracks in concrete subjected to short time load has been directly related to the nonlinear behavior of the stress strain curve.¹⁴

The initial cracking under load occurs at the aggregate mortar interface after its unconfined bond strength is exceeded.¹⁷ The cracked interface, if inclined to the axis of the load, continues to carry load through friction. As the load is increased, local cracks start to form through the mortar, at or near existing bond cracks, allowing the aggregate to slide relative to the mortar. As the load is further increased, the local stress in the uncracked portion of the system rises more rapidly than the average stress on the specimen, since the cracked interface can resist no additional load, until the compressive strength of the mortar is exceeded locally, causing the formation of mortar cracks. The continuation of this process leads to ultimate failure of the concrete system.¹⁸

It has been shown that for sustained loads below 70 percent of the ultimate strength of concrete, creep is accompanied by an increase in bond cracking.¹⁵

This is demonstrated in Fig. 1-8¹⁵ where crack length is plotted versus strain for short time and long time loaded specimens as well as for unloaded specimens. This increase in cracking results in an increase in time dependent strain, as illustrated in Fig. 1-9¹⁵ where the creep response of specimens in which cracks were induced (prior to the application of the sustained load) are plotted along with the creep response of non preloaded specimens.¹⁵ The higher creep in the non preloaded specimens can be attributed to deformation associated with cracking (aggregate particles sliding relative to the mortar).⁸

Thus, it seems that for a given aggregate and water-cement ratio the effect of aggregate content is two fold. Firstly, as the interface area is increased (overall aggregate content or fineness increased) the incidence of cracking and therefore creep is increased. Secondly, as the aggregate content is increased, the creep is decreased by the addition of inert material. The first effect does not seem to be significant at working stress levels (see Fig. 1-6), but can have a marked effect at stress levels in the mortar cracking range (above .70 f'_c). The dual effect of aggregate content at high stress levels is demonstrated in Fig. 1-10, where the creep response of various concretes with the same water-cement ratio are plotted against volume of hardened cement paste. Fig. 1-10 shows, that up to a critical aggregate content, creep is increased by increasing aggregate cement ratios (the effect of the interface). Beyond the critical aggregate content, creep is decreased by increasing aggregate cement ratios (the effect of replacing cement with aggregate that does not creep).¹⁹

EFFECT OF ADMIXTURES

The inclusion of admixtures in the section on concrete constituents does not imply that admixtures are considered, by the authors, to be an integral part of all concrete mixes. The subject is placed here for convenience of presentation only. Admixtures which vary the water requirement of the fresh concrete or the rates of setting and hardening influence shrinkage and creep insofar as they change the volume concentrations of hydrate adsorbed water and capillary water. Other admixtures such as fly ash, blast furnace slag and gypsum may also yield hydration products which have important differences of behavior and morphology comparable to those occurring with various C_3A contents. The effects of commonly used admixtures will be discussed in relation to shrinkage and creep.

Shrinkage — It has generally not proved possible to predict the influence of admixtures on shrinkage. Although any admixture which increases the water requirement of a mix can generally be expected to lead to increased shrinkage, the converse is not necessarily true. Notable exceptions are calcium chloride and triethanolamine, both of which are commonly used to offset the retarding action of some water reducing admixtures and both of which invariably lead to increased drying shrinkage.²⁰ Simple air-entraining agents do not appear to affect shrinkage.²¹

Although it is generally believed that the chemical composition of cement affects shrinkage, the tricalcium aluminate having the most effect,²² some investigators are of the opposite opinion.²³ Adverse shrinkage characteristics are believed to result from high C_3A contents²² and a deficiency in calcium sulphate.²⁴

The optimum amount of calcium sulphate for minimum shrinkage is believed to be slightly larger than the optimum calcium sulphate content necessary to avoid flash setting.²⁵ Wide variations of CaSO_4 and C_3A contents in commercial cement create special problems in relation to the general evaluation of the effect of additives on shrinkage. Although some attempt²⁶ has been made to overcome the problem by blending cements from different sources, it still becomes necessary to evaluate the performance of admixtures to specific concrete materials and the context of their use.²⁷

Special care is necessary in the interpretation of shrinkage tests on laboratory specimens when additives are evaluated. Since shrinkage depends on moisture loss it is necessary to observe shrinkage as nearly as possible to its terminal value and to record the associated weight change. It is therefore desirable to conduct shrinkage tests for as long as possible, and 56 days is considered the minimum acceptable testing period.⁸

The sequence of the mixing operation also appears to be important as far as the relative rates of reaction of CaSO_4 with C_3A and C_3A with the additive are concerned, since these reactions determine the availability of the additive to participate in slower silicate reactions.^{24,25,26} Delayed addition of the admixture results in a marked increase in its modifying influence. It appears that a delay of, say, two minutes before the addition of an admixture permits the calcium sulphate time to dissolve and coat the C_3A component of the cement. The coated C_3A is then unable to adsorb any appreciable amount of the admixture which leaves a large amount to modify the silicate hydration reactions. When the admixture is added with the mixing water it is already in the dissolved state and has time to react with, or be adsorbed on, the C_3A before the calcium sulphate has dissolved and is available for reaction with the C_3A . Thus relatively little additive remains available to modify the reaction of the silicates. This action of the C_3A is also the probable reason underlying the increased effectiveness of admixtures on cements with low C_3A vis-a-vis cements with high C_3A contents.

The use of blast furnace slag in portland cement concrete generally causes an increase in shrinkage.^{28,20,30,31} Low carbon fly ash, unlike most other pozzolans, reduces shrinkage.^{32,33}

Creep – Very little work has been reported concerning the influence of admixtures on the creep of concrete. Two papers^{34,35} concerning the influence of water reducing set retarding admixtures on the creep of normal weight and lightweight concrete indicate that a considerable increase in creep can be experienced under certain conditions. It has also been found that blast furnace slag substitution for part of the portland cement increases creep,²⁹ but that certain types of fly ash reduce creep.³⁶ The reason for this behavior is not known nor is it known if the increased creep is a function of the particular combinations of admixture, cement and aggregate used for these tests. Tests with the same admixtures and several cements³⁷ indicate that the chemical composition of the cement has a bearing on the increased creep of a concrete containing either a lignosulphonic acid or hydroxylated carboxylic acid admixture. Some concretes show reduced creep with an admixture but this behavior is perhaps the exception rather than the norm. Certainly there is enough evidence at the present time

to indicate that creep tests should precede the use of admixtures in structures in which an increase in the normal creep rates would be a cause for concern.

It seems likely that the presence of an additive modifies the gross morphological structure of cement paste. The hydration products of C_3A and also of C_4AF produced in the presence of lignosulphonic acid are markedly different from those produced in distilled water.^{38,39} A similar behavior has not been observed, however, with the silicate compounds but since direct observation of the silicate hydrates is difficult a less pronounced modification in a structure would pass undetected.

The ease with which cement paste loses water on heating appears to be related to the rate of creep of mixes with and without water reducing admixtures. Calcium chloride is found to increase creep substantially under a wide range of test conditions.⁴⁰ The simultaneous use of triethanolamine and a lignosulphonic acid increases creep at early ages of loading but not at later ages.

EFFECT OF ENVIRONMENT

General – The effects of ambient temperature and humidity on shrinkage and creep before, during and after loading are important insofar as these factors influence the volume proportions of hydrate, adsorbed and capillary water, and the rate at which moisture diffuses in the concrete and evaporates to the atmosphere.

A concrete unit in service may be subjected simultaneously to: (a) drying (shrinkage) and cooling (contraction); (b) drying (shrinkage) and heating (expansion); (c) wetting (swelling) and cooling (contraction); or (d) wetting (swelling) and heating (expansion).

Curing period before loading – Both temperature and humidity influence the rate of hydration. The extent of hydration in saturated concrete is approximately proportional to the integral of temperature versus time, referred to a datum temperature at which the rate of hydration is negligible (about 11 F).⁴¹ Powers⁴² has suggested that hydration stops at about 80 percent relative humidity, but it has been shown that this effect is negligible at a depth of about four in. from a free surface. At very high temperatures (say 180 C) the morphology of the hydration is changed, the surface area is materially reduced, and both creep and shrinkage are of negligible magnitudes compared to concretes cured at room temperature.

Ambient temperature and humidity after loading – Increase in thermal activity of water, accompanying temperature rise, increases the reaction rate of diffusion controlled processes such as creep and shrinkage. According to K. B. Hickey,⁴³ after six months of continual drying of concrete while under a sustained compressive stress of 800 psi:

1. At an ambient temperature of 290 F and ambient relative humidity (RH) of less than 1 percent, creep is about five times that occurring at 73 F and 50 percent RH;
2. Creep at any age less than six months varies directly with ambient temperature; between 73 F and 180 F the variation is linear; between 180 and 240 F the variation is curvilinear; and

3. At ambient temperatures of 130 F (where RH is 4 percent) or higher (where RH becomes less than 1 percent), creep is very rapid during the first 14 days under sustained load; from 14 days to 180 days under load, the rate is considerably less and the creep approaches a constant value.

C. R. Cruz⁴⁴ also found that creep increases as the ambient temperature increases. After being subjected to constant sustained compressive stress of 1800 psi for 5 hr: (1) the creep of concrete in dry air maintained at 1200 F is nearly 33 times that in dry air maintained at 75 F; (2) at 900 F, nearly 15 times that at 75 F; (3) at 600 F, about six times that at 75 F; and (4) at 300 F, about three times that at 75 F. Work by Ross and England⁴⁵ also shows that creep of concrete stored in air and under simulated mass curing conditions increases with temperature, the increase being greater in the range 68 to 180 F than for higher temperature up to 280 F. Neville and Nasser⁴⁶ have shown that a similar temperature effect is present when the concrete is stored under water. Some of these results are shown graphically in Fig. 1-11.

Timusk and Ghosh⁴⁷ showed that creep is increased when cement paste hydrates while under load. The creep component associated with the hydration process diminished logarithmically with the age at loading and was completely eliminated when hydration was halted at various stages of hydration, prior to loading, by cooling the specimens to -11 C. (Fig. 1-12).

According to Mitzel & Klapoc⁴⁸: creep in sealed specimens is about 20 percent less than similar unsealed specimens (values up to about 40 percent have been obtained). It should be noted that in structural members of substantial dimension the rate of moisture diffusion is sufficiently slow so that their behavior approximates that of sealed laboratory specimens.

EFFECT OF STRESS

EFFECT OF TYPE, VARIATION, MAGNITUDE AND DURATION

In the following discussion, creep is defined in its simplest context as the time-dependent strain caused by the application of a sustained stress. In general, uniaxial creep is determined experimentally by subtracting from the total strain at the time of observation, the algebraic sum of the initial strain at the application of stress and the shrinkage of an unstressed specimen between the time of stress application and the time of observation. In the case of torsion, shrinkage strains have no shear component and are, therefore, not subtracted from the time dependent torsional strains.

In the case of torsional or flexural loading, it is convenient to associate creep with sustained torques or moments instead of stresses, and the corresponding creep is measured as time dependent angular changes or deflections. Shrinkage strains are, again, neglected in interpreting the data.

Upon removal of stress (or load, torque or moment), there is an elastic recovery followed by a time dependent recovery designated creep recovery. The

creep recovery reaches a limiting value leaving an irrecoverable strain or permanent set.

In discussing the effects of stress on creep, the following factors are included:

1. Type of stress.
2. Variation of stress.
3. Magnitude of stress.
4. Duration of loading.

Although considerable research has been conducted on creep of cement paste, mortar and concrete, only creep of concrete is discussed.

Type of stress — The stress at any point in a body is usually considered in terms of two components: a normal stress and a shear stress. In general, creep tests have been conducted under conditions resulting in very simple states of stress such as uniformly distributed, uniaxial normal tensile or compressive stresses. Some tests have been conducted with uniformly distributed biaxial and triaxial stresses and some with normal stress gradients due to eccentric normal loads or due to flexure. Creep strains due to shear stress have been investigated only by means of torsional loading.

(a) *Uniaxial compressive stresses*: — By far, the vast majority of creep tests have been conducted under conditions of sustained concentric compressive loads on cylinders or prisms resulting in uniformly distributed compressive stresses which may be maintained constant with time or, in a few cases, increased to keep pace with an increase in strength. Because so much information is available under this state of stress, it is common practice to compare creep under all other states of stress with compressive creep.

Typical examples of creep under compressive stresses have been reported by Freudenthal and Roll,⁴⁹ Keeton,⁵⁰ Jones, Hirsch and Stephenson⁵¹ and Harboe.⁵² The papers cited form a small portion of the total literature on the subject.⁷⁸

The amount of creep to be expected depends upon the various factors previously discussed in this paper, but to illustrate the order of magnitude, to be expected, reference is made to Table 1-1. It is seen, for example, that a concrete with a compressive strength of 4000 psi (at the time of load application) which has been loaded to a working stress of 1800 psi (45 percent of its ultimate strength) would have an ultimate creep of approximately 1440 microinches per in.

(b) *Uniaxial tensile stresses*: — The creep behavior of concrete in tension is qualitatively very similar to that of compressive creep. There are, however, some differences of opinion regarding the relative magnitudes of tensile and compressive creep.

Early tests by Glanville⁵³ indicated that after six months, the creep in tension was almost exactly the same as that in compression.

Tests by Davis, Davis and Brown⁵⁴ indicated that the rate of tensile specific creep (creep per psi) was initially higher than the rate of compressive specific

creep, but after several weeks the rate of compressive creep was higher. The ultimate tensile and compressive specific creeps were approximately equal.

Hanson⁵⁵ found that the specific creep due to tensile stresses which were less than one-third of the ultimate tensile strength were equal in magnitude to the compressive specific creep for sealed specimens.

In a research investigation on the effects of biaxial stresses, Ross⁵⁶ conducted control tests on hollow cylinders subjected to longitudinal compressive stresses alone and to circumferential tensile stresses alone. He found that after 60 days of loading, the creep strains in tension and compression were equal for equal stresses.

Illston,⁵⁷ like Davis, Davis and Brown,⁵⁴ found that the rate of tensile creep was initially higher than that of compressive creep but at later times the reverse was true. No definite conclusions regarding the magnitudes of tensile and compressive creep could, however, be drawn. In one series of tests, the tensile creep was found to be approximately twice as large as the compressive creep for equal magnitudes of tensile and compressive stresses. In another series, however, Illston found that the tensile creep was approximately 20 percent higher than the compressive creep for equal stresses.

As with Illston, no definite relationship between the magnitudes of tensile and compressive creep were reached by Dutron⁵⁸ and L'Hermite⁵⁹ in their research on creep in tension.

It should be kept in mind that, as previously defined, tensile creep is evaluated by algebraic subtraction of the shrinkage and initial strains from the total strain at the time of observation. Since the applied stresses are necessarily low (approximately 100 to 200 psi), the initial strain and subsequent tensile creep strains are also low. In fact, in a concrete permitted to dry during test, the shrinkage strains may be as much as ten times as great as the tensile creep strains. Consequently, although initially the test specimen will exhibit elongation, it may with time begin to show considerable shortening due to shrinkage even though it is loaded in tension. It would appear that environmental conditions have a strong influence on the relation between tensile and compressive creep. Illston tried to examine this effect by conducting comparable tests of wet and drying concrete; these tests showed the tensile creep as 20 percent higher than the compressive creep.

(c) *Biaxial and triaxial stresses:* – Early tests by Davis, Davis and Hamilton⁶⁰ using triaxial loading found that the lateral stresses (required to maintain constant lateral dimensions) decreased rapidly during the first few days of loading and then more slowly at later ages. Although this is really a relaxation test rather than a creep test, it is equivalent to the phenomenon of creep recovery which occurs when sustained stress has been removed. Creep recovery occurs very rapidly for the first day or two after unloading and then very slowly until recovery is complete. They also found that the longitudinal creep that occurred when the lateral dimensions were kept constant was the same order of magnitude as the creep that occurred under uniaxial stresses with no lateral restraint.

Duke and Davis⁶¹ conducting tests on concrete cylinders under low triaxial compressive stresses (axial stress, $\sigma_1 = 500$ psi; lateral stresses, $\sigma_2 = \sigma_3 = 250$ psi) found that the presence of the lateral stresses reduced the longitudinal creep considerably compared to cylinders with no lateral stresses. They also found that the lateral compressive creep under triaxial compressive stresses exceeded the lateral extensions that occurred due to the effect of Poisson's ratio when cylinders were loaded with axial stresses only.

In the same series of tests previously discussed,⁵⁶ Ross tested concrete prisms under low biaxial compressive stresses. He found that the creep curves showed the characteristic creep curve obtained under uniaxial stresses with a high initial creep rate which decreases with time. In tests with equal biaxial stresses ($\sigma_1 = \sigma_2 = 300$ psi, $\sigma_3 = 0$), it was noted that although the creep due to the first applied stress σ_1 exceeded that due to the second stress σ_2 , the average of the two was equal to the creep due to the same one-dimensional creep of similar specimens. It would appear that the creep strains are influenced by the sequence of loading. Other tests conducted with unequal compressive stresses ($\sigma_1 = 500$ psi, $\sigma_2 = 250$ or 165 psi, $\sigma_3 = 0$) indicated that the creep in the direction of the major stress σ_1 is essentially equal to the creep that occurs due to uniaxial stresses of the same magnitude as σ_1 . This implies that Poisson's ratio for creep is zero and that volumetric creep on concrete can occur.

King and Timusk,⁶² on the other hand, found that the sequence of loading did not affect creep under biaxial loading. They also found creep Poisson's ratio to be dependent on the state of stress, and the presence of drying shrinkage. For sealed and unsealed cement pastes and concretes, creep Poisson's ratio was invariably higher for uniaxial loading than for equal biaxial loading. The superposition of shrinkage also had the effect of lowering the creep Poisson's ratio. While it did not vary with time for cement pastes, in the case of concrete it diminished with time, becoming equal to zero after about 100 days under equal biaxial load and 1/10 for corresponding uniaxial loading.

Ross also conducted tests on hollow cylinders subjected to combinations of longitudinal compressive stresses and equal circumferential tensile stresses ($\sigma_1 = 100$ psi compression, $\sigma_2 = 100$ psi tension, $\sigma_3 = 0$). In general, he found no consistency in the test results which he attributed to differences in the ambient humidity conditions under which the tests were run. He did find, however, that the sum of the magnitudes of the creep strains, under equal but opposite principal stresses was essentially equal to the sum of the magnitudes of the creep strains under simple compressive and tensile stresses ($\sigma_1 = 100$ psi comp., $\sigma_2 = \sigma_3 = 0$; $\sigma_1 = \sigma_3 = 0$, $\sigma_2 = 100$ psi tension).

In recent tests by Hannant,⁶³ cylinders sealed against moisture loss were tested under various combinations of axial and biaxial (radial) compressive stresses at two temperatures. Contrary to Ross' findings, the stresses at right angles to the axial stress reduced the creep in the direction of the axial stress when compared with axial creep due to uniaxial stresses. The reduction of creep was approximately proportional to the magnitude of the orthogonal stresses. One important result of Hannant's tests was that considerable creep was found to occur under a hydrostatic state of stress, even after two years, indicating again that volumetric

creep of concrete is possible. Contrary to Ross' results, however, a creep value of Poisson's ratio was found with a mean value of 0.18 which remained virtually constant over the two year test period. It should be noted that Ross' tests were conducted at variable relative humidities whose means, for each test series, varied from 57 percent to 87 percent. Although four of the surfaces of the test prisms were covered either by the bearing surfaces of the test equipment or by a water proofing material, the two larger surfaces were exposed to the ambient environment. Hannant's specimens, however, were sealed.

(d) *Normal stress gradients:* – Comparatively little information is available regarding creep due to initially imposed strain or stress gradients caused by eccentrically imposed sustained normal loads. This would be the type of loading condition caused by post tensioned prestressing forces if the post tensioning were periodically adjusted to maintain constant load. Based on tests conducted at the Technische Hochschule München under sustained eccentric loads, Rusch⁶⁴ stated, "... the determination of a basic law of stress distribution presupposes that, for each rate of strain in the various fibers of the flexure compression zone, there appear the same stresses as in corresponding fibers of a concentrically loaded prism. This need not be strictly true in reality, because a mutual interaction of the variously deformed fibers of the compression zone in flexure may be possible due to transverse deformation. In any case the resulting errors are small—as proven by comparative tests—and do not substantially change any derived conclusions."

Hansen⁶⁵ also stated, "Creep is not influenced by an eccentric loading when a linear relationship exists between stress and creep strain."

Roll, in a series of unreported tests on eccentrically loaded post-tensioned prisms, indirectly confirmed Hansen's statement.

In spite of the simplicity of the test procedure, there is a surprising lack of information available on the creep behavior of concrete under sustained flexural loading. Considerable work has been done with mortar and cement paste beams, and with reinforced or prestressed concrete beams, but very little with plain concrete. Davis, Davis and Brown⁵⁴ found, in tests on unreinforced beams subjected to sustained bending moment, that the initial creep of the tensile fibers exceeded the creep of the corresponding compressive fibers. It was also found that the location of the neutral axis remained in the same position during the test, moving neither up nor down.

Chang and Kesler⁶⁶ and Vaishnav and Kesler⁶⁷ reported that the creep in flexure was considerably higher than would be expected from tests under axial loads.

Similarly, tests by Zia and Stevenson⁷⁹ conducted on tee, triangular, and square cross-sections indicated that specific creep due to a nonuniform stress distribution was significantly greater than that due to a uniform stress on the same specimens. Bayazid and Pauw,⁸⁰ commenting on this latter paper, point out that same uncertainty exists when creep data is obtained by subtracting shrinkage strains of companion unloaded specimens which are of different shapes than the creep specimens. By measuring strains on several gage lines of eccentrically loaded prisms, Bayazid and Pauw were able to obtain creep strains without using separate shrinkage specimens. When this was done, they found that there was no

significant difference between the specific creep determined by uniformly loaded and eccentrically loaded prisms.

(e) *Shear stresses—torsion:* — As in the case of flexure, there is little information available on the creep of concrete due to shear stresses and that which is available is for sustained torsional moments only. Early tests by Anderson⁶⁸ indicated that creep in torsion showed the same general characteristics as creep in compression. They also showed that the specific creep (creep per psi) in torsion was approximately twice as great as the specific creep due to normal stress. This important fact was also confirmed by Duke and Davis.⁶¹ Pandit⁶⁹ briefly reported the results of one torsional creep test in which he found that the angle of twist under sustained torque was 2.43 times the short time angle of twist.

Variation of stress — The vast majority of creep tests have been conducted under constantly sustained loading conditions. Real concrete structures are, however, subjected to changing stresses. The stresses may be gradually declining, such as occurs due to creep and shrinkage losses in prestressed concrete members, or they may consist of increasing step functions due to construction stages. They may, of course, be a combination of the two, or even cycles of stress reversals. The variations may be gradual or rapidly fluctuating.

As previously indicated, complete removal of stress results in a time dependent recovery which reaches a limiting value resulting in permanent set. Partial removal of stress results in two opposing motions: creep recovery due to the removal of stress, and continued creep due to the remaining stress. The resulting strain will be the algebraic sum of the two time dependent strains.⁷⁰ Application of additional stress of the same type causes additional creep; application of additional stress of the opposite sense causes a reduction or even a reversal of creep.

To determine the creep of concrete due to variation of load with time, McHenry⁷¹ proposed a superposition principle taking into account the homothetic aging of concrete. Tests by Hanson⁵⁵ indicated that the superposition principle was valid for creep of concrete under loads which were approximately one-fifth of the ultimate strength of the concrete. It should be noted that the tests described by McHenry and Hanson were on sealed specimens.

Roll⁴⁹ conducted tests of concrete cylinders on which loads were periodically sustained for 28 days and then completely removed for 28 days. This load unload cycle was repeated three times after which the loads were sustained for an additional 203 days. It was found that the shape of the creep recovery curves were significantly different than the creep curves, thereby tending to refute the principle of superposition. These tests, however, were conducted on unsealed cylinders so drying took place during the test period.

In a significant series of tests, Ross⁷² conducted tests on axially loaded cylinders subjected to: (a) decreasing increments of stress; (b) increasing increments of stress; (c) decreasing and increasing increments of stress. He then compared three methods of evaluating time dependent strains due to variable stresses. It was found that the use of an effective modulus of elasticity gave poor results, i.e., underestimating strains when stresses are decreased, and vice versa. It also predicted complete recovery when stress was removed. The rate of creep method,

described in the paper, gave a fair agreement between theoretical and experimental results but it could not predict creep recovery following a reduction of stress. It tended to over estimate creep under a declining stress. The superposition method predicted creep strains fairly well. Although Ross' tests were on unsealed specimens, the tests were conducted at 93 percent relative humidity. Typical results for decreasing and increasing stresses are shown in Fig. 1-13.

Illston⁵⁷ conducted two series of tests in which concrete was subjected to stress reversals. In one series, the concrete was first loaded in compression and then in tension. In the second series, the cycle was reversed, i.e., first tension, then compression. Control specimens loaded only in tension or compression were also tested. It was found that the strains of the second cycle of loading were unaffected by the first cycle. It was also found that the sum of the strains of the control specimens at any time was essentially equal to the strain due to the two-phase loading at the same time. The results of the compression-tension cycle are shown in Fig. 1-14.

In addition to the three methods suggested by Ross for predicting creep due to variable stresses, England and Illston⁷³ proposed another method designated the "rate of flow method." This method requires more creep parameters than does the rate of creep method but gives better results. On the other hand, it requires far less data than the superposition method.

The above tests were conducted under variable sustained stresses which were applied as step functions. Tests under fluctuating stresses have also been conducted. Probst⁷⁴ found that the creep strains that occurred in a given length of time decreased as the frequency of rapidly fluctuating stresses increased. He also found that uniform fluctuations caused less creep than irregular fluctuations of comparable range. L'Hermite⁵⁹ also found that creep strains were reduced under fluctuating stresses compared with the creep due to sustained loads.

Magnitude of stress — It is a well established fact that below a certain value of stress level (ratio of applied stress to ultimate strength), creep of concrete is a linear function of stress. The particular value of stress level at which nonlinearity occurs depends primarily upon the mix parameters and the environmental conditions under which the creep tests are conducted. Stress levels of approximately 25 to 50 percent have been observed by various investigators.

Freudenthal and Roll⁴⁹ observed nonlinearity at 23 to 35 percent stress levels. Glanville⁵³ observed it at 22-1/2 percent; Davis and Davis⁷⁵ at 26-1/2 to 40 percent; Mamillan⁷⁶ at 30 percent; and Harboe⁵² at 35 percent. Keeton,⁵⁰ in an extensive series of tests of different size cylinders, tested at various relative humidities, found nonlinearity for all stress levels. King and Timusk⁶² observed linear behavior in the stress range of 20 to 40 percent f'_c whereas at low stresses the relationship was non-linear and vanished at the origin (Fig. 1-15). Practically speaking, however, even in these tests, creep could be considered as a linear function of stress up to stress levels of about 25 to 30 percent.

It is interesting to note that in an investigation of microcracking of concrete, Hsu, Slate, Sturman and Winter¹⁸ found that bond cracks at the interface between coarse aggregates and mortar began to increase noticeably at stress levels of

approximately 25 to 33 percent. It is reasonable to assume, therefore, that the nonlinearity of creep is due to the beginning of bond crack propagation, as discussed in the section on Effect of Aggregate-Cement Paste Interaction. Prior to the findings of Hsu, et. al., Roll^{49,77} had earlier suggested that a contributory cause of the nonlinearity of creep above a certain stress level was due to "localized fracture."

Typical examples showing the nonlinearity of creep at higher stress levels are shown in Fig. 1-16.

Duration of load — When concrete is subjected to a sustained stress that is less than approximately 75 percent of the ultimate strength, creep initially increases at a very high rate which soon begins to decrease. It is not known if the creep rate decreases to zero so that creep reaches an asymptotic value or if the rate reaches a constant value so that creep continues indefinitely. Long time creep tests by Troxell, Raphael and Davis¹³ show that even after 30 years of sustained load, creep is still increasing. Practically speaking, however, it would seem reasonable to assume that after a very long time, creep does reach an asymptotic value.

Troxell, et. al. found that, in general, one-quarter of the 20-yr. creep took place in the first two weeks of loading, about one-half in the first two to three months, and about three-fourths in the first year. Based on Troxell's results, Neville and Meyers⁹ showed the average increase in creep compared with the one-yr creep. These values are shown in Table 1-2.

It should be kept in mind that Table 1-2 represents only average values. Some concretes reached only two-thirds of their 20-yr. creep in the first year, and some reached five-sixths. It should also be noted that the stress levels at which these tests were conducted were such that creep was a linear function of stress. No information is available for very long-time creep at higher stress levels.

If the applied stress exceeds approximately 75 percent of the ultimate load, it has been found^{49,64} as in the case of lower stresses that creep initially increases at a very high rate which soon begins to decrease. The decrease, however, is not as marked as in the case of lower stresses and, after a few days or weeks, the creep rate begins to increase again finally reaching an unstable condition resulting in creep failure. The concrete thus exhibits the classical behavior of a material failing in creep with primary, secondary and tertiary creep periods. The secondary period is quite short. Roll found that concrete loaded to 80 percent of its strength failed in 10 days; Rusch found failure after approximately 7 days at the same stress level.

SUMMARY

This paper has attempted, where possible, to present the current state of knowledge in the area of time dependent deformation of plain concrete. There are many theories for the prediction of concrete creep available in the literature and no attempt has been made to reproduce them. Instead, the first section of

the paper has dealt with the general effects of the constituents and their interaction on the deformational behavior of plain concrete; in the subsequent sections, with the effects of various stimuli external to the system, such as environmental and applied stress.

The authors, who have summarized what has been observed and what is known, hope that future research will be built on what they have presented here.

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TABLE 1-1—CREEP OF CONCRETE OF DIFFERENT STRENGTH¹¹

Compressive strength at time of application of load, psi	Ultimate specific creep, 10^{-6} per psi
2000	1.40
4000	0.80
6000	0.55
8000	0.40

TABLE 1-2—AVERAGE INCREASE IN CREEP⁹

Creep after 1 year	1.00
Creep after 2 years	1.14
Creep after 5 years	1.20
Creep after 10 years	1.26
Creep after 20 years	1.33
Creep after 30 years	1.36

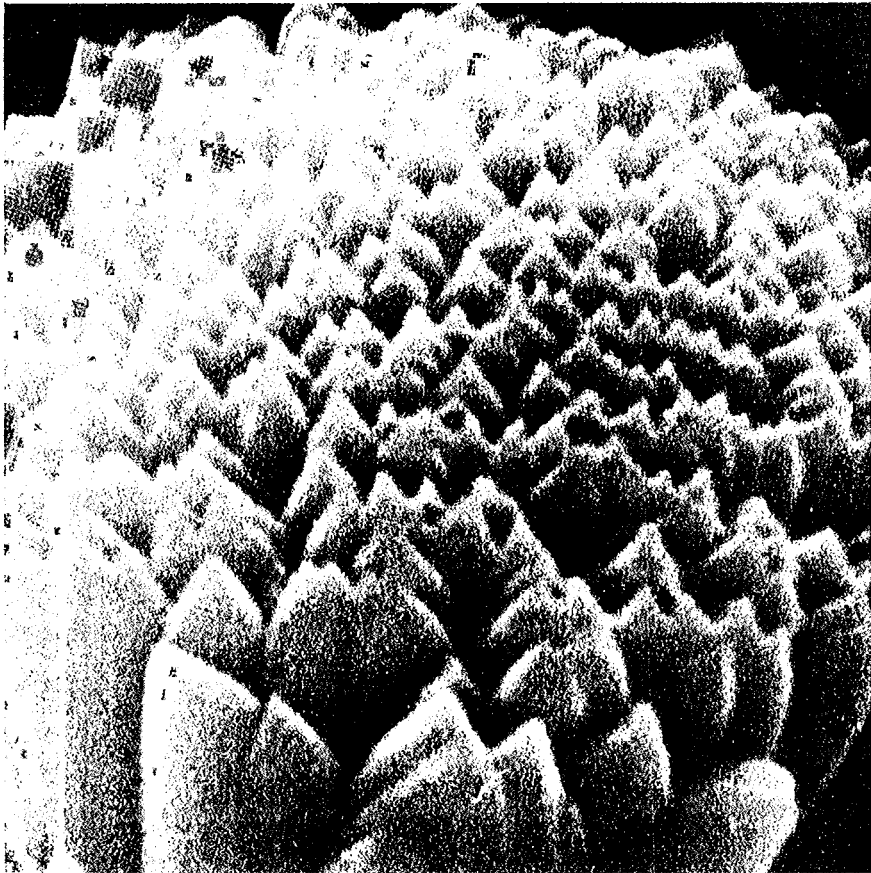


Fig. 1-1—Surface-scanning electron micrograph of calcium silicate hydrate “flowers” growing from nucleation sites on portland cement grains (X 28,000)



Fig. 1-2—Petal-shaped rolled foils of calcium silicate hydrate from ball-milled portland cement ultrasonically dispersed (X 100,000)¹

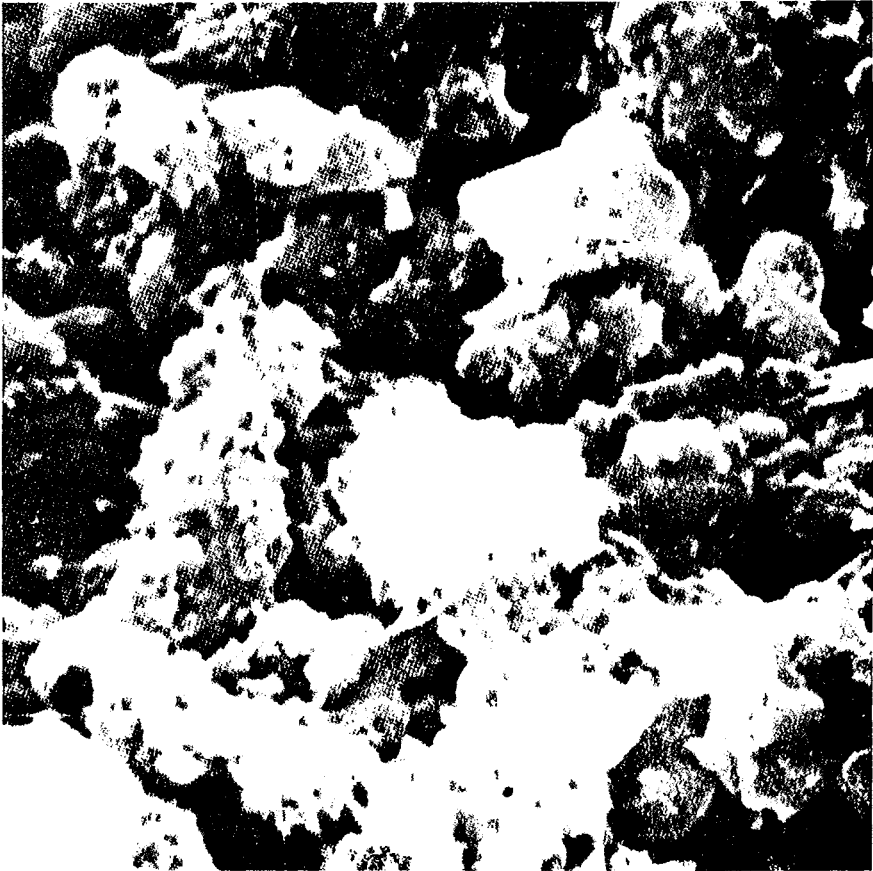


Fig. 1-3a—Amorphous material thought to be compacted or truncated flowers adjacent to crack from creep specimen of mix C (X 35,200)¹

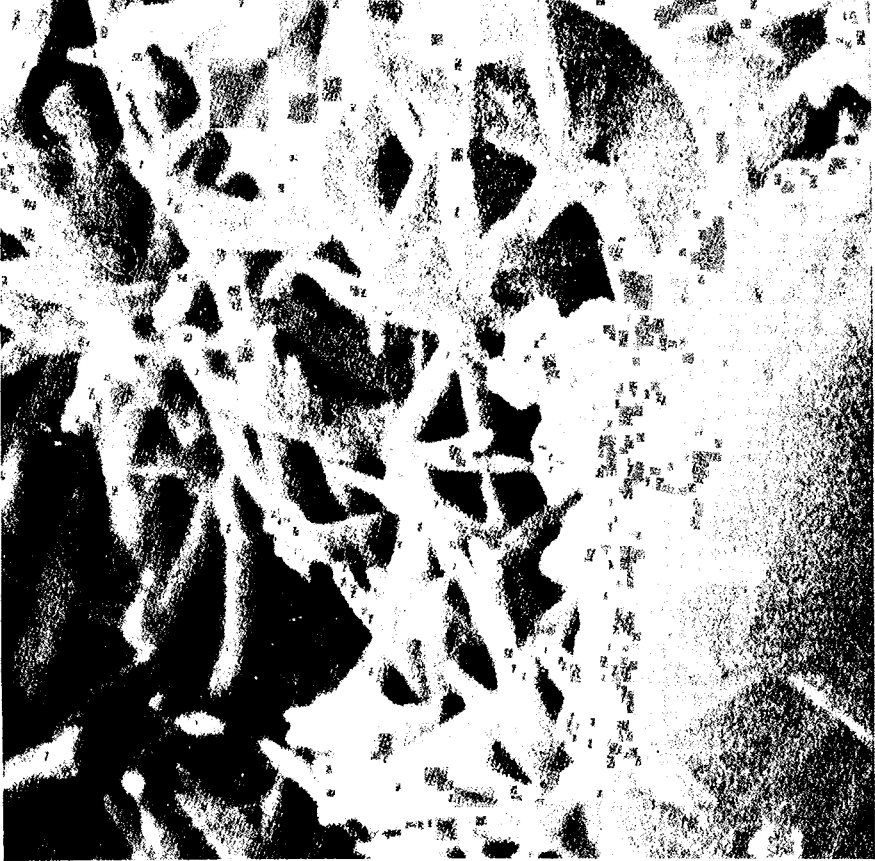


Fig. 1-3b—Uncompact flowers in shrinkage specimen (X 70,000)¹

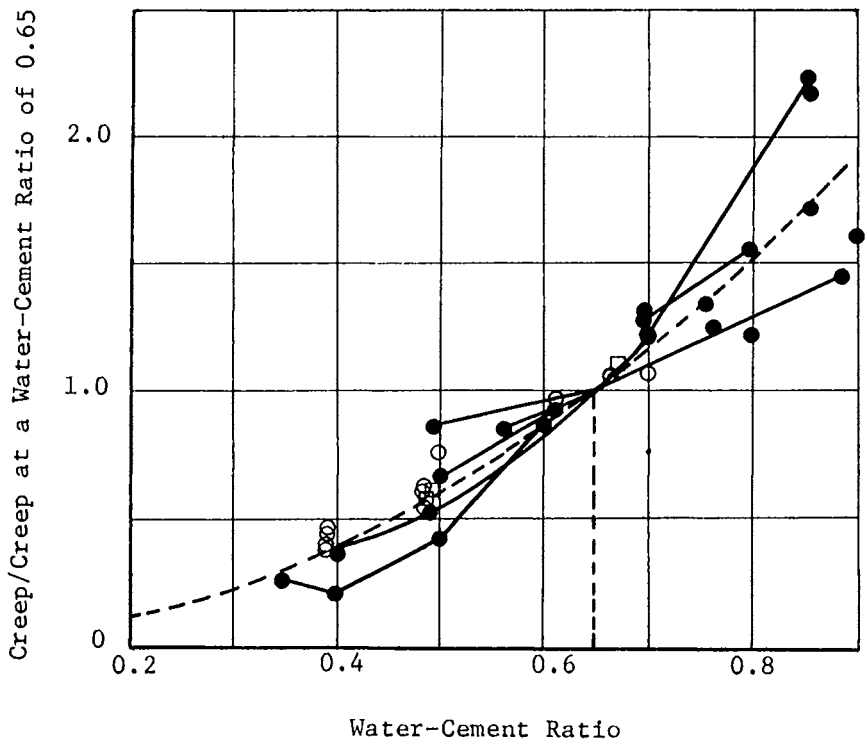


Fig. 1-4—Effect of water-cement ratio on creep^{9,10}

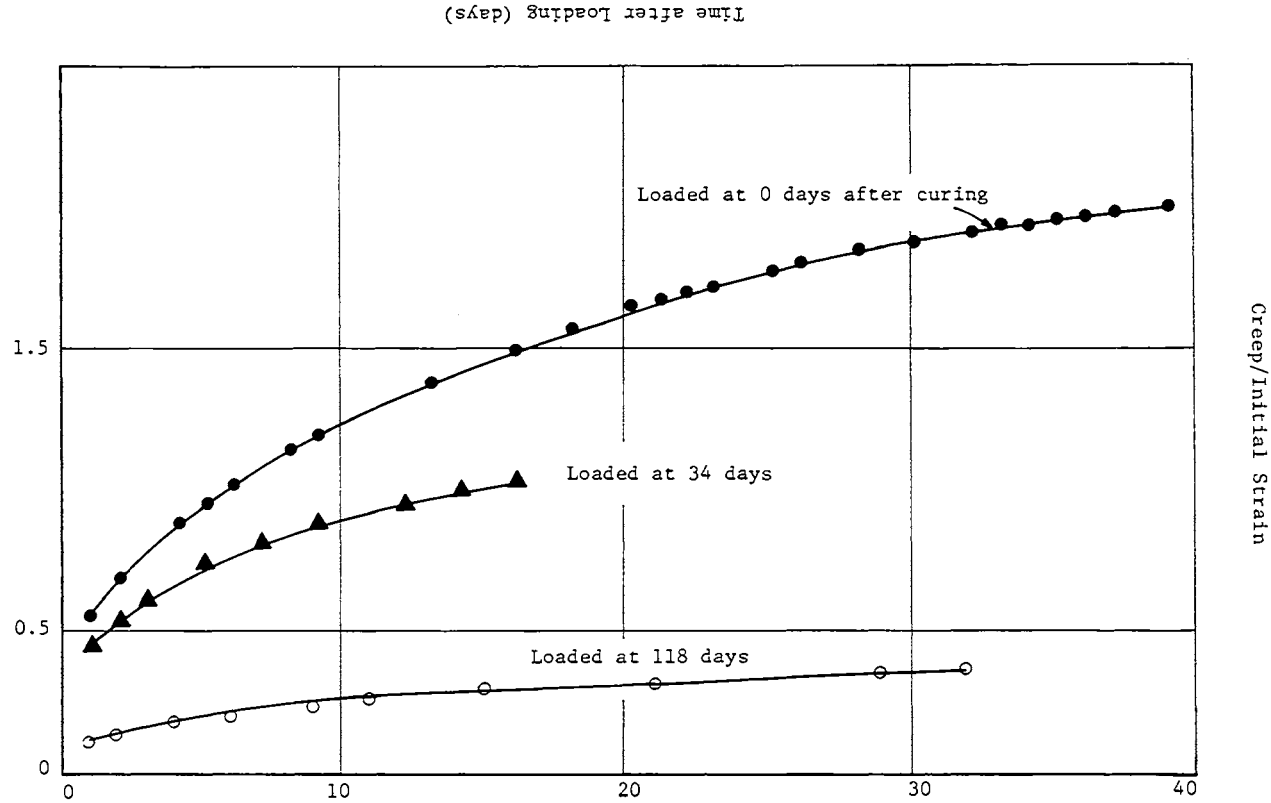


Fig. 1-5—Effect of age at time of loading on creep⁵

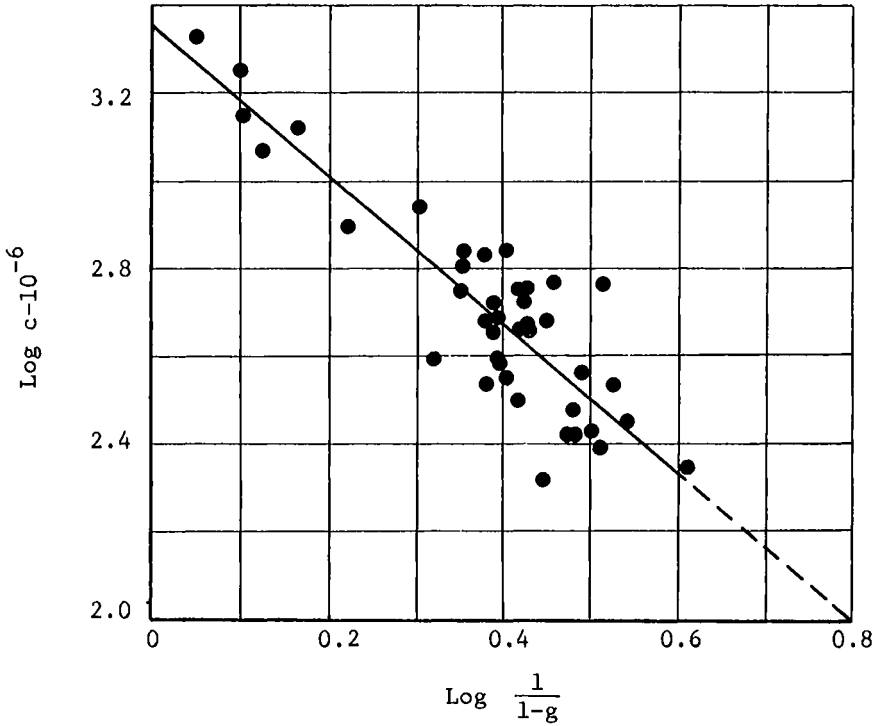


Fig. 1-6—Effect of aggregate content on creep¹²

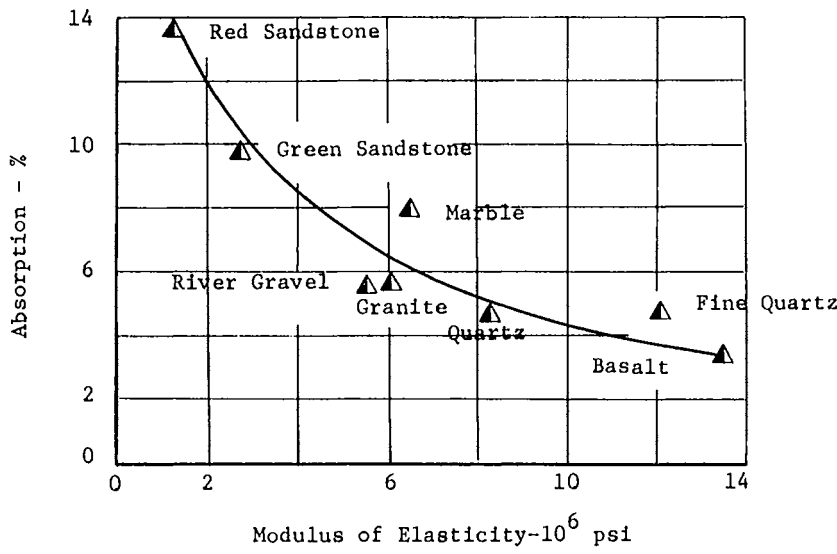


Fig. 1-7a—Effect of absorption on modulus of elasticity¹⁴

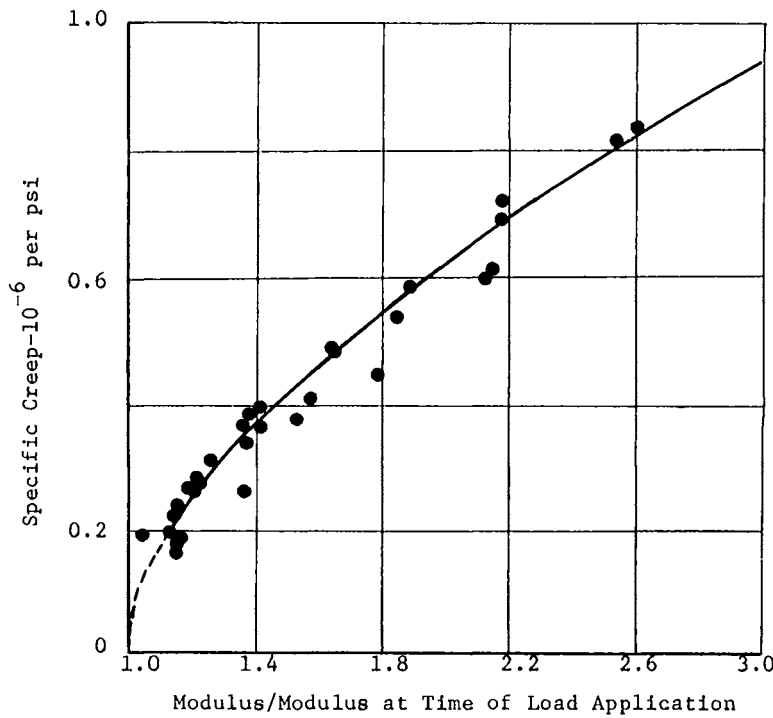


Fig. 1-7b—Effect of modulus of elasticity on creep¹⁴

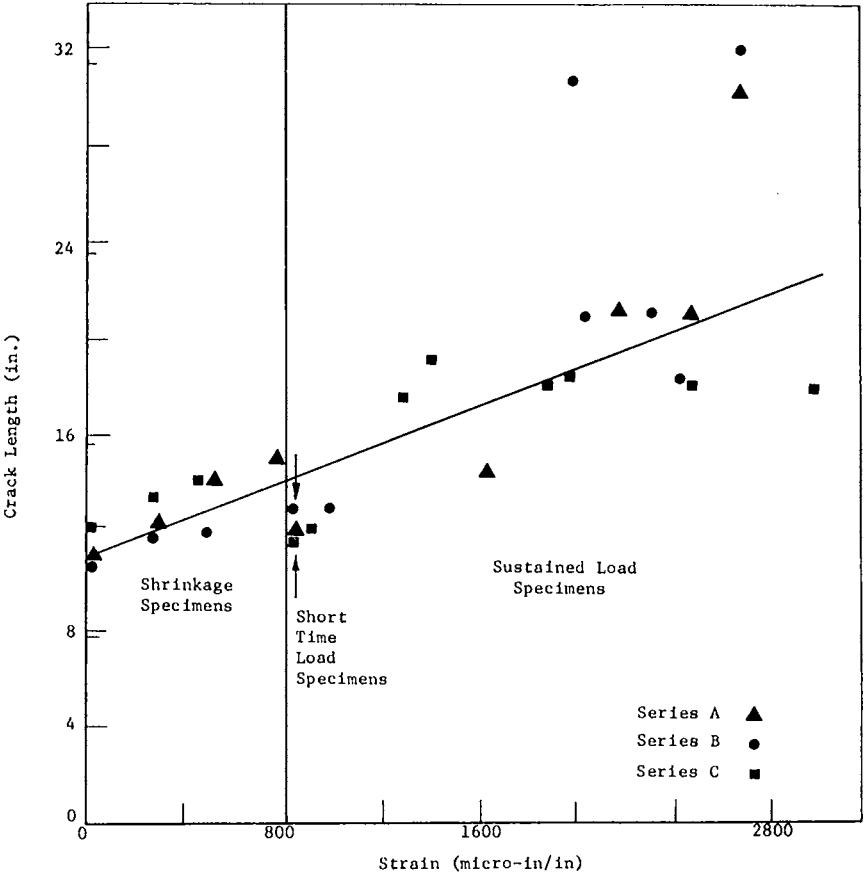


Fig. 1-8—Relation between time-dependent strain and cracking¹⁵

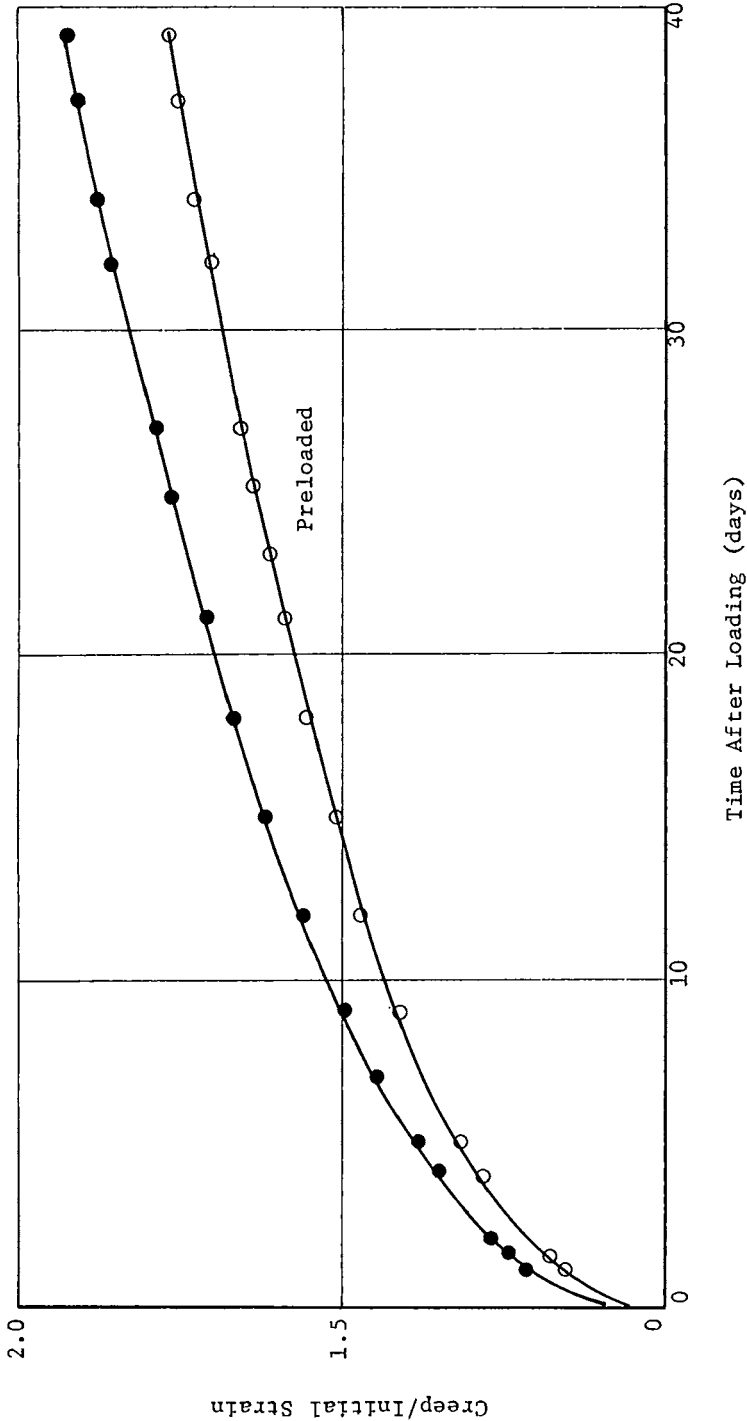


Fig. 1-9—Effect of prior cracking on creep^{1,2}

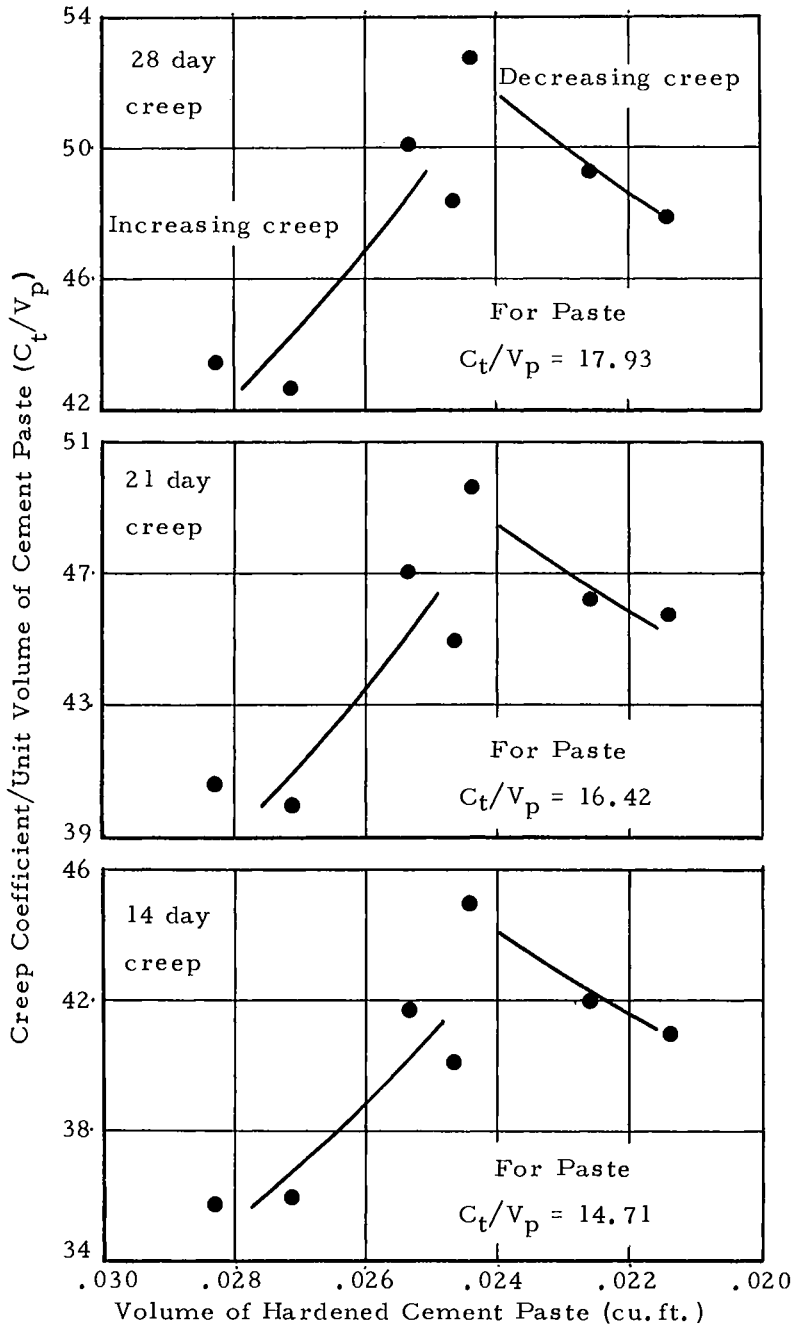


Fig. 1-10—Effect of coarse aggregate content on the creep response of concrete

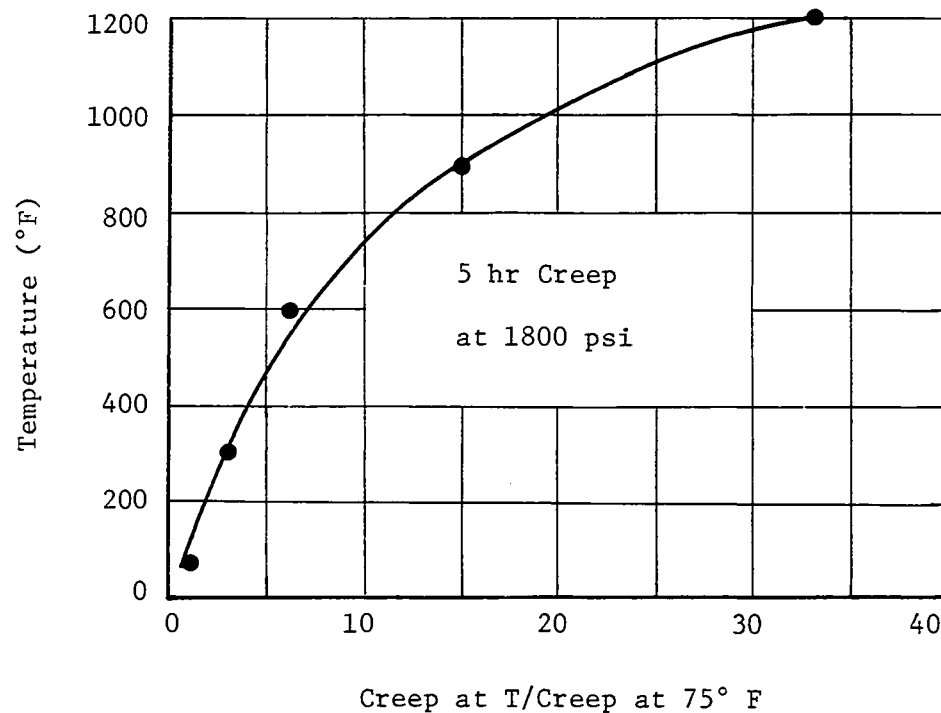


Fig. 1-11—Effect of temperature on creep⁴⁴

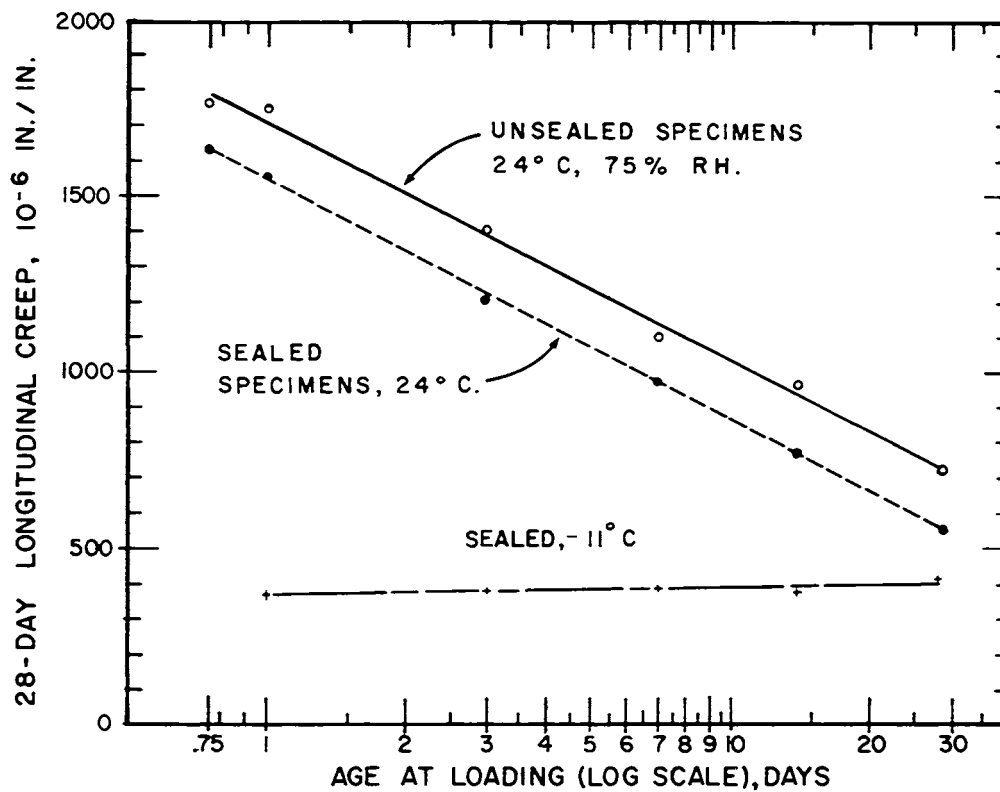


Fig. 1-12—28-Day creep of sealed and unsealed specimens⁴⁷

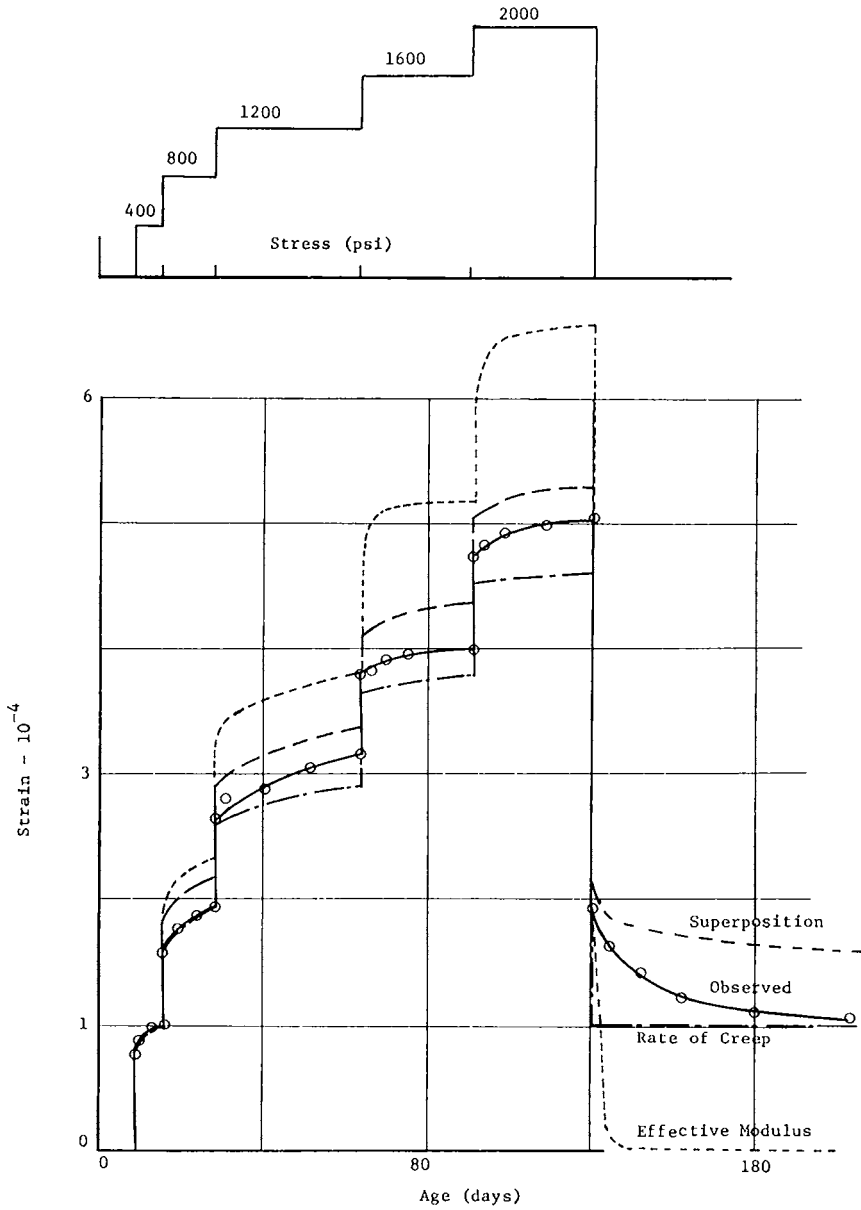


Fig. 1-13a—Predicted creep for specimens subjected to increase in stress¹²

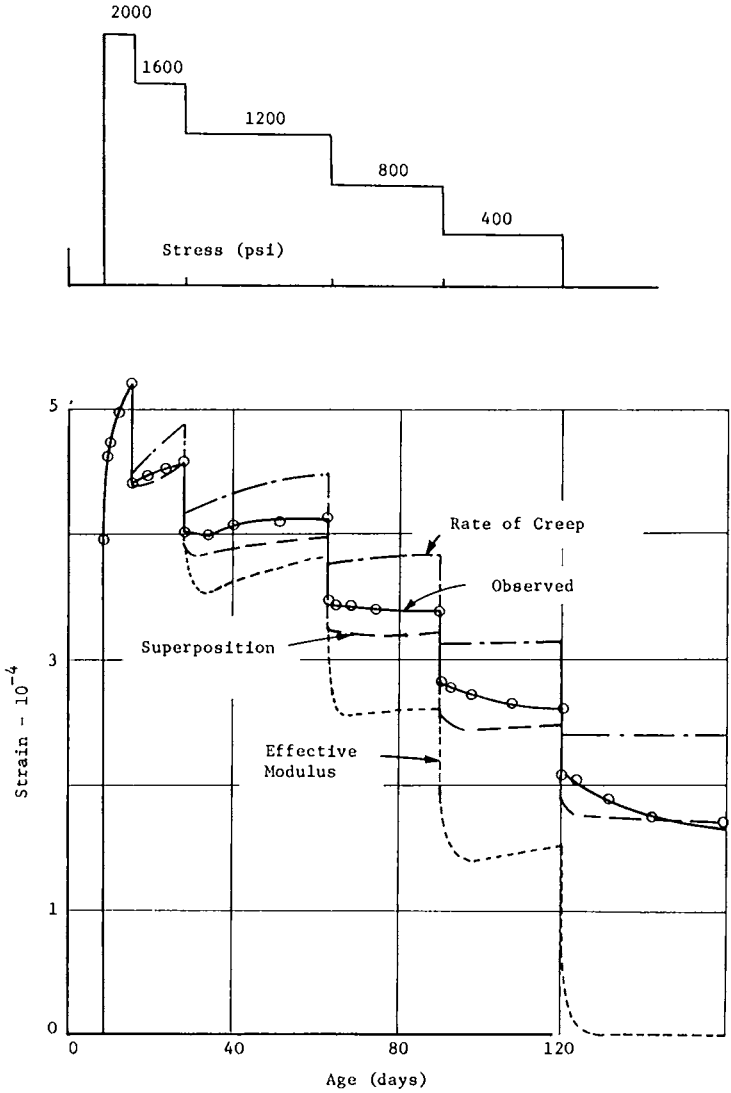


Fig. 1-13b—Predicted creep for specimens subjected to decrease in stress⁷²

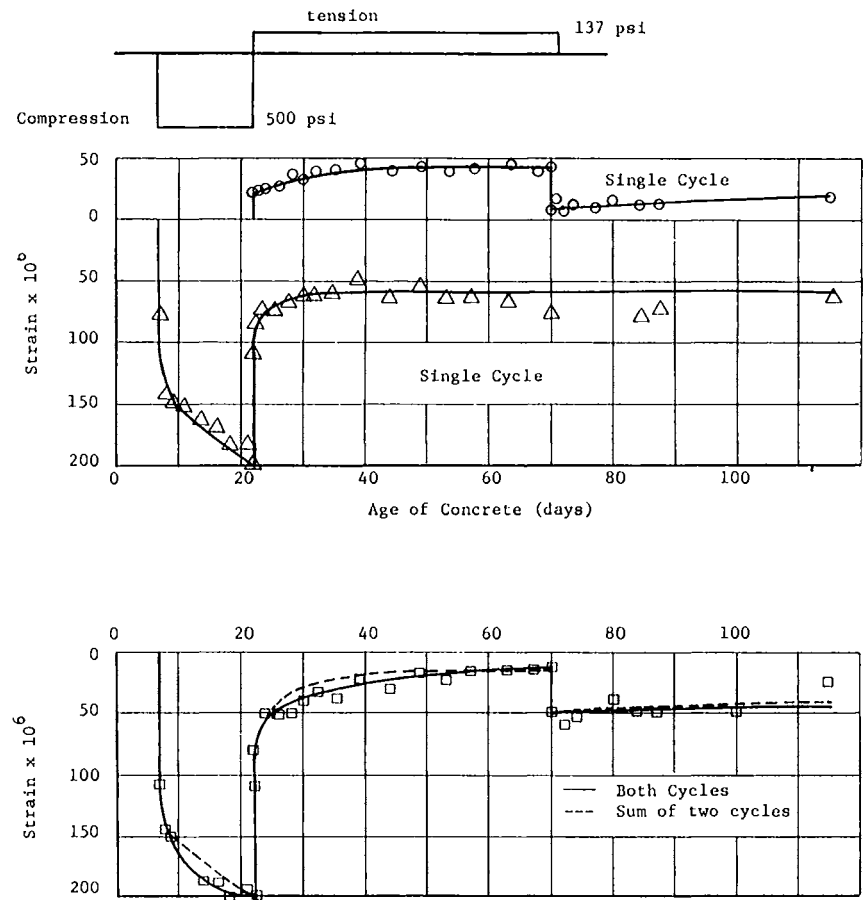


Fig. 1-14—Strain of concrete under two-phase loading⁵⁷

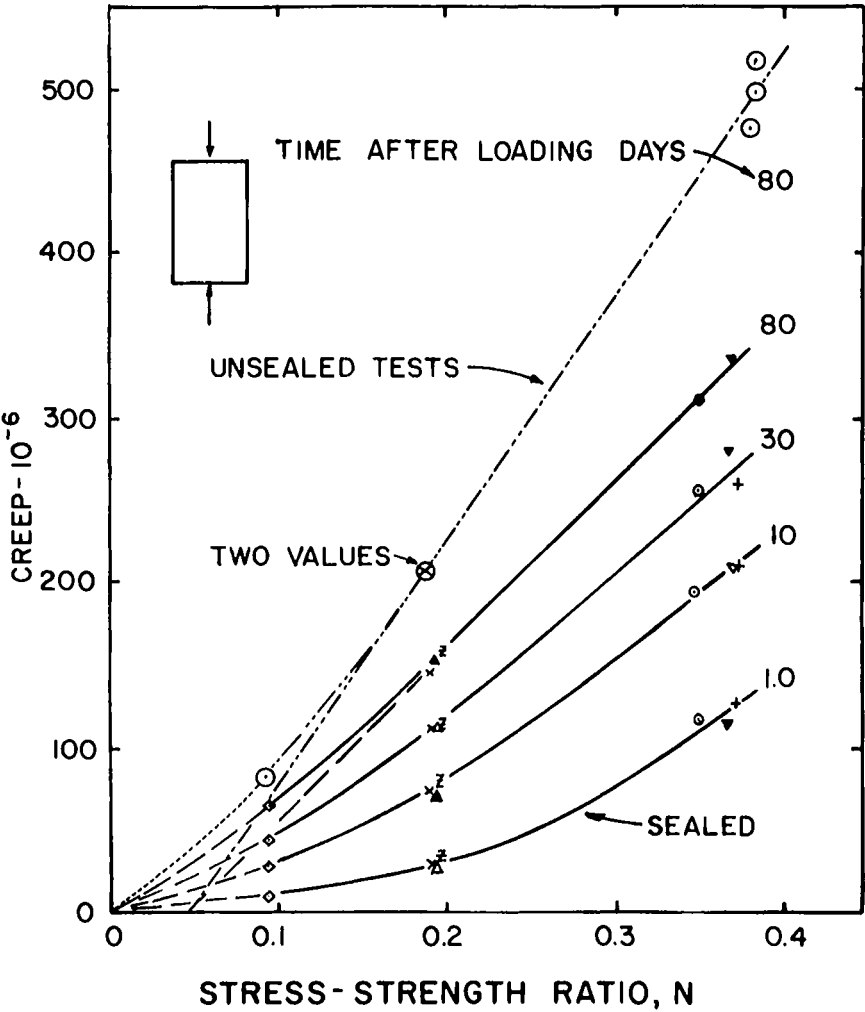


Fig. 1-15—Effect of stress-strength ratio on creep⁶²

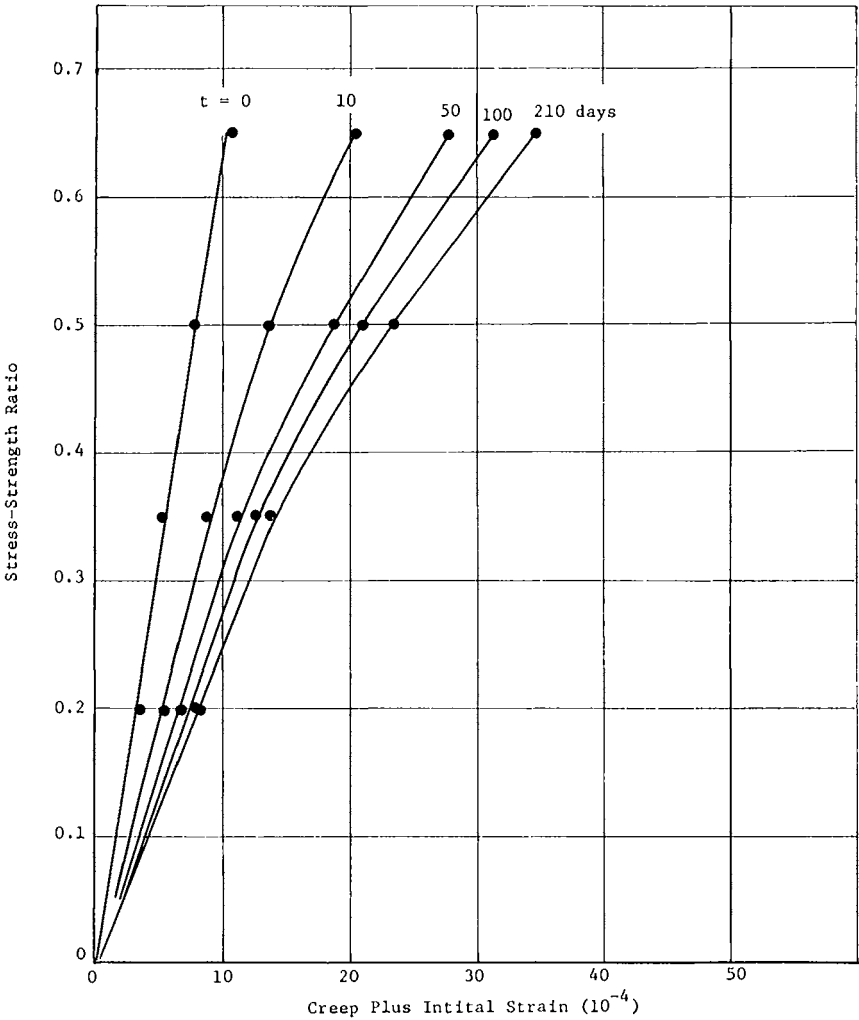


Fig. 1-16--Effect of stress-strength ratio on creep at high stresses⁷⁷

THERMAL AND SHRINKAGE STRESSES— THEY DAMAGE STRUCTURES!

By JAMES N. De SERIO

This paper outlines some of the many problems and structural distress conditions which can be caused by insufficient provision in the design and construction if thermal changes are neglected. Four structures, completely different from each other are discussed. Each showed signs of cracking and distress, even though they were designed and detailed in accordance with ACI 318 and ACI 315. An analytical study of each of the structures indicated that severe stresses were set up by thermal changes, and that for such conditions careful design and detailing must be provided. It is pointed out that normal code requirements for temperature reinforcement are adequate only for average conditions which occur in a mild climate area. However these requirements are inadequate in areas of extreme temperature changes, or in areas which have a hot climate or a very cold climate, or in areas which are subject to continual high winds or variable humidity conditions. In each of the structures studied it is pointed out that the very small saving in reinforcing steel resulted in a very expensive repair operation.

Keywords: cracking (fracturing); drying shrinkage; environments; failure; foundations; frames; garages; humidity; reinforced concrete; shrinkage; silos; stresses; structural design; temperature; thermal expansion; thermal stresses; torsion; waffle slabs; walls.

□ The practice of designing for thermal and shrinkage stresses is the most neglected part of today's design practice. The problem is complex due to the interaction of forces and to the variable climatic conditions which may cause them. Structures constructed in an area of relatively high temperature-humidity conditions will require an entirely different analysis and treatment from structures built in a hot dry area or from structures in the northern climates which have a wide variation in both temperature and humidity, and where these wide ranges can occur in a matter of hours. Other factors which influence the condition are such items as degree of enclosure, exposure to prevailing winds and sunlight. So much time is spent on refining design methods to save the last ounce of steel that the problem of thermal stresses has been relegated to the statement—"make provision for it!" Yet no competent authority has prepared any suitable criteria, data, or procedures to accomplish this except the ACI Committee on Chimney Design, and this was a very special type of structure.

In our regular practice we are often called in to study and report on structures which show signs of structural distress and in a large majority of the cases the basic cause was due to inadequate provision for thermal stresses in one form or another.

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EXPOSED CONCRETE DECK

Such a problem developed on a three story open parking ramp in upstate New York. The ramp dimensions were 267 by 243 ft with column bays approximately 30 by 27 ft. The construction of the floor was a concrete waffle system of 6 in. wide joists spaced 36 in. on centers in each direction with a solid section over the columns. The depth was 12 in. plus a 4½ in. structural slab. Design and construction was in accordance with ACI 318-51, and details were made according to ACI 315-51. A one in. concrete topping was applied to each of the lower floor slabs, but was omitted on the top level where a 2 in. bituminous wearing surface was applied.

The structure is exposed to ambient temperatures which range from -5 F to 90 F, but it was found that the surface temperature of the top slab often reaches a temperature above 140 F. During winter weather the temperature on the under side of the slab is well below the freezing level while the upper surface is well above freezing during the daylight hours, with a differential temperature of 30 to 40 F.

The bottom reinforcing bars in the mid bands were detailed and installed with a length equal to the center to center of column spacing with no laps provided. At these points the normal stresses are flexural compression at the bottom and flexural tension at the top, with shear at a very low stress level.

During an inspection of the structure it was found that in each rib at the column line there was a crack, open at the bottom and running upwards at an angle of approximately 15 deg to the underside of the slab. In locations where there was no cross joist the crack occurred directly at the point where the bars were cut. In the places where there was a cross joist at the centerline there was a crack on each side of this cross joist. An analysis indicated that flexural tension due to temperature stresses was in excess of the compressive stresses due to dead and live loads and the cracks were due to failure of the concrete in tension at those points. Further studies showed that this condition did not exist on the two lower levels which were not exposed to the high differential temperature between the top and bottom surfaces. (Fig. 2-1)

Aside from aesthetic considerations the cracking has not injured or reduced the capacity of the structure. The condition could have been prevented by

lapping the bars at this point for continuity, regardless of the minimum stress conditions indicated by the normal loading conditions.

FORCED DRYING SHRINKAGE PROBLEM

Changes in temperature and humidity combined can create more severe loading conditions than the normal design requirements, and the effect cannot be predicted.

We designed a large school complex consisting of two and three story concrete frame classroom wings with connecting utility building of one-story construction including auditorium, gymnasium, music rooms, etc. The total ground floor area was in excess of 100,000 sq ft. The foundations were installed in medium clay, with a 3 ft 6 in. crawl space under the ground floor. Hot and cold water and steam distribution lines were run in the crawl space. Open ventilation was provided for the space. An organic type of insulation was used over the steam and hot water lines. During periods of changing temperature and humidity conditions a dew point was reached where condensation was formed on the insulation to such an extent that the insulation became fully saturated and ineffective and allowed the crawl space to become very hot and moist. To further complicate matters the maintenance staff started to use portions of the space for storage of desks, chairs, and other school equipment. In order to create a more suitable atmosphere for the storage large openings were cut in the walls at one end and exhaust fans placed at the other end to ventilate, cool and dehumidify the space. At the time we were called in the temperature in the crawl space was above 100 F and the relative humidity was down below 30 percent. The exposed clay was dried out and fissured with random fissures up to 2 in. wide and up to 3 ft deep, to such an extent that the footings were endangered. The walls, beams and floor construction completely dried out and the resulting shortening due to shrinkage caused extensive tension cracks in the walls and slabs, caused beams to pull at the walls and break the seats. The walls curled due to being wet on the outside and hot and completely dried out on the inside. The damage was extensive and required a large amount of corrective work.

The first correction was to replace the pipe insulation with more efficient material to cut down the heat transfer and lower the temperature. Then the area was flooded for an extended period of time to restore the moisture content to the soil and finally structural repairs were made where distress was indicated.

The problem of this structure was unique in that it was caused by a breakdown of material which normally has little or no effect on the structure, and over which the designer has no control.

TORSION AS A RESULT OF THERMAL AND SHRINKAGE STRESSES

The resulting effect of combined stresses due to shrinkage and temperature is more difficult to recognize and analyze and the distress indications can be very misleading. Such a condition developed in a hospital built in Buffalo during World War II under the design regulations of the War Production Board.

The structure is a three story building approximately 320 ft long with no expansion joints. Floors and roof are concrete joists with clay tile fillers and clay tile soffits bearing on two exterior masonry bearing walls and two interior masonry bearing walls. Basement walls are of plain concrete without reinforcing steel. The basement is approximately 6 ft below the surrounding grade and the top of the walls is about 3 ft above grade. One long wall faces the west.

After a period of time, and usually during periods of hot dry weather, it was found that the clay tile fillers or the soffits would split and the bottom would fall (along with the plaster). In some instances patients were hit and injured by the falling plaster and tile. All of these were on the west side of the building.

The investigation showed a series of cracks in the top of the concrete wall, open at the top and closed at the surface of the grade. The brick masonry walls had diagonal step pattern cracking at the corner of most of the windows. The analysis indicated that the interior walls are in a state of constant temperature-humidity. The bottom of the exterior walls is exposed to earth and is in a state of constant temperature humidity whereas the top of the wall is exposed to a variable temperature humidity condition.

With the length of wall involved it was found that the top of the wall becomes convex during high temperature periods and becomes concave during low temperature periods, and at the same time the interior bearing wall is in a static condition. This differential curvature between the walls causes a torsion and twist in the floor construction that shears the webs of the clay tile fillers causing the bottoms to fall. The use of expansion joints in the structure could have eliminated the problem.

THERMAL STRESSES IN SILO WALLS

One of the most common type of structures that show signs of distress due to temperature are concrete silos. Many are seen that have a series of ugly scars where the cracks have been patched,—they look like road maps. In many cases these are the direct result of insufficient provision in the design for temperature stresses. Text books and design hand books gloss over the subject of temperature steel, or omit reference to it altogether. Even ACI Committees place little emphasis on the subject in their recommendations. In the proposed report of ACI Committee 313, “Suggested Practice for Bin Wall Design,” Par. 5.5 states: “vertical shrinkage or temperature reinforcement shall be placed in exterior walls of cast-in-place construction. The minimum amount of such reinforcement shall be .0015 of the gross area of the concrete. Jack rods in slip form construction may be considered as part of this reinforcement as limited by the connection or splice provided. The maximum spacing between vertical reinforcement shall not exceed three times the wall thickness.” This minimum provision for temperature steel in an exposed location is even lower than recommended by ACI 318 for an enclosed and protected condition.

We were called upon to investigate a nest of concrete silos which showed signs of distress shortly after being put into operation. The structure is

approximately 90 by 120 ft in plan, 120 ft high and the walls are 8 in. thick with reinforcing steel placed near the center of the wall. There are 12 circular silos and the cellular spaces between, for the storage of bulk cement. The structure is located on one of the Great Lakes and receives cement delivered by lake carriers and unloaded by an airslide. The temperature of the cement as delivered varies from 160 F to 200 F. The friction of the grains on each other during handling elevate this temperature by as much as 60. Ambient temperatures vary from summer highs of +90 F. to winter lows of -20 F. Shortly after the silos were put into operation a series of vertical and horizontal cracks occurred. As these were patched and repaired they reopened, and additional ones appeared. These cracks extended completely through the walls allowing water to enter and contaminate the product. The designers blamed it on the contractor stating that they used too much water in the mix and placed the concrete too wet,—not very convincing since this was constructed by the slip form method.

A design analysis indicated that no provision had been made for flexural stresses caused by the temperature differential between the inside and outside faces of the concrete walls. It was found that a temperature differential of from 60 F to 120 F could occur between the inside and outside face, and this differential could be in either direction depending upon whether the bin was full or empty. This difference in temperature between the two wall faces induces bending moments in a vertical as well as horizontal direction with resulting stresses at the face which exceed the modulus of rupture of the concrete and cause continued cracking. The use of a few additional tons of reinforcing steel at the time of construction could have saved a very expensive remedial project. (Example sheet 1) (Fig. 2-2)

CONCLUSIONS

It is my personal feeling that more emphasis should be placed on provisions for stresses due to shrinkage and temperature change in all of ACI Standards. This should go beyond the mere cautioning stage, and actually set criteria which are realistic and adequate for the condition. As it is now under the ACI Standards the same minimum provision is set for Alaska as for the southern states, with the requirement being inadequate or incorrect for either of the areas.

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2. Taylor, C. P., and Turner, L., *Reinforced Concrete Chimneys*, Concrete Publications, Limited, London, 1940, 80 pp.
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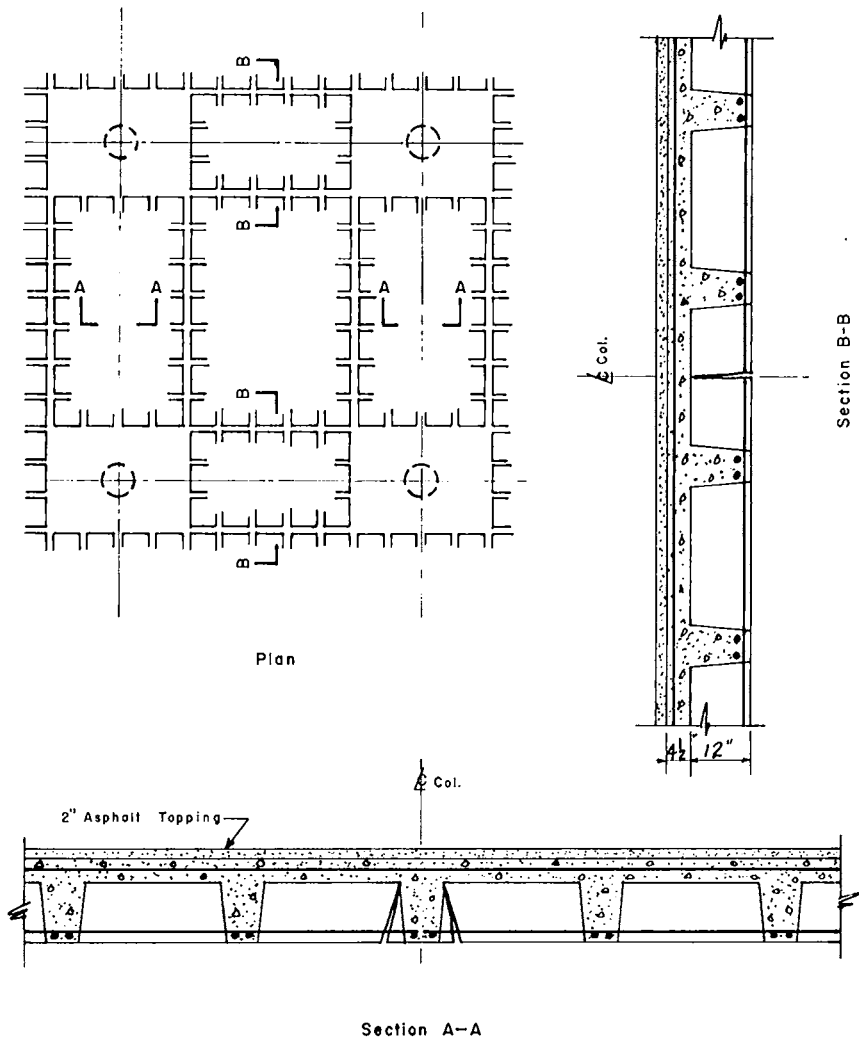


Fig. 2-1—Details of exposed parking deck

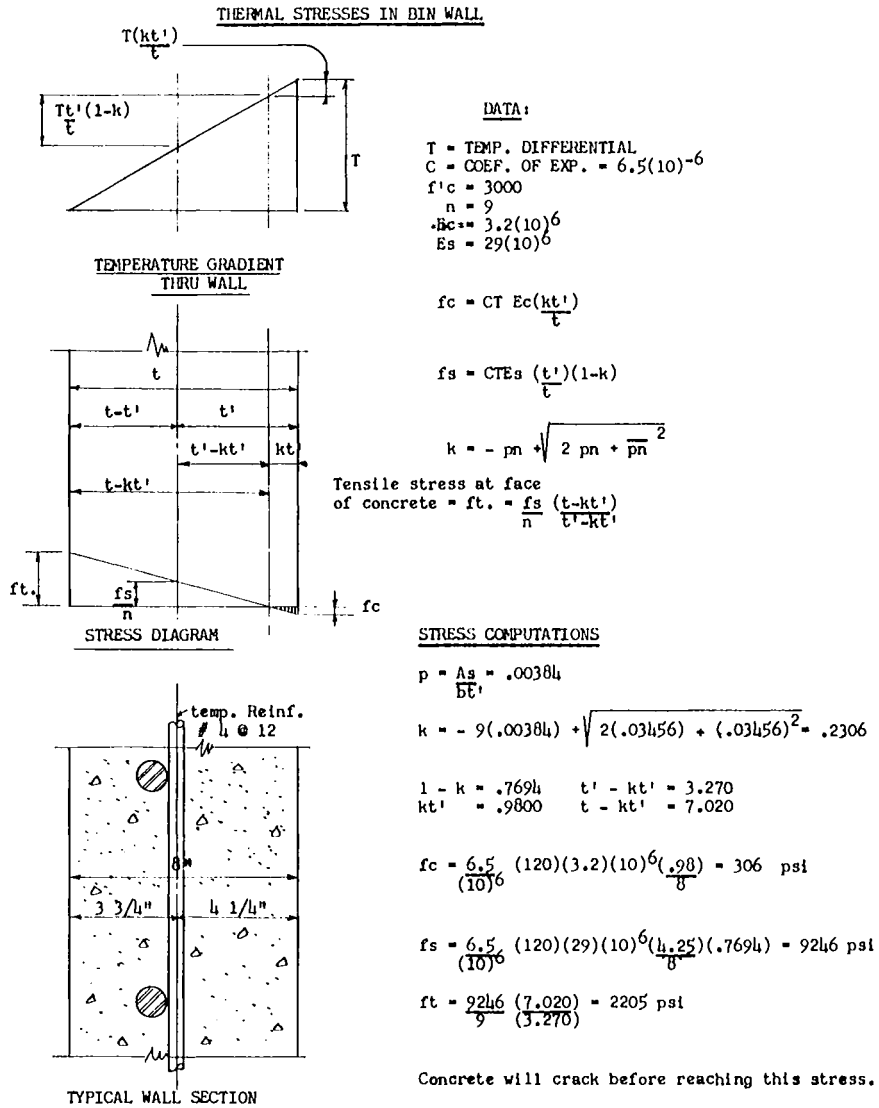


Fig. 2-2—Design example no. 1

PREDICTION OF CREEP, SHRINKAGE, AND TEMPERATURE EFFECTS IN CONCRETE STRUCTURE

Reported by ACI Committee 209

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This paper presents a review of methods for predicting creep, shrinkage and temperature effects in concrete structures as related to reinforced beams and one-way slabs, two-way construction, prestressed beams, and composite beams. The paper is generally consistent with the ACI Building Code (ACI 318-71) and includes material indicated in the Code, but not specifically defined therein.

Keywords: age; beams (supports); camber; composite construction (concrete to concrete); compressive strength; concrete slabs; concretes; cracking (fracturing); creep properties; curing; deflection; flat concrete plates; flexural strength; girders; lightweight-aggregate concretes; modulus of elasticity; moments of inertia; precast concrete; prestress loss; prestressed concrete; reinforced concrete; reviews; shoring; shrinkage; strains; structural design; temperature; two-way slabs; warpage.

☐ The prediction and control of time-dependent deformations and their consequences in concrete structures is exceedingly complex and involves a relatively

*Member of Subcommittee 2 which prepared this report.