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DEVELOPMENT OF EXPANSIVE CEMENTS

BY

G. L. KALOUSEK

Synopsis: This report reviews the research and development of expansive cements during the past 50 years which culminated in commercial applications of these as competitive building materials. The studies of Lossier in the 1940's, who is credited with discovery of expansive cements, showed promise but none of his findings were reduced to practice. The Soviets developed expansive cement mortars in the 1950's. However, through 1968 they reported small volume productions mostly used in precast units. Klein and his associates at the University of California, Berkeley, starting in 1958 with a newly discovered expansive agent developed two types of expansive cements, shrinkage-compensating and self-stressing. The first field trial of the shrinkage-compensating cement in 1963 was successful and the production and application of this type of cement has been expanding rapidly in this country and Japan. The self-stressing concrete continued in developmental stages until 1971 when a factory started production of self-stressed concrete building modules. This operation is reported to be proceeding satisfactorily.

Keywords: aluminate cements and concretes; calcium aluminates; calcium sulfoaluminates; drying shrinkage; ettringite; expansive cement concretes; expansive cement type K; expansive cement type M; expansive cement type S; expansive cements; hydration; self-stressing cements; self-stressing concretes; shrinkage-compensating cements; shrinkage-compensating concretes.

ACI member, G. L. Kalousek, Research Adviser - Concrete General Research and Development Division, Bureau of Reclamation, Denver, Colorado (now consultant in the concrete industry) holds Ph.D. in Chemistry from the University of Maryland. He is member of ACI Committees 114 - Research and Development, 115 - Current Research, 223 - Expansive Cement Concretes, and 561 - High Pressure Steam Curing. He is author of many publications, holds several patents, and is a professional Engineer (Colorado).

INTRODUCTION

The development of expansive cements is covered in several excellent reports, Halstead(1),* Mather(2), Li(3), North(4), Mehta(5), Aroni, Polivka and Bresler(6), and ACI Committee 223 report(7). The annotated bibliography by Lyon and Butzine(8) supplies abstracts from 1936 to 1965. References 1 and 5 are concerned largely with chemistry, Reference 4 with patents, and the others are general reports. This report is slanted more to a coverage of development of expansive concretes and products as competitive materials at the marketplace.

Expansive concretes are divided into two categories, shrinkage-compensating and self-stressing. The ACI(7) definitions are: "Shrinkage-compensating concrete is an expansive cement concrete in which expansion if restrained induces compressive stresses which approximately offset tensile stresses in the concrete induced by drying."

"Self-stressing concrete is an expansive cement concrete in which expansion, if restrained, induces compressive stresses of a high enough magnitude to result in significant compression in the concrete after drying shrinkage (and creep**) has occurred.

The compressive stresses induced by shrinkage-compensating cements in concrete are about 25 to 100 psi (2 to 7 kg/cm²), and by self-stressing cements above about 100 to about 1,000 psi (7 to 70 kg/cm²).

All expansive cements consist predominantly of portland cements, about 94 to 70 percent. The chemical compositions are similar to those of portland

* All numbered references refer to listings in Literature Citations.

** Added by author.

cement except for higher sulfate contents (about 4.5 to 6.5 percent $\bar{S}^*$), generally higher alumina contents (about 5 to 9 percent A) and generally lower silica contents. The lower the amount of the expansive agent the greater is the similarity in compositions. Representative oxide compositions of the three types of shrinkage-compensating cements currently being produced in the United States are given in following table.

Type cement	K	S	M-X
SiO ₂ , percent	21.6	18.3	18.4
Al ₂ O ₃	4.8	8.8	7.7
Fe ₂ O ₃	2.8	1.9	2.1
CaO	62.4	63.7	61.1
MgO	0.7	1.1	3.0
SO ₃	6.2	5.3	4.7
Na ₂ O	0.18	0.04	0.04
K ₂ O	0.18	0.30	0.04
L-O-I	1.06	-	-

The three types of shrinkage-compensating cements are characterized by different alumina-bearing agents, $C_4A_3\bar{S}$ in Type K, $CA + C_{12}A_7$ (calcium aluminate cement) in Type M-X, and C_3A in Type S. The sulfate is present as gypsum, hemihydrate, anhydrite and, in part in Type K cement, as $C_4A_3\bar{S}$. From the earliest days of expansive cements, researchers studied effects of many A-bearing reactive agents, alunite, bauxite, ignited bauxite, clays, ignited clays, hydrocalumite (synthetic C_4AH_{13}), calcium aluminate monosulfate (monosulfate), slags and others in addition to those already mentioned. The search continues. There remains little doubt that ettringite formation is the source of expansive energy according to the reaction, $6C + A + 3\bar{S} + 32H \rightarrow C_3A \cdot 3\bar{CS} \cdot H_{32}$. No one of the oxides actually occurs as a reactant except water in part. Phase rule dictates expression of the reactants as oxides in a general reaction.

The practical application of expansive cement is based on controlling the kinetics and extent of the ettringite formation reaction. Ettringite starts forming the instant water comes into contact with the cement grains. Many factors affect early and abundant formation of ettringite among which are aeration of the cement, frequently water-reducing admixtures, time of mixing, increasing temperatures, etc. Under certain combinations of unfavorable circumstances up to 80 percent of the total maximum amount of ettringite has been observed to precipitate in the 3-minute mixing period. In Figure 1 are shown amounts of ettringite formed in pastes of a Type K cement paste with and without lignosulfonate and with fly ash during curing at 73° F (23° C) for 1 to 24 hours. The lignosulfonate caused formation of 6 percent ettringite, by weight of paste, during the first hour of curing compared to

* The abbreviations for oxides are, $CaO=C$, $SiO_2=S$, $Al_2O_3=A$, $Fe_2O_3=F$, $SO_3=\bar{S}$, and $H_2O=H$.

4 percent for the other two pastes. Following the initial burst of ettringite formation the reaction slows down. For successful prestressing two conditions must be fulfilled, (1) the ettringite reaction must continue after the paste has attained sufficient strength to accept compressive stress through the restraint without significant deformation, and (2) the reaction must terminate in not more than 10 days, preferably less.

The chemistry of hydration of Mikhailov's(9) cement differs from that of other expansive cements because the monosulfate, due to the selected composition of the cement, forms as an intermediate phase. A representative composition of Mikhailov's cement is 18 percent calcium aluminate cement, 15 percent gypsum and 67 percent portland cement. The relatively large amount of the calcium aluminate cement dominates the early hydration reaction. Apparently the requisite for monosulfate formation, a high concentration of aluminate ions in solution, is fulfilled. Once formed the monosulfate transforms to ettringite relatively slowly. It need be noted that the calcium aluminate cement developed by the Research Laboratories of Universal Atlas Cement Division of United States Steel and designated Type M-X differs fundamentally from the Mikhailov cement. Monosulfate does not form in Type M-X expansive cement and for this reason it behaves generally much like conventional portland cement.

Mikhailov's precast units require a unique curing cycle, 1 day at room temperature, then few to several hours at 70° to 95° C (158° to 203° F), and finally in water at room temperature for 3 to 5 days. It appears that the heating at the elevated temperature stabilizes the monosulfate and prevents further formation of ettringite which is unstable at temperatures above about 80° C (176° F). Simultaneously the hydration of the calcium silicates is accelerated. The gypsum not being able to combine as ettringite reacts with the residual A-bearing phases (if available) and with the hydrating calcium silicates to form C-(S, $\bar{S}$)-H. Upon resumption of curing in room temperature water the C-(S, $\bar{S}$)-H (and gypsum if any remains), reacts with the monosulfate to form ettringite. Formation of ettringite causes expansion whereas formation of the monosulfate does not.

The effect of CH in cement paste on the stability of ettringite requires some clarification. The stability of ettringite relates to the concentrations of Ca^{++} , SO_4^{--} , $(\text{AlO}_2)^{-}$, OH^{-} and excess water, not that of Ca^{++} and OH^{-} alone. Results(10) in Figure 2 show the solubilities of ettringite and monosulfate (expressed as amounts of A) in solutions containing CH in amounts of 0.2 to 1.1 g/l of C, and 0.2 g/l of $\bar{S}$. The solubility, and therefore stability, of ettringite is independent of the SO_4^{--} concentration above about 0.03 g/l of $\bar{S}$ (and possibly much less) over the given range of CH concentration. Reduction of the $\bar{S}$ concentration below about 0.03 g/l causes the ettringite to start to transform to the monosulfate. These results permitted the prediction(10) that ettringite in a cement paste would react with the residual C_3A

to form the monosulfate (or related solid solution). This prediction has been confirmed many times.

The DTA curves in Figure 3 for pastes cured for 7 hours, 1, 4, 7, and 28 days illustrate this transformation for Type M-X and S cements. The monosulfate could not nucleate in the Type K cement paste because this sample of cement was made with a Type V cement(11). Ettringite remains stable and continues to form in cement pastes over a broad range of CH concentration provided the $\bar{S}$ concentration is maintained above about 0.03 g/l and a reactive A-bearing phase is present. In order to attain an equilibrium CH concentration of 0.2 g/l of C in the aqueous phase of a cement paste about 50 percent of the cement C would have to be leached out. It is the exhaustion of the $\bar{S}$ in expansive cements that causes stoppage of the ettringite-formation reaction, not the depletion of C or A. As a consequence of this the A concentration increases and the ettringite starts to transform to the monosulfate.

History

According to Bogue(12) the beginning of the history of expansive cements is occasionally associated with the discovery of ettringite by Candlot in 1890; Michaelis 2 years later suggested that formation of ettringite caused deterioration of concrete exposed to sulfate and sea waters. The idea of controlling the expansive energy of ettringite formation to useful purpose probably occurred to chemists of that time. However, nearly a generation of time passed after discovery of ettringite before researchers at the Lafarge Cement Company in France started experimentation in the 1920's on offsetting drying shrinkage of concrete by deliberate formation of ettringite. Mixtures of portland cement, calcium aluminate cement and gypsum served as cementitious materials in the tests. The tests were unsuccessful. The loss of water causing drying shrinkage also terminates the ettringite reaction. The development of a limited and predetermined amount of expansion to compensate for subsequent drying shrinkage was not practical until the concept of restraining the concrete was developed. These Lafarge experiments are of interest as the beginning of the history of expansive cements, about 50 years ago.

Lossier(13) credited with discovery of expansive cements, started experimentation in 1936. He prepared a clinker by burning ground mixture of bauxite, limestone and gypsum which he believed was an anhydrous calcium sulfoaluminate. Lossier's objective, using this clinker, was chemical prestressing of concrete. Results of laboratory tests appeared promising although difficulty was experienced in controlling expansion. The findings were not reduced to practice and the cement never produced commercially. However, literature mentions the use of Lossier's cement for repairing a masonry bridge arch.

In 1952, Lafuma(14) supplied X-RD evidence showing Lossier's cement to be a mixture of calcium aluminates, anhydrite and gamma C_2S . He proposed the use

of calcium aluminate cement with gypsum, or alternately bauxite or pozzolan and gypsum as expansive agents instead of Lossier's clinker. The discussions of Lafuma's paper in part expressed the feeling that expansive cements held little promise for prestressing concrete. Lafuma held a similar view.

The Soviets conducted extensive research on expansive cements. Mikhailov(15) published results in 1955, and in 1960 (in English)(11). He developed a cement, in which artificial hydrocalumite and gypsum served as expansive agents, which he called waterproofing expansive cement. He also developed a cement already discussed which was called stressing cement. Both products were very fast setting. Tartaric acid and sulfite liquors delayed final set to 15 to 20 minutes but decreased expansion by 30 to 50 percent. The stressing cement apparently has not been used in conventional concrete. Mikhailov described shotcreting and two other processes for manufacturing 1:1 mortar pipe. The extent of commercial application was not clear. Budnikov and coworkers studied a variety of aluminates including burnt clays in developing expansive cements. Budnikov and Kravchenko(16) described the status of the expansive cement industry in the Soviet Union in 1968. Three kinds of expansive cements are mentioned, waterproofing expansive cement, gypsum-alumina cement and expansive portland cements. The differentiation between the latter two is not clear but both apparently are based on calcium aluminate cement. The waterproofing cement is used in the joints of cast sections of city subways and for joining separate precast concrete members into a single unit. The stressing cement(17) is also used for integration of separately cast sections, the protruding reinforcing bars being welded together prior to filling the joint with the expansive mortar. The expansive aluminous cement is reported to be used in sealing porous rock formations in oil and gas wells. It appears the Soviets developments were directed mostly or totally to self-stressing mortars. Mention(16) was not made in 1968 about production of expansive cement pipes. In 1970, ACI Committee 223 reported that self-stressing concretes were still in experimental stages. This observation is in substantial agreement with the Budnikov and Kravchenko summary statement: "It is necessary to state however that despite the broad promise of expansive cements as a building material their output and their use has not yet attained the volume necessary."

Alexander Klein at the University of California, Berkeley, considered that an anhydrous calcium sulfoaluminate might offer a better way for controlling the ettringite-formation reaction than did the other A-bearing expansive agents. He prepared such compound which X-RD results showed to be a new compound. Fukuda(18) reported the exact composition, C_4A_3S , for this compound in 1961. Klein's first report with coauthor was published in 1958(19). An extensive research and development program on expansive cements was undertaken at the University of California by Klein and associates, Bertero, Bresler, Lin, Mehta, Pirtz, Polivka, and others. The results are described in many publications reviewed in several reports mentioned in the introduction to this report. Klein's first invention has the priority date of October 18, 1961.

Patents in several countries were granted on this invention in 1964. Another patent was granted to Klein, as a continuation-in-part of the first, on May 17, 1966.

Following the successful laboratory experimentation, field trials of the two expansive cements, shrinkage-compensating and self-stressing, already defined, were undertaken. For this purpose and for later promotion, a group consisting of inventor, engineers, producers, contractors and public relation personnel was incorporated under the name of Chemically Prestressed Concrete Corporation. Patent rights were assigned on the Klein cement to the new corporation which since has been granting licenses to producers.

Fields trials conducted in 1963 and later are described in several review reports. Three of these applications are milestones in the history of expansive cement concrete and are as follows.

A roof of shrinkage-compensating concrete, 43 by 100 feet in size, was placed on a Yuba City Savings and Loan building in March 1963. This was the first field application of shrinkage-compensating cement. Only minor cracks appeared and leakage occurred in only one spot due to concrete segregation. Waterproofing coating was not applied. This was a successful application.

Two experimental sections of concrete highway near Palmdale and Lodi, California, were installed in 1963, using shrinkage-compensating concrete without steel reinforcement. The subgrade friction and end anchors were expected to supply needed restraint. The pavements cracked due to drying shrinkage apparently because of insufficient restraint.

A prototype road slab in Connecticut, 185 by 13 feet in size, was cast without expansion joints using self-stressing concrete and 0.23 percent longitudinal steel and 0.53 percent transverse steel. Prestress based on expansion was 150 psi. The slab remained crack free. This trial was followed by installation of three joint-free 500- by 24-foot sections of State Highway No. 2 near Glastonbury, Connecticut, using self-stressing concrete. Difficulties developed, and the concrete from standpoint of ultimate objectives was a failure. The result, however, was productive of useful information including possible limitations on lengths of joint-free pavements which would be less than 500 feet.

Since 1963 application of Klein's shrinkage-compensating cements increased rapidly and development of different expansive cements was undertaken by others. Monfore(20) described laboratory tests and results on expansive cements made with portland cement, calcium aluminate cement and gypsum. His compositions and procedures tended to be like those of Mikhailov, especially the curing cycle. A new shrinkage-compensating cement based on calcium aluminate cement was developed in the Research Laboratories of Universal Atlas Cement Division United States Steel Corporation in the mid-1960's. This

cement differs distinctly from the Soviet calcium aluminate cement expansive cement and was marketed in 1970 as Type M-X expansive cement by its producer. Type M-X cement is used in conventional concrete, placed by conventional methods and curable at ordinary temperatures. None of these three factors applies to the Soviet Type M cement.

Greening of the Portland Cement Association developed the Type S cement which is the same as Type I except for a higher C_3A content and higher $\bar{S}$ content. This development is described in part in a report by Gustaferrero, Greening and Klieger(21). This cement was first marketed in 1968 and is now produced by two companies.

The production of expansive cements including Types K, S, and M-X is shown from 1963 to extrapolated value for 1972 in Figure 4.

The Japanese are applying shrinkage-compensating concretes in a variety of structures. For structural concrete 10 to 11 percent of an expansive agent containing $C_4A_3\bar{S}$ is added as an admixture to the concrete mixer. For self-stressing concrete products, higher percent of the admixtures is used. The growth of application of expansive admixtures, expressed as equivalent amounts of cement is included in Figure 4.

Until about 1970 self-stressing concretes were still in experimental stages as already noted. In 1970, Benuska, Bertero, and Polivka(22) described in considerable detail a self-stressing concrete for fabricating precast building modules, 12 by 36 by 8 feet in size with wall thickness of 2 inches (3.7 by 11 by 2.4 meters and wall thickness of 10 cm). Prestress of close to 300 psi was attained after drying shrinkage and creep were stabilized. The authors concluded, "It is possible to fabricate structurally sound and safe self-stressed concrete slabs and walls." Experimental testing was done using Type K and M-X expansive cements.

The first factory producing the mentioned building modules went into production early in 1971. It has been reported that production is proceeding satisfactorily and that the new product is competitive with other building materials.

It may be concluded that after many years of research and development self-stressing concrete is finally beginning to fulfill its promise.

Properties

The properties of fresh shrinkage-compensating concrete frequently differ to various degrees from those of portland cement concrete.

The water requirement for equal slump at time of placement generally is higher for the shrinkage-compensating cement, but workability at equal slump is about the same for both. To obtain the desired slump more water is added to the mix. Prediction of field performance of concrete from laboratory results may frequently be misleading. Concrete in the field may be in the mixer up to an hour or more before being placed; laboratory specimens are cast 10 to 15 minutes after mixing water is added. Shrinkage-compensating concretes show negligible bleeding. This calls for early wetting of the surface of the concrete to prevent plastic shrinkage cracking. The time of set of shrinkage-compensating concrete is about 30 to 60 minutes shorter than of portland cement concrete. Set-retarding admixtures prolong setting time but infrequently cause reduced expansion.

Differences in properties of concretes made with the three types of cement may occur. Also the properties of any one type of the cements may differ depending on the storage history of the cement at the time it is used.

Expansion of the concrete is the most critical property to control and one subject to several variables. The expansiveness of each type of cement is predetermined in the laboratory and the composition established for desired expansion. Expansiveness is retained if the cement is protected against aeration and other precautions taken. Aeration deterioration is a subtle occurrence with unpredictable results. Beside aeration deterioration, expansiveness is generally decreased by increased fineness of cement, high water factor, decreased amount of cement, increased size of specimen, prolonged mixing, high temperature of fresh concrete, frequently by use of admixtures, always by inadequate curing and other factors. Increasing the amount of restraint decreases amount of expansion. Such increase, per se, is desirable provided the use of a greater amount of restraint is consistent with design and costs. However, the amount of expansion must be larger than the amount of ultimate shrinkage in order to insure that drying-shrinkage cracking does not occur.

The properties of hardened shrinkage-compensating and portland cement concretes tend to be the same, making allowance for effects during the expansion period. Shrinkage-compensating concrete exhibits lower strength and modulus of elasticity during expansion, thereafter these properties approach those of portland cement concrete. Drying shrinkage after expansion is about the same for both concretes. Bond strength is lower for uniaxially restrained, but higher for biaxially and triaxially restrained expansive concrete compared to portland cement concrete. Poisson's ratio and coefficient of thermal expansion are about the same for both concretes.

Resistance to freezing and thawing at comparable entrained-air contents is about the same for both concretes. Shrinkage-compensating concrete generally shows slightly better resistance to deicer scaling. Sulfate resistance of Type S and M-X concretes would be expected to be relatively poor as that

of a Type I cement concrete due to high A and low F contents. The resistance of Type K cement to sulfate attack would vary from susceptible to resistant depending on type of portland cement used (Type I, II or V). Shrinkage-compensating concretes show excellent resistance to alternate wetting and drying.

The properties of hardened self-stressing concretes tend to be generally comparable to those of shrinkage-compensating cements, with two exceptions, and making allowance for degree of restraint. The properties of fresh shrinkage-compensating concrete differing from those of portland cement concrete, are accentuated to same degree by the increased amount of expansive agent in self-stressing concrete. The degree of restraint influences some properties of hardened concrete. Thus compressive strength is directly related to degree of restraint and therefore inversely to amount of expansion. The successful commercial application of self-stressing concretes will be productive of new data. A further review of results available in Reference 7 and other papers does not appear necessary at this time.

Attention is directed instead to the two excepted properties. Deicer resistance of self-stressing concrete is poor, and freeze-thaw deterioration relates to amount of restraint (21) and therefore to expansion prior to freeze-thaw tests. Unrestrained concrete deteriorates relatively rapidly and resistance improves with increasing amount of restraint. At 1.7 percent steel, the most highly self-stressed concrete (about 500 to 600 psi) met the maximum limit of 0.1 percent expansion after 300 freeze-thaw cycles.

At some degree of prestress and using nominal amount of restraint the freeze-thaw durability of self-stressing concrete (at self-stress perhaps above 300 psi) would be questionable. It was postulated that such susceptibility might be due to formation of a network of interconnecting microcracks permitting ingress of water or solution of the deicer salt. Previous determinations on relating volume of ettringite formed and the increase in volume of the pastes gave two answers. In shrinkage-compensating cement pastes the volume of ettringite formed was several times greater than the increase in volume of paste. In self-stressing cement pastes the reverse was true, the increase in volume of paste was much larger than the volume of ettringite formed. The above and other consideration on shrinkage-compensating cement paste is attributed to precipitation of large portion of ettringite in the voids of the cement paste. The large volume increase in the self-stressing paste is attributed to formation of voids by cracking.

An experiment was conducted to supply information on the above supposition on the self-stressing cement. Unrestrained paste and mortar bars, 1 by 1 by 11-1/4 inches (2.5 by 2.5 by 29 cm) in size, were made, stripped at 20 hours, measured for initial length, stored in water at 73° F (23° C) and measured periodically. Paste and mortar bars, 3 by 3 by 10 inches (7.6 by 7.6 by 25.4 cm) in size, were made with longitudinal restraint, a 3/16-inch

(0.48-cm) diameter threaded steel rod. Gage reference points were also located at midspan for measurement of lateral expansion and the specimens processed the same as the unrestrained bars. The results are compiled in Figure 5.

The unrestrained paste expanded 3.1 percent which also was the lateral mid-span expansion of the restrained paste. The longitudinally restrained paste expanded 0.31 percent. The longitudinal expansion of the unrestrained mortar was 1.3 percent and, of the restrained mortar, 0.32 percent. The lateral expansion at midspan of the restrained mortar was 0.96 percent substantially less than the free expansion, 1.3 percent. This indicates that the sand grains cause a partial restraining effect even at distance of 5 inches from the point of restraint. Paste specimens were used to determine the volume of ettringite, and this was compared to the volume increase of the paste. The 3.1 percent free expansion is equivalent to 9.6 percent volume increase. DTA data showed the formation of 5.5 percent ettringite, by weight of paste, or 3.1 percent, by volume, during the period volume increased 9.6 percent. The 6.5 percent difference between increase in volume of paste and volume of ettringite is considered to be due to formation of interconnecting network of cracks.

It is proposed that increasing restraint of self-stressing concrete has the effect of decreasing internal cracking. The precipitation of ettringite is forced into the voids of the paste and contributes to reducing permeability. The expansive force of formation of ettringite from $C_4A_3\bar{S}$ was determined by Mehta and Klein(23) in a unique experiment to be about 35,000 psi. Computations(24) using thermodynamic data indicated that a restraining force of 48,000 psi would be required to stop formation of ettringite from C_3A . The improvements in resistance to freeze-thaw cycles and sulfate attack may be attributed to decrease in permeability by the deposition of ettringite in the voids of the paste.

These results serve only to indicate that more research is needed regarding use of self-stressing concretes subjected to cycles of freezing and thawing. Although test results are still too few to permit conclusions it appears probable that concrete with prestress in the range of 250 to 350 psi using nominal amount of restraint and proper air entraining should show satisfactory resistance.

Conclusions

This report reviewed the developments of expansive cements which culminated in commercial use of these materials as competitive building materials. In 1952, expansive cements, specifically self-stressing, were regarded as holding some promise but skepticism prevailed over optimism. A few years later

Alexander Klein discovered an anhydrous calcium aluminate sulfate and proceeded, with associates at the University of California, Berkeley, to develop two kinds of expansive concretes, shrinkage-compensating and self-stressing. Field trials in 1963 on the shrinkage-compensating concrete showed immediate success. The production of this cement has amounted to an estimated 2,000,000 barrels (380,000 tons) in 1972.

In retrospect, it is evident that Klein's contribution to concrete technology is monumental. It is not alone the engineering and chemical accomplishments that are a measure of his, and his associates', achievements. Equally important was the surmounting of the skepticism of potential users and the establishment of a market for the shrinkage-compensating cements against heavy odds. Further, self-stressing concrete stemming from the University of California investigations appears to be finally fulfilling its promise with first commercial production in 1971.

Production data in Japan during the past 3 to 4 years shows an accelerating growth of expansive concretes and products.

The Soviets contributions have culminated in several commercial applications of expansive cements, principally in precast products and special uses.

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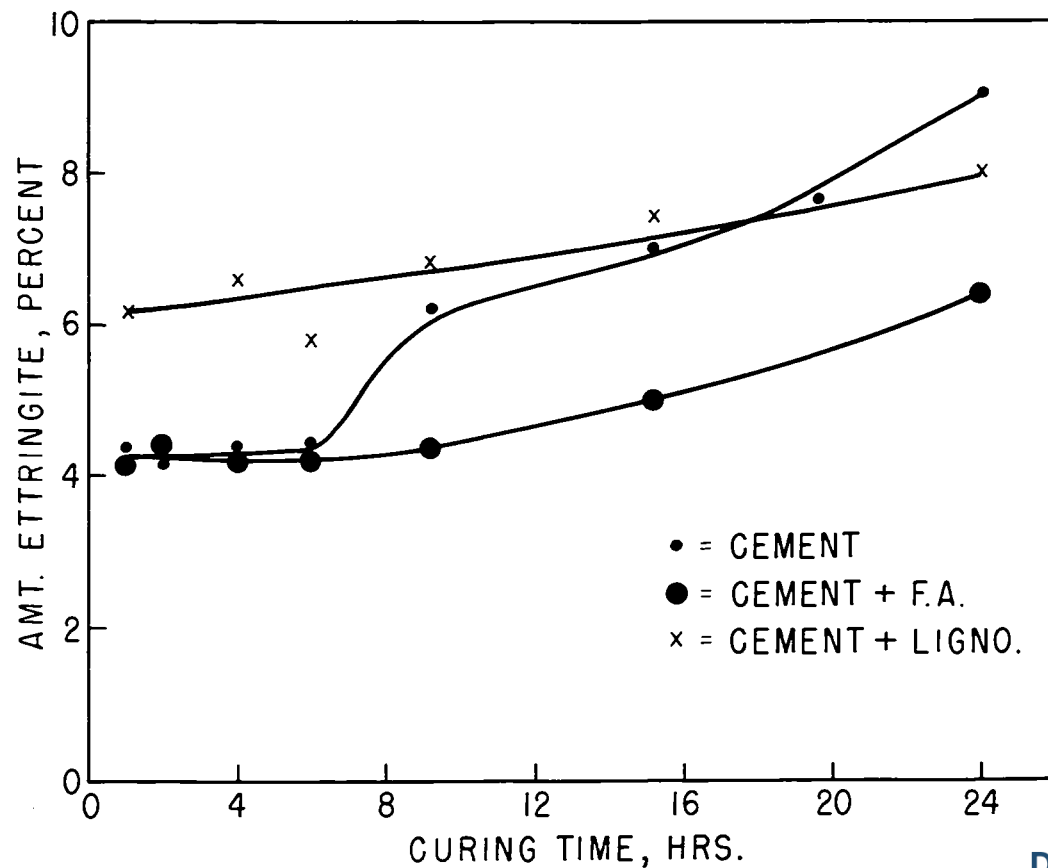


Fig. 1--Amounts of ettringite formed in Type K cement pastes cured at 73F (23C) for 1 to 24 hours

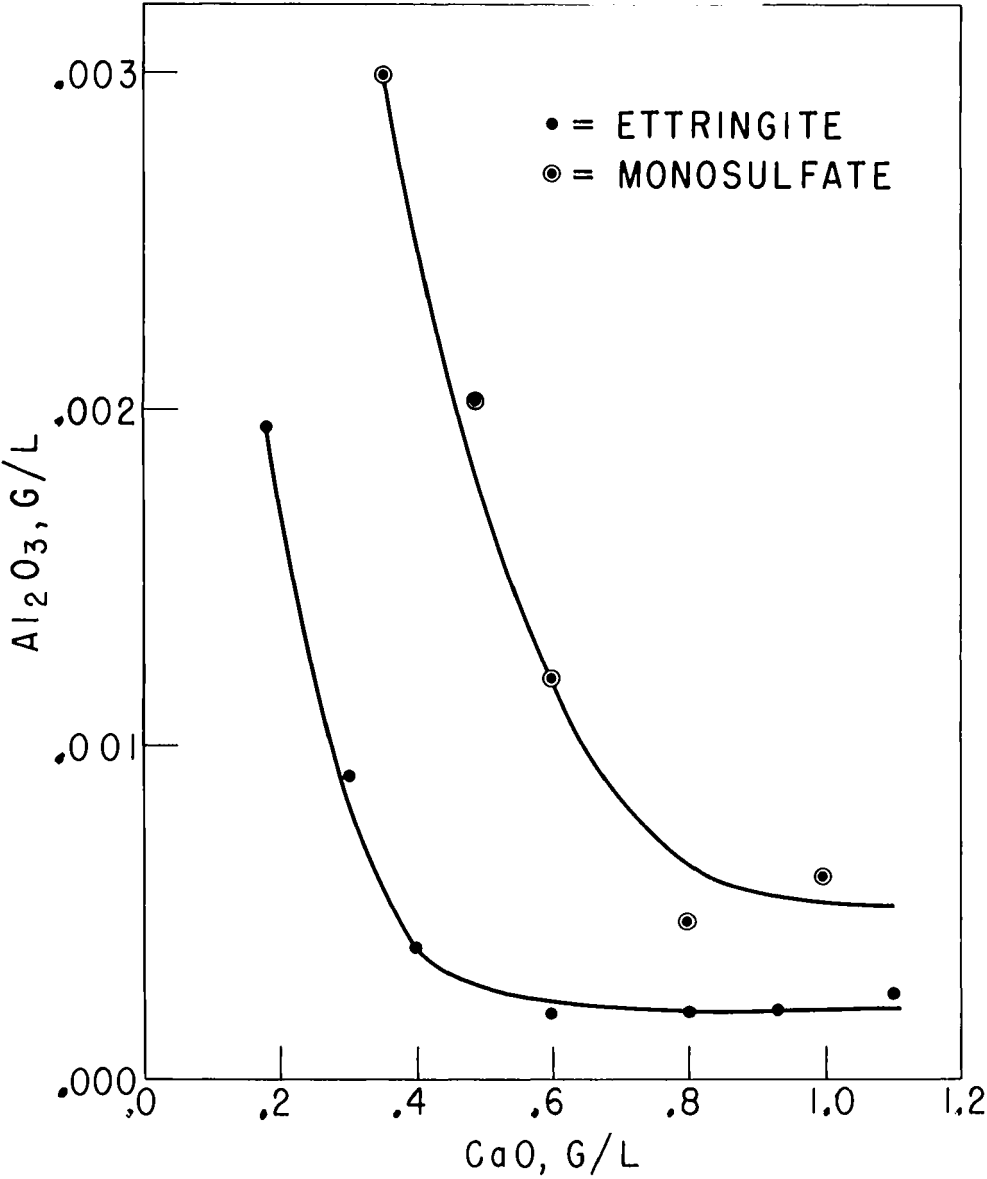


Fig. 2--Solubilities of ettringite and monosulfate in CH solutions containing 0.2 g/l of $\bar{S}$

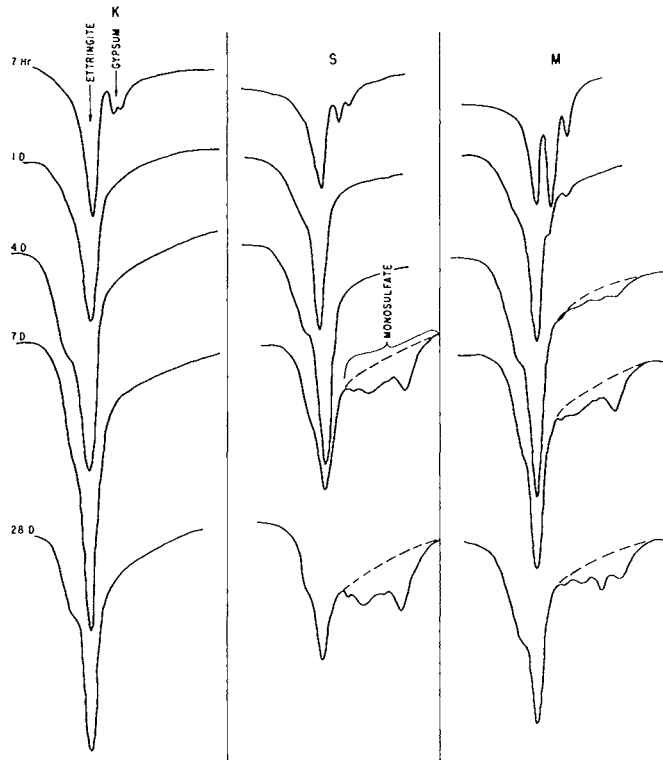


Fig. 3--DTA curves showing combination of gypsum to form ettringite and transformation of ettringite to monosulfate

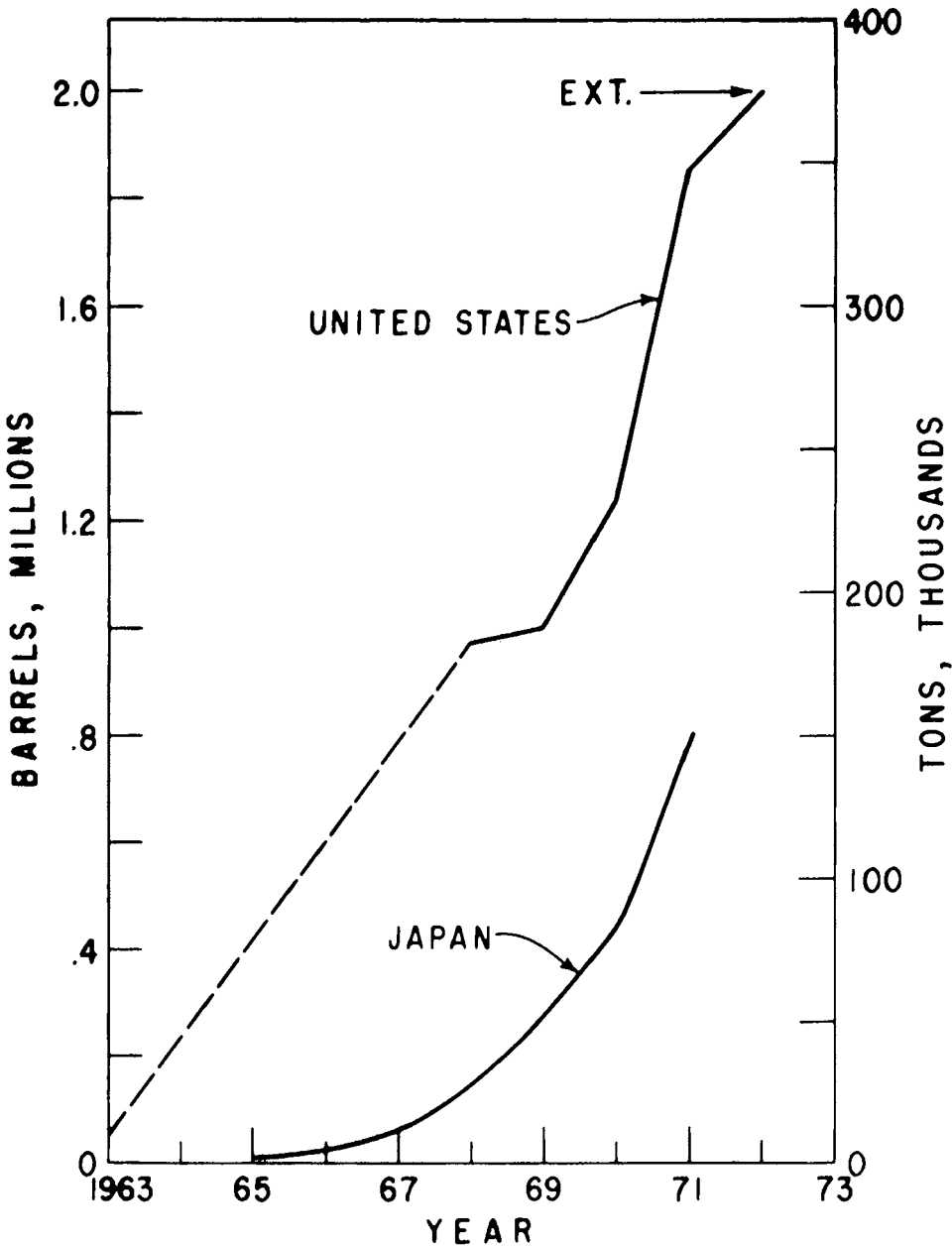


Fig. 4---Production data on expansive cements in Japan and the United States

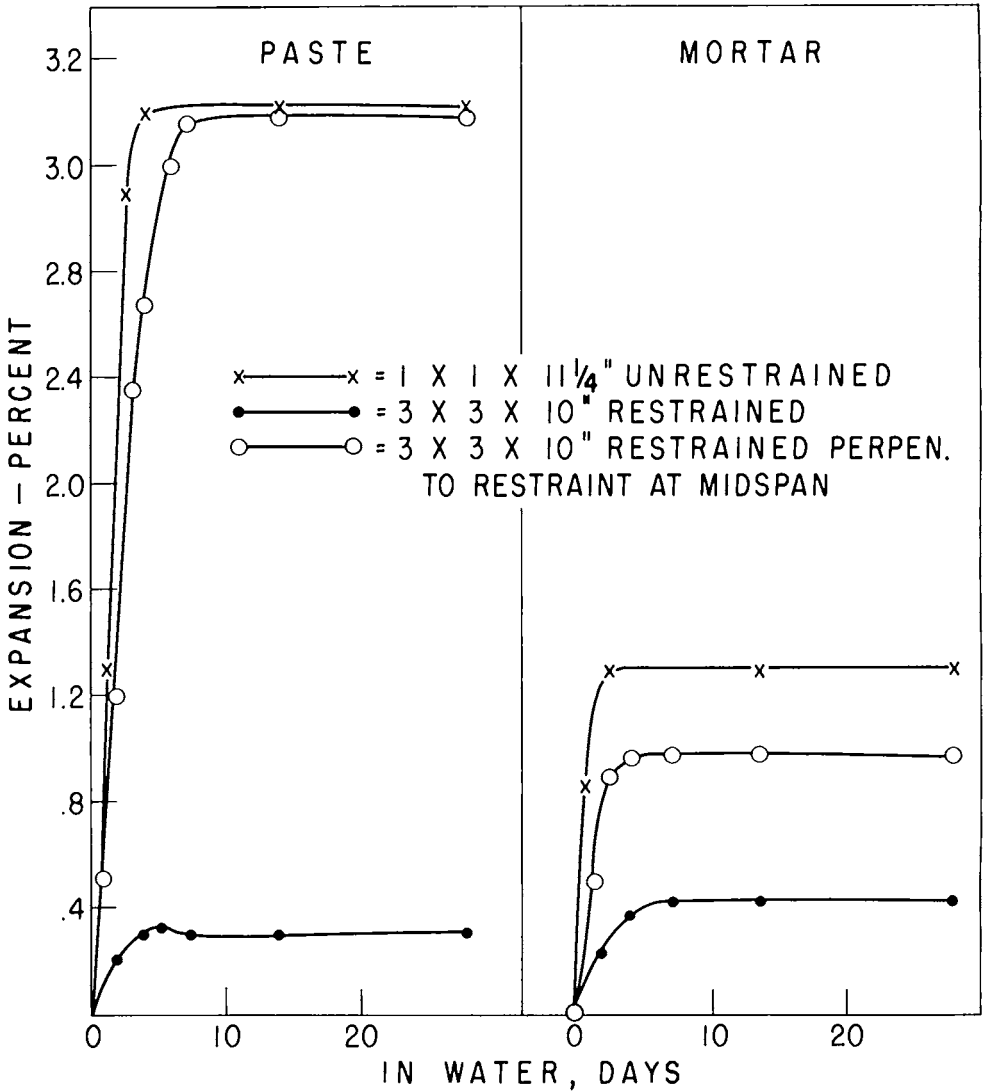


Fig. 5--Longitudinal and lateral expansions of restrained and unrestrained pastes and mortars

SP 38—2

THIN SHELL EXPANSIVE CEMENT CONCRETES

BY
JOHN R. KEETON

Synopsis: Expansion and subsequent shrinkage characteristics of shrinkage-compensating concrete were determined in mesh-reinforced prisms as well as in modified ASTM C-490 bars. The maximum aggregate size was 3/8 in. (0.95cm). The specimens were cured in fog for 14 days and then subjected to drying in either 25% or 50% relative humidity at 73°F (23°C). Mixes were made with several different cement contents, but most were made with 705 lb of cement per cu yd (418 kg/m³). For comparison purposes, similar mixes were made with portland cement. The specimens with shrinkage-compensating concrete showed 20% less shrinkage strains at 50% RH than those of portland cement concrete. The effects of the amount of restraints on expansion and shrinkage were also determined. Compressive strengths and Young's moduli of shrinkage-compensating concrete compared favorably or exceeded similar values for portland cement concrete.

Keywords: compressive strength; concretes; creep properties; curing; expansive cement type K; expansive cements; mix proportioning; portland cements; reinforcing steels; research; shells (structural forms); shrinkage; shrinkage-compensating concretes; water-cement ratio.

ACI member John R. Keeton has been closely associated with research in concrete for a number of years with the Naval Civil Engineering Laboratory. He has authored several reports and publications on concrete creep and shrinkage and is currently Chairman of ACI Committee 209. He is also a member of ACI Committees 223 and 548.

INTRODUCTION

The development of expansive cements and expansive concretes has been described in considerable detail.^{1,2,3,4,5} The terms used in this paper conform to the definitions presented by ACI Committee 223.⁴

The motivating stimulus behind development of expansive cements was twofold: (1) elimination of shrinkage cracking, and (2) inducement of relatively high levels of precompression in the concrete, much as is done by mechanical prestressing. The twofold objective resulted in development of (1) a shrinkage-compensating expansive cement and (2) a self-stressing expansive cement. The principal difference between the two classes of expansive cements is the amount of expansive component provided for reaction with portland cement ingredients to form ettringite, the expansion producer.

The current research program at the Naval Civil Engineering Laboratory (NCEL) is designed to determine the efficacy of expansive cement concretes for Naval construction and, if the product is worthwhile and cost-effective, to enable specification, design, and construction of thin-shell and ferrocement structures.

RESEARCH PROGRAM

Test Specimens

In order to obtain test data comparable to those of other investigators, a series of length change tests was made on prismatic specimens nominally 3 in. (7.6 cm) square and 10 in. (25.4 cm) long, cast in molds conforming to ASTM C490-70. For restrained specimens, the molds were modified to take a threaded mild steel rod 3/16 in. (0.48 cm) in diameter (nominal) along the central longitudinal axis. A mold with threaded bar in-place is shown in Figure 1. In this paper concrete specimens made in the modified ASTM C-490 molds are referred to as "bars".

To simulate concrete thin shells, prismatic test specimens 5 in. (12.7 cm) wide and 12 in. (30.5 cm) long were made in thicknesses of 1 in. (2.5 cm), 2 in. (5.1 cm), and 4 in. (10.2 cm). An example of each prism size is shown in Figure 2. The prisms were reinforced with galvanized welded wire fabric of the following spacing and wire sizes: 2 in. by 2 in. (5.1 cm), No. 14 and No. 12 and 1 in. by 1 in. (2.5 cm by 2.5 cm), No. 14 and No. 12. Examples of placement of the fabric with respect to the forms is shown in Figure 3, Figure 4 and Figure 5.

Several different percentages of reinforcement were obtainable, as indicated in Table 1. Computation of reinforcement percentage was based on the total cross-sectional area of the horizontal wires viewed from either end. The vertical wires completing the fabric were not considered in computing the steel percentage. In this report, reinforcement is designated by a lower case p followed by a percentage, e.g. $p=0.19\%$.

Also visible in Figure 3, Figure 4, and Figure 5, are the screws which serve as reference points for measurement of length changes with a mechanical strain gage. The reference screws are 5 in. (12.7 cm) apart. To date, all expansive cement concrete has been made with Santa Clara River gravel and sand having a maximum size of 3/8 in. (0.96 cm).

Expansive Cement Concrete Mixes

Research thus far has been on shrinkage-compensating Type K expansive cement. Mixes were made over a wide range of cement contents (590 to 1,330 lb/yd³ - 350 to 789 kg/m³) and water-cement ratios (0.376 to 0.596 by weight), both with and without entrained air. For comparison purposes, a few mixes were made with Type II portland cement. In this report the concrete mixes are designated by the number of equivalent sacks per cubic yard followed by the type of cement used, e.g. 7.5 SCA means 7.5 sacks per cu yd, shrinkage-compensating cement, air entrained. Mixes of portland cement concrete are designated in the same manner, e.g. 7.5 PC (7.5 sacks per cu yd, portland cement).

Concrete Mixing Procedure

To simulate the average mixing and hauling time of truck mixers, the following mixing procedure was adopted:

1. Mix aggregate, cement, and 3/4 of water for 3 minutes
2. Stop the mixer for 5 minutes (total time 8 minutes)
3. Mix for 2 minutes (total time 10 minutes)
4. Stop the mixer for 5 minutes (total time 15 minutes)
5. Mix for 2 minutes (total time 17 minutes)
6. Stop the mixer for 5 minutes (total time 22 minutes)
7. Mix for 2 minutes (total time 24 minutes)
8. Stop the mixer for 5 minutes (total time 29 minutes)
9. Add the remaining mixing water
10. Mix for 2 minutes (total time 31 minutes)

11. Measure the slump (nominal 4 in. (10.2 cm), adjust if necessary) and measure air content
12. Cast the specimens, consolidating by vibration

Curing of Test Specimens

Most of the test specimens were cured in fog for 14 days prior to being placed in one of the two controlled temperature and humidity environments (25% RH or 50% RH at 73°F). This period of 14 days was found to be typical for job-curing in the Los Angeles area. For comparison purposes, a few specimens were fog-cured for 28 days prior to being transferred to drying environments.

Length Change and Strain Measurements

All measurements of expansion on the test specimens began about six hours after casting, immediately after removal from the molds. The specimens were then placed in fog and measurements were made daily for 1 week, then weekly for about 3 months, and monthly after about 90 days. Length changes in the modified C-490 bars were measured with a comparator, and strains in the prisms were measured with a mechanical strain gage.

Test Results

Compressive strengths -- Results of compressive strength tests on unrestrained cylinders 3 in. (7.6 cm) in diameter and 6 in. (15.2 cm) long are presented in Figure 6 as a function of cement content and in Figure 7 as a function of water-cement ratio. In both Figure 6 and Figure 7 the straight lines only show trends and do not represent approximate least square lines. It can be seen in Figure 6 that there are no significant differences, for a given cement content, in the strength of shrinkage-compensating portland cement concrete with or without air. As indicated in Figure 7, however, the strengths of the shrinkage-compensating concretes tend to be a little higher at water-cement ratios below about 0.50. Perhaps the most significant fact is that although the shrinkage-compensating concrete expands while curing and therefore should theoretically have a lower strength, the test results indicate that it is at least as strong as ordinary portland cement concrete. As expected, both air-entrained concretes (shrinkage-compensating and portland cement) show lower strengths than those without air. Table 2 shows compressive strengths of restrained 1 in. (2.5 cm) by 5 in. (12.7 cm) prisms ($p=0.30\%$) after 14 and 28 days of fog curing. Although the compressive strengths show sizeable increases for the 28-day values, the Young's moduli increased very little over the 14-day value. Reasons for this may include the effects of the expansion occurring between 14 and 28 days upon the stiffness, of which Young's modulus is a measure.

Expansion and shrinkage - modified C-490 bars -- Expansion and shrinkage of bars made with 7.5 SC and with $p=0.19\%$ are presented in Figure 9. Shrinkage in 25% RH was ultimately about 12% greater than in 50% RH. Negative strain in 25% RH was about 40% greater than in

50% RH. It should be mentioned that had the bars been shrinking in a higher humidity, such as 75% RH, the shrinkage curve might have been closer to the zero line. Expansion and shrinkage are reported as "microstrain", which is microinches per inch.

Similar curves for 7.5 SCA are presented in Figure 10. Shrinkage in 25% RH was ultimately about 14% more than in 50% RH. Negative strain in 25% RH was about 52% more than in 50% RH. A comparison of 7.5 SC and 7.5 SCA curves is shown in Figure 11 for bars shrinking in 50% RH. The 14-day expansions were almost equal for 7.5 SC and 7.5 SCA. At 147 days shrinkage of the 7.5 SCA bars was about 6% higher than the 7.5 SC bars. Considering the durability advantages of the air-entrained concrete, the slight increase in shrinkage may be inconsequential. Negative strain in 7.5 SCA was about 17% more than in 7.5 SC.

The effects of different amounts of restraint are shown in Figure 12 for 7.5 SC bars in 50% RH. There seems to be a delicate relationship between amount of restraint -- in this case the diameter of a threaded rod -- and the expansion and subsequent shrinkage. One would expect, since it is the steel that provides the restraint which in turn induces the compression into the concrete, that the more steel the better. As the steel percentage increases, the amount of initial expansion decreases, and the amount of precompression in the concrete increases. An optimum amount of reinforcing probably exists for each combination of concrete mix, configuration of structure, and subsequent drying environment. Figure 12 indicates, as far as the data have progressed, that the three curves may approach each other in time, in which case the $p=0.19\%$ would seem to be the most economical since it requires the least steel. The author believes however, that the amount of precompression induced in the concrete may be of primary importance and that relationships as shown in Figure 12 should emphasize the concrete stress rather than the strain. As expected, shrinkage for $p=0.19\%$ was 14% higher than for $p=0.36\%$ and 25% higher than for $p=0.58\%$.

The effects of the cement content upon expansion and shrinkage of bars with $p=0.19\%$ are shown in Figure 13. As stated earlier, the amount of expansion during the curing period is directly proportional to the amount of shrinkage-compensating cement in the mix, other things being equal. Figure 13 verifies this statement and the subsequent shrinkage results reflect the effects of the higher expansions. The shrinkage strains were in direct proportion to the cement content i.e., highest in 11.0 SC and least in 7.5 SC.

Expansion and shrinkage - prisms -- Expansion and shrinkage of 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms of 7.5 SC with $p=0.30\%$ are shown in Figure 14. At 175 days shrinkage in 25% RH was about 25% higher than in 50% RH. Ultimate negative strain in 25% RH was about 95% higher than in 50% RH. Comparing Figure 14 with Figure 9 it is interesting to note that ultimate negative strains in 25% RH were not much different, but there were some basic differences in the specimens and reinforcement which enter the picture. The 3 in. (7.6 cm) square bars were reinforced with a threaded rod at $p=0.19\%$ while the 1 in. by

5 in. (2.5 cm by 12.7 cm) were reinforced with 2 in. by 2 in. (5.1 cm by 5.1 cm) no. 14 mesh (see Table 1). It should also be recalled that the 1 in. by 12 in. (2.5 cm by 30.5 cm) edges of the prisms were sealed to prevent loss of moisture, to simulate a continuous thin shell 1 in. (2.5 cm) thick; drying was permitted only from the 5 in. by 12 in. (12.7 cm by 20.5 cm) faces. Drying was permitted from all four sides of the bars. It has been well established previously that higher shrinkage can be expected from smaller (and thinner) specimens.^{6,7} The most significant factor in comparing Figure 9 and Figure 14 is the difference in reinforcement (0.19% versus 0.30% and the as yet unclear effects of the configuration of the reinforcement (rod versus mesh). The 14-day expansion of the bars was slightly higher than that in the prisms.

Expansion and shrinkage curves for 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms of 7.5 SCA are presented in Figure 15. The relationships between curves for 25% RH and 50% RH are quite similar to those already presented. At 134 days, shrinkage in 25% RH was about 16% higher than in 50% RH, while negative strain was about 62% higher in 25% RH.

Comparison curves for 7.5 SC and 7.5 SCA in 50% RH are shown in Figure 16, which shows about the same relationship between the two curves as seen in Figure 11 for bars. In Figure 16, however, the 14-day expansion of the 7.5 SCA was shown to be about 20 percent higher than the 7.5 SC. Having entrained air bubbles, and therefore lower strength, the hardening 7.5 SCA offered lower resistance to the initial expansive forces during the curing period. In spite of the higher initial expansion, however, the strain curve for 7.5 SCA dropped below that for 7.5 SC, and the inherently weaker air-entrained concrete shrank 22% more in the same period (134 days) than did the 7.5 SC. Negative strain in the 7.5 SCA, however, was 26% higher than in the 7.5 SC.

Expansion and shrinkage curves for 2 in. by 5 in. (5.1 cm by 12.7 cm) prisms of 7.5 SC with $p=0.30\%$ are shown in Figure 17. At 119 days, shrinkage in 25% RH was about 30% higher than in 50% RH. This difference between the two curves is slightly greater than the corresponding differences between 1 in. by 5 in. (2.5 cm by 12.7 cm) from Figure 14. In the same period, negative strain in 25% RH was 129% higher than in 50% RH.

Expansion and shrinkage curves for 4 in. by 5 in. (10.2 cm by 12.7 cm) prisms of 7.5 SC with $p=0.30\%$ are presented in Figure 18. Shrinkage in 25% RH at 119 days was about 14% higher than in 50% RH. Although the curve for 25% RH dropped slightly below the zero line, it appears that the 50% RH line might approach zero asymptotically or, at most, drop below very slightly in time. These curves in Figure 18 emphasize the size or mass effects of the larger 4 in. (10.2 cm) thick prisms compared with the smaller ones. The quantitative effects are only now becoming apparent, but continued research should point the way to rational design of thin-shell structures reinforced with mesh rather than with typical reinforcing bars. The mesh does provide a certain restraint in a second direction; that is, a direction perpendicular to but in the same plane as the longitudinal reinforcement.

Effects of size (thickness) of prisms are shown in Figure 19 for 7.5 SC concrete, $p=0.30\%$, and shrinkage in 50% RH. Ultimately, there was very little difference in the shrinkage of 1 in. (2.5 cm) and 2 in. (5.1 cm) prisms, but a dramatic difference between them and the 4 in. (10.2 cm) prisms. It is further interesting to note that the 14-day expansion of the 4 in. (10.2 cm) prisms exceeded that of the others by about 12%.

The effects of the amount of restraint on the expansion and shrinkage of 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms of 7.5 SC are shown in Figure 20, with shrinkage being in 50% RH. As indicated in Figure 12 for bars, the higher the percentage of reinforcement, the lower the initial expansion. Subsequent shrinkage values in 50% RH show questionable value of reinforcement higher than 0.30% and inject the question as to what the performance would be in a 1 in. (2.5 cm) prism reinforced with mesh at a value somewhere between $p=0\%$ and $p=0.30\%$. This will be investigated in future research. As expected, the curve for $p=0\%$ showed the highest expansion, and the shrinkage curve stayed above the zero line, but it should be emphasized that expansion of this concrete was not resisted and therefore the concrete was in a state of tension from the beginning. In addition, the strength properties of this concrete were relatively low due to the large expansions. Tests at $p=0\%$ are usually conducted to evaluate the expansion potential of a given cement. Again it should be emphasized that as the percentage of steel increases, the precompression in the concrete increases and the optimum level of this precompression may be of primary importance, particularly in a structure.

A summary of the effects of different amounts of restraint on 14-day expansions of 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms is presented in Figure 21. It should be reemphasized, however, that there appears to be an optimum amount of a given type of restraint for each concrete mix, interdependent upon the drying environment the concrete is expected to undergo and upon the configuration of the member.

Examples of the possible benefits of expansive concretes are indicated in Figure 22 and in Figure 23. Figure 22 shows expansion and shrinkage curves for 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms of 7.5 SC and 7.5 PC, with both subjected to drying in 25% RH. The reduction in ultimate values of negative strain, and therefore in tensile stress, in the 7.5 SC concrete is quite dramatic. After 91 days, shrinkage in 7.5 SC was about 8% less than in 7.5 PC. Similar results are shown in Figure 23 for prisms shrinking in 50% RH. After 91 days, shrinkage in 7.5 SC was about 25% less than in 7.5 PC.

Creep tests -- Results of tests in which 2 in. (5.1 cm) prisms were loaded in 50% RH at $0.25f'_c$ (25% of their ultimate strength) and at $0.50f'_c$ are shown in Figure 24. For comparison purposes, corresponding curves are also shown for tests made in connection with thin shell roofs reported in Reference 7. Although the cement content of the portland cement mix (775 lb per yd^3 - 460 kg per m^3) was higher than that of the shrinkage-compensating mix (705 lb per yd^3 - 418 kg per m^3), it is unlikely that this difference can account for the rather large variations in negative creep-plus-shrinkage strains

observed between the two mixes. Since the prism size, type and percentage of reinforcement, fog curing time (14 days), and shrinkage environment were identical, one can only conclude that creep-plus-shrinkage of shrinkage-compensating concrete at 147 days is about 13% less at $0.25f'_c$ and about 8% less at $0.50f'_c$ than creep-plus-shrinkage for portland cement concrete. The important observation, however, is that the negative strain at $0.25f'_c$ was 43% lower for the expansive concrete than the portland cement concrete; the corresponding reduction at $0.50f'_c$ was 30%.

SUMMARY

Compressive strengths of unrestrained concrete made with shrinkage compensating cement were equal to or greater than strengths of similar portland cement concrete. Young's moduli of restrained prisms were about 12% higher than in similar unrestrained prisms.

In restrained bars and prisms of shrinkage-compensating concrete, shrinkage in 25% RH ranged from 12% to 30% more than in 50% RH while corresponding negative strain values ranged from 40% to 129% higher in 25% RH than in 50% RH. Shrinkage of air-entrained shrinkage-compensating concrete in 50% RH ranged from 6% to 22% higher than similar non air-entrained concrete; corresponding negative strain values ranged from 17% to 26% higher in air-entrained than non air-entrained concrete.

Shrinkage curves for bars with different amounts of restraint (threaded rods) seemed to approach each other in the negative strain area. On the other hand, similar curves for mesh-reinforced prisms showed some divergence and seemed to indicate that an optimum restraint might exist for a given combination of cement content, type of restraint, amount of shrinkage expected (drying environment), and configuration of member.

Expansion-shrinkage curves for prisms with thicknesses of 1 in. (2.5 cm), 2 in. (5.1 cm) and 4 in. (10.2 cm) showed definite effects of size and mesh configuration. Over a period of 119 days, shrinkage values for prisms 1 in. (2.5 cm) and 2 in. (5.1 cm) thick were well into the zone of negative strain.

Prisms 2 in. (5.1 cm) thick, mesh-reinforced at $p=0.30\%$, were subjected to constant sustained compressive loads at $0.25f'_c$ and $0.50f'_c$ and results indicated that creep-plus-shrinkage of shrinkage-compensating concrete was 13% less at $0.25f'_c$ and 8% less at $0.50f'_c$ than similar concrete made with portland cement concrete. A more important aspect is that the negative strain values of the shrinkage-compensating concrete were 43% and 30% lower, respectively, at $0.25f'_c$ and $0.50f'_c$ than the portland cement concrete.

While the ultimate amount of shrinkage strain is important for a given situation, a more important consideration is the ultimate amount of negative strain, because it is in this area that the restrained concrete will be subjected to tensile stresses and thus to the susceptibility of cracking.

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Table 1. Amounts of Reinforcement Using Welded Wire Fabric

Prism Thickness, in.	Number of Pieces of Fabric	Spacing of Fabric Openings and Wire Gage	Reinforcement Percentage
1	1	2 in. x 2 in. No. 14	0.30 ¹
1	1	2 in. x 2 in. No. 12	0.47
1	1	1 in. x 1 in. No. 14	0.50
1	1	1 in. x 1 in. No. 12	0.78
2	2	2 in. x 2 in. No. 14	0.30
2	2	2 in. x 2 in. No. 12	0.44
2	2	1 in. x 1 in. No. 14	0.50
2	2	1 in. x 1 in. No. 12	0.78
4	4	2 in. x 2 in. No. 14	0.30
4	4	2 in. x 2 in. No. 12	0.44
4	4	1 in. x 1 in. No. 14	0.50
4	4	1 in. x 1 in. No. 12	0.78

¹ Reinforcement percentage = the total cross-sectional area of the horizontal wires at the end of the prism $\div$ the total cross-sectional area of the prism, viewed from the end.

Table 2. Strength Properties of 1 in. 5 in. (2.5 cm by 12.7 cm) Prisms, $p=0.30\%$

Strength Property	Concrete Mix Designation			
	7.5 SC		7.5 SCA	
	14 Days ¹	28 Days ²	14 Days	28 Days
Compressive Strength, psi (kgf/cm ²)	5,110 (359)	6,020 (423)	4,050 (285)	4,480 (315)
Young's Modulus, 10 ⁶ psi (10 ⁶ kgf/cm ²)	3.59 (0.25)	3.61 (0.25)	2.90 (0.20)	2.97 (0.21)

¹ Tests made after 14 days of curing in 100% RH.

² Tests made after 28 days of curing in 100% RH.

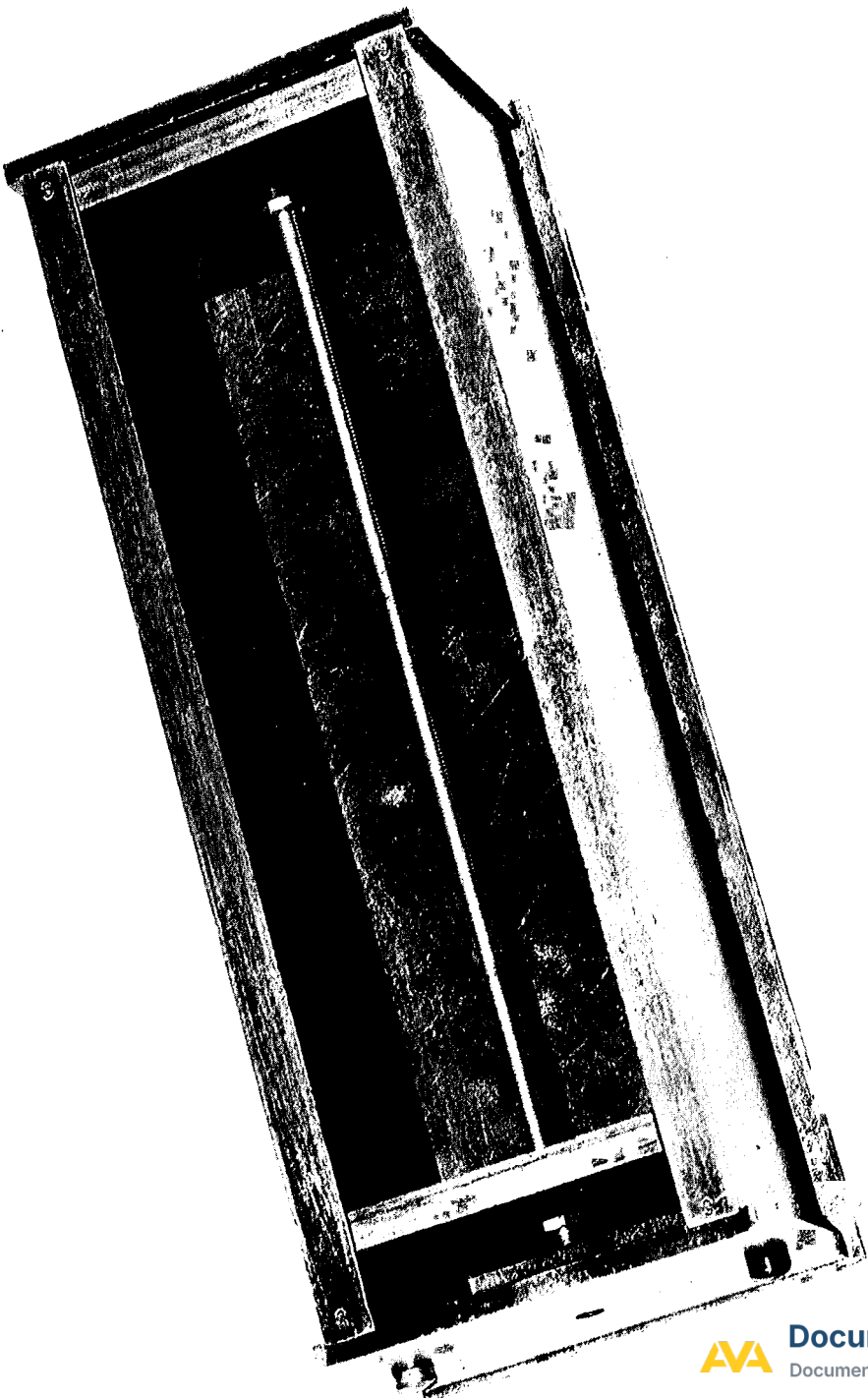


Fig. 1--Modified ASTM C-490 bar mold with threaded rod

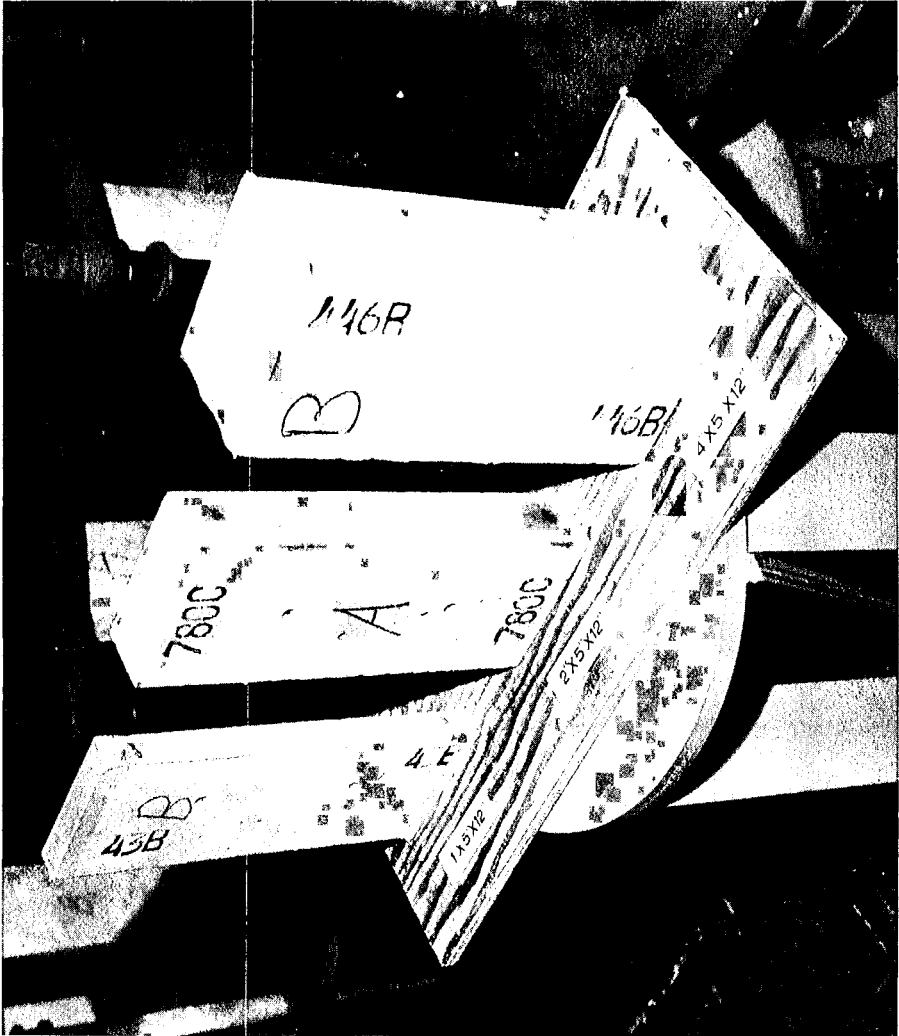


Fig. 2--Prismatic test specimens

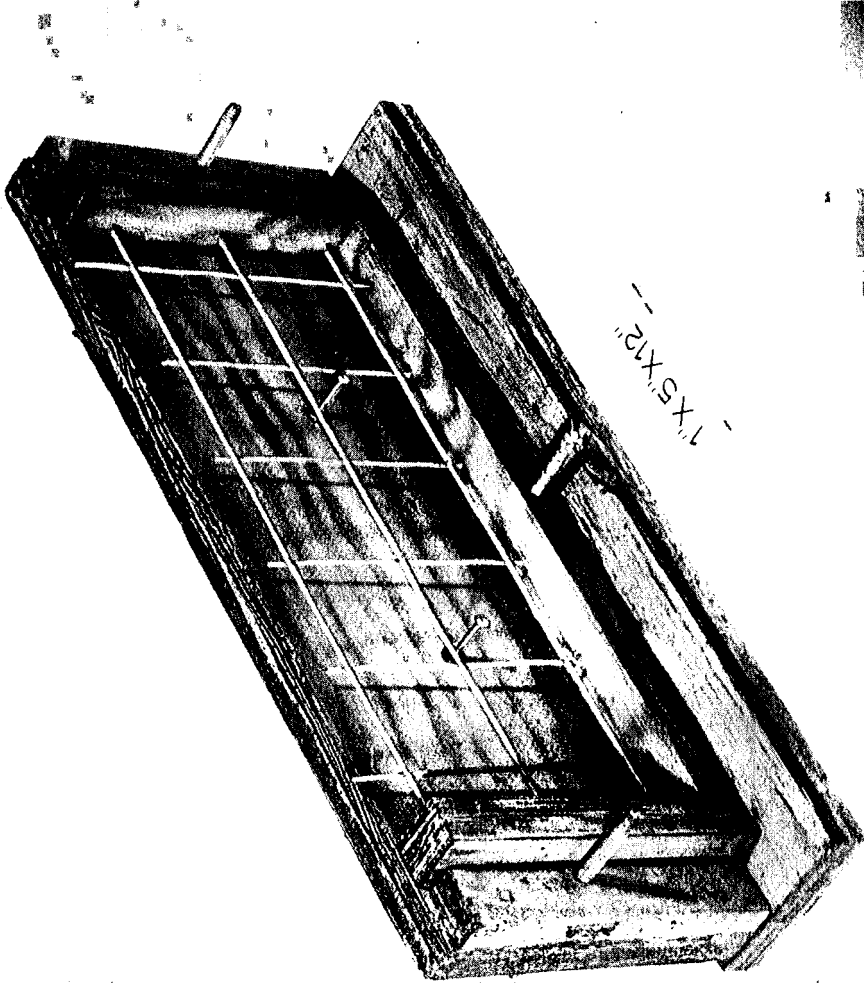


Fig. 3--Form for prisms 1 in. (2.5 cm) thick, showing reinforcement

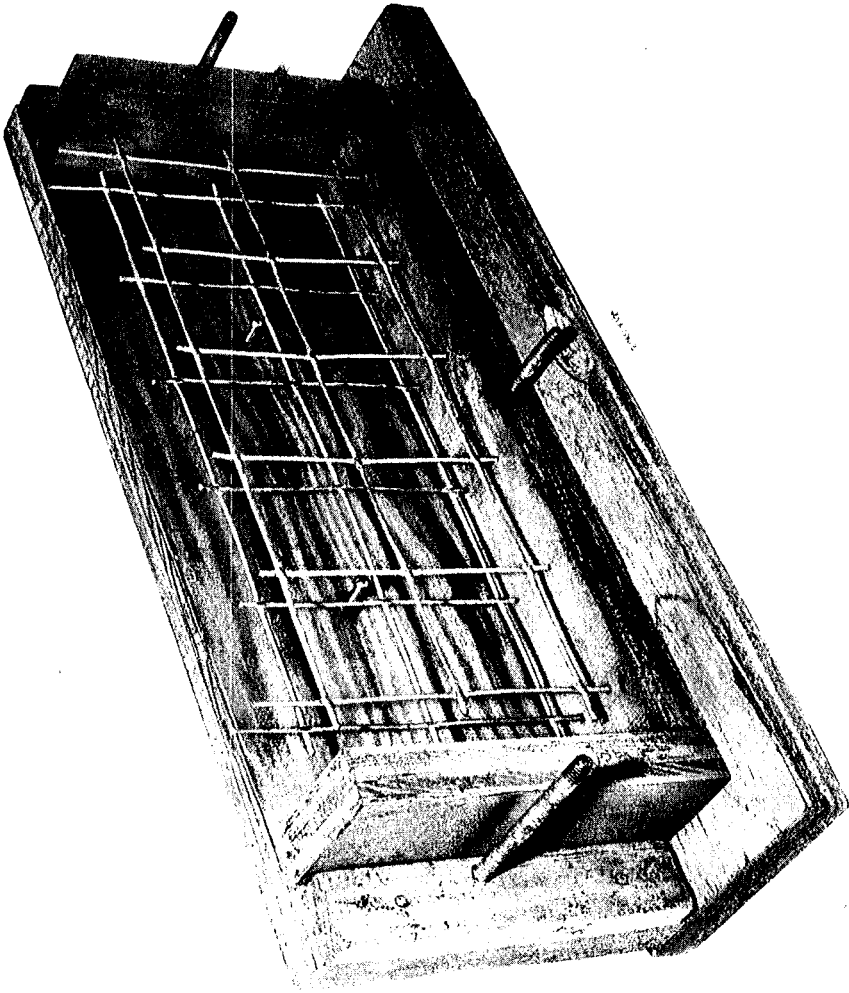


Fig. 4--Form for prisms 2 in. (5.1 cm) thick, showing reinforcement

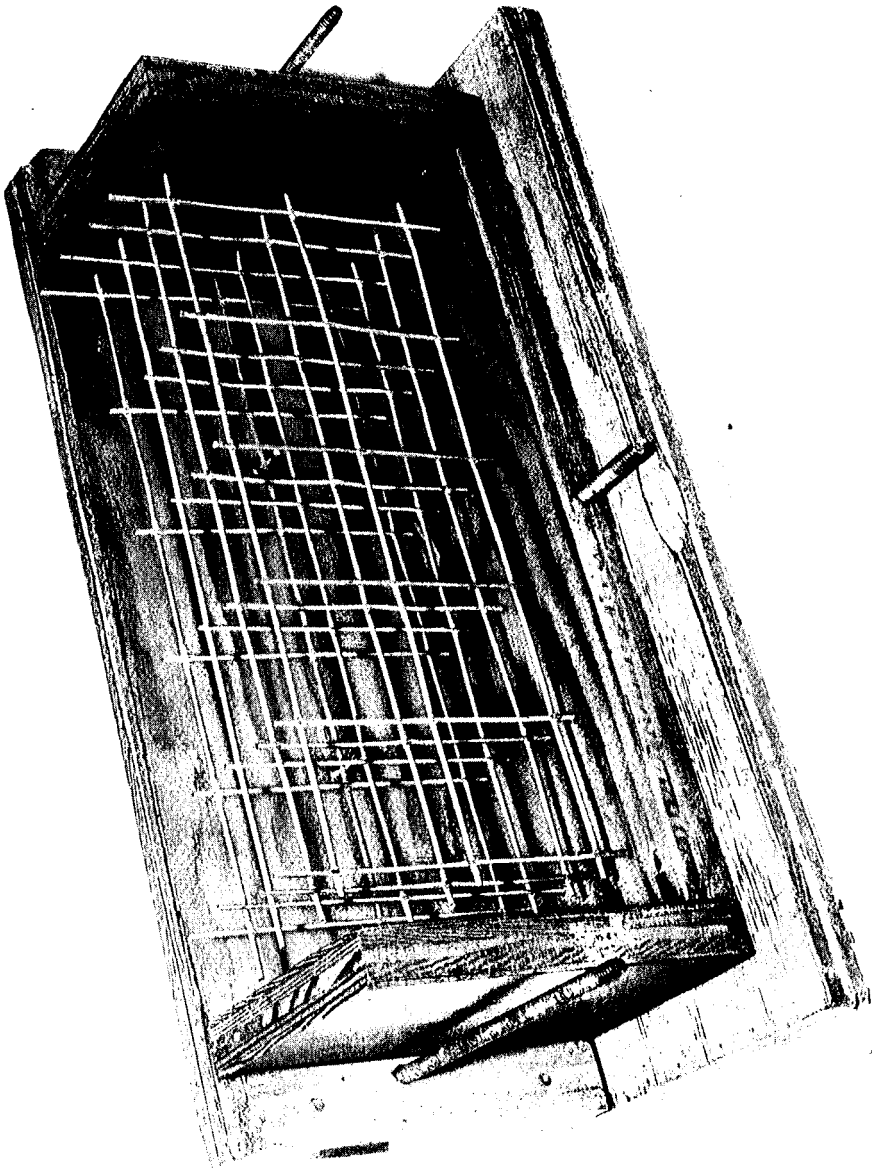


Fig. 5--Form for prisms 4 in. (10.2 cm) thick, showing reinforcement

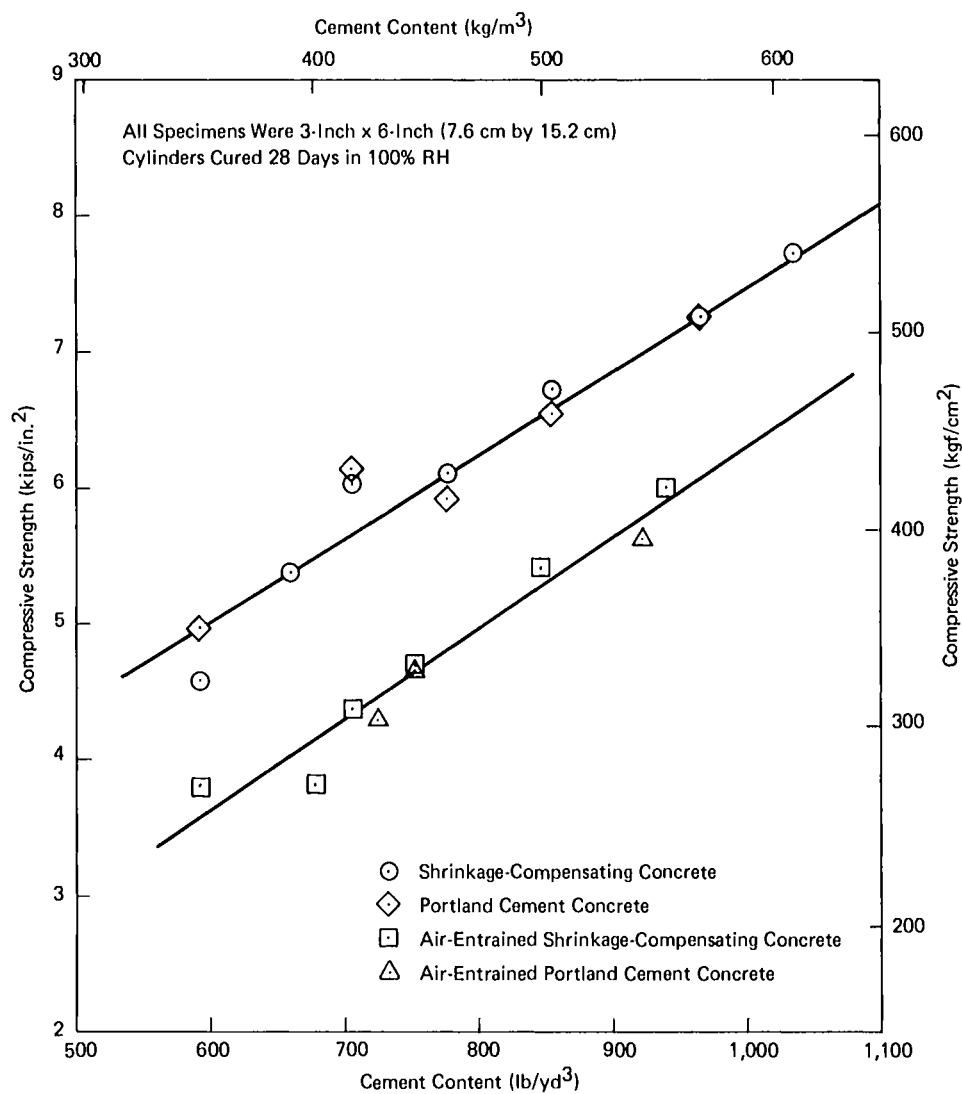


Fig. 6--Compressive strengths of unrestrained cylinders as a function of cement content

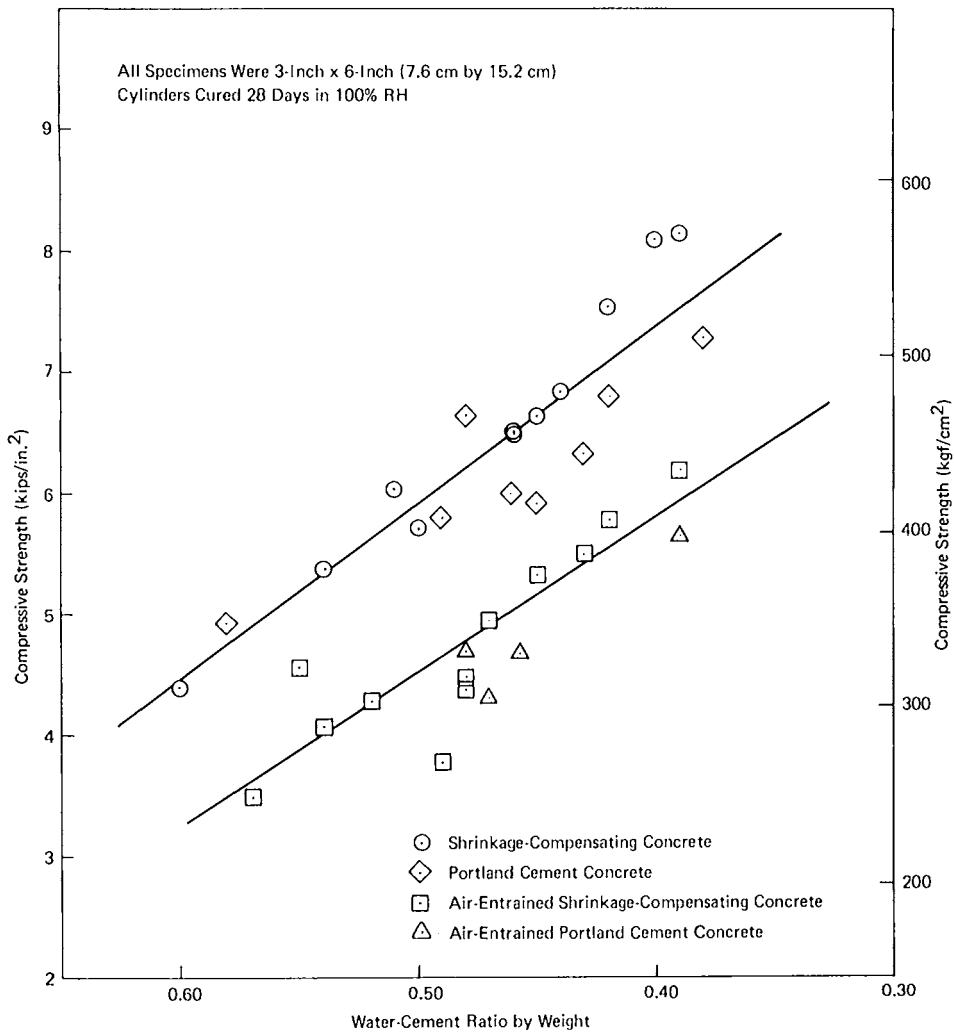


Fig. 7--Compressive strengths of unrestrained cylinders as a function of water-cement ratio

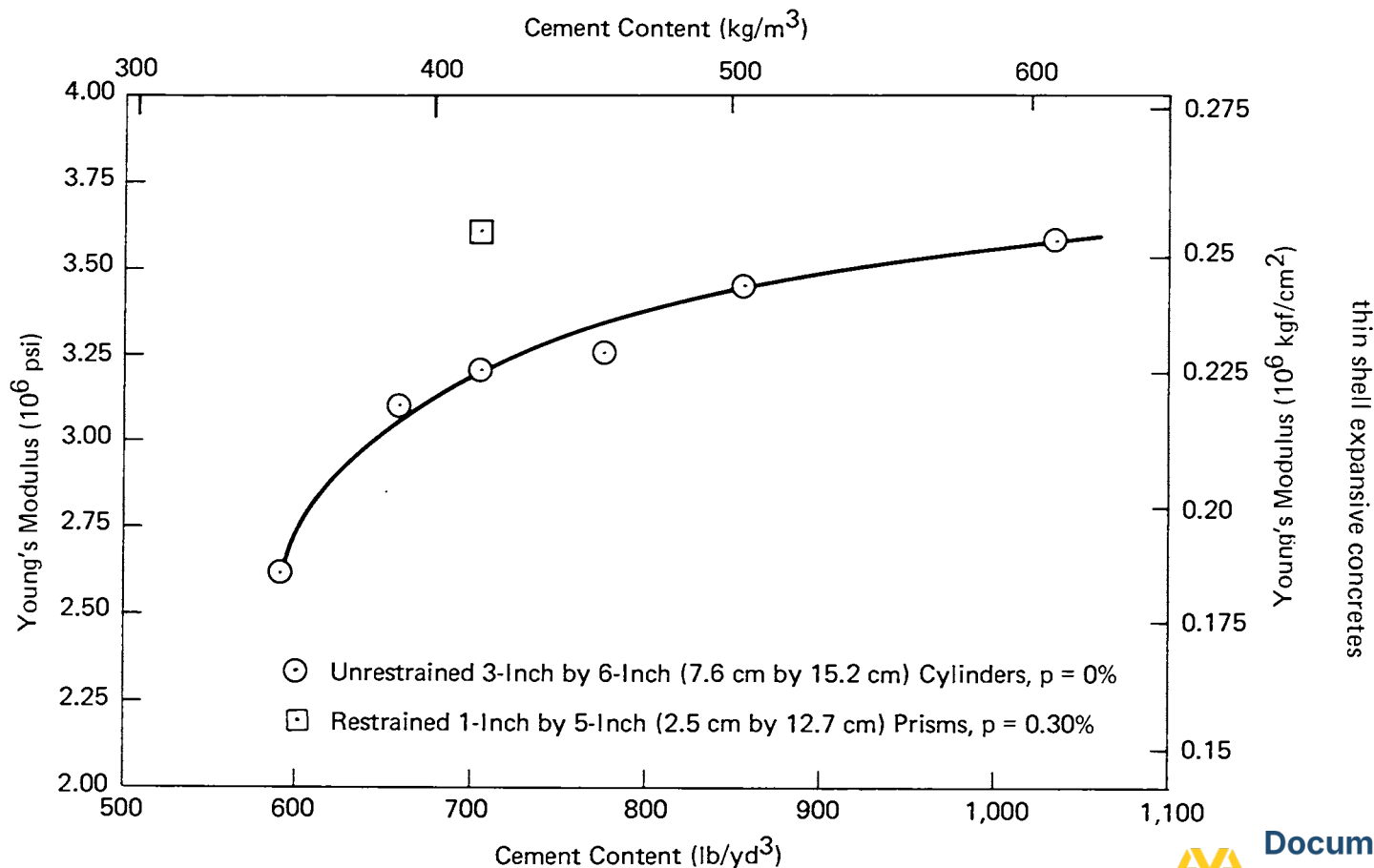


Fig. 8--Young's moduli of shrinkage-compensating concrete after 28 days of fog curing

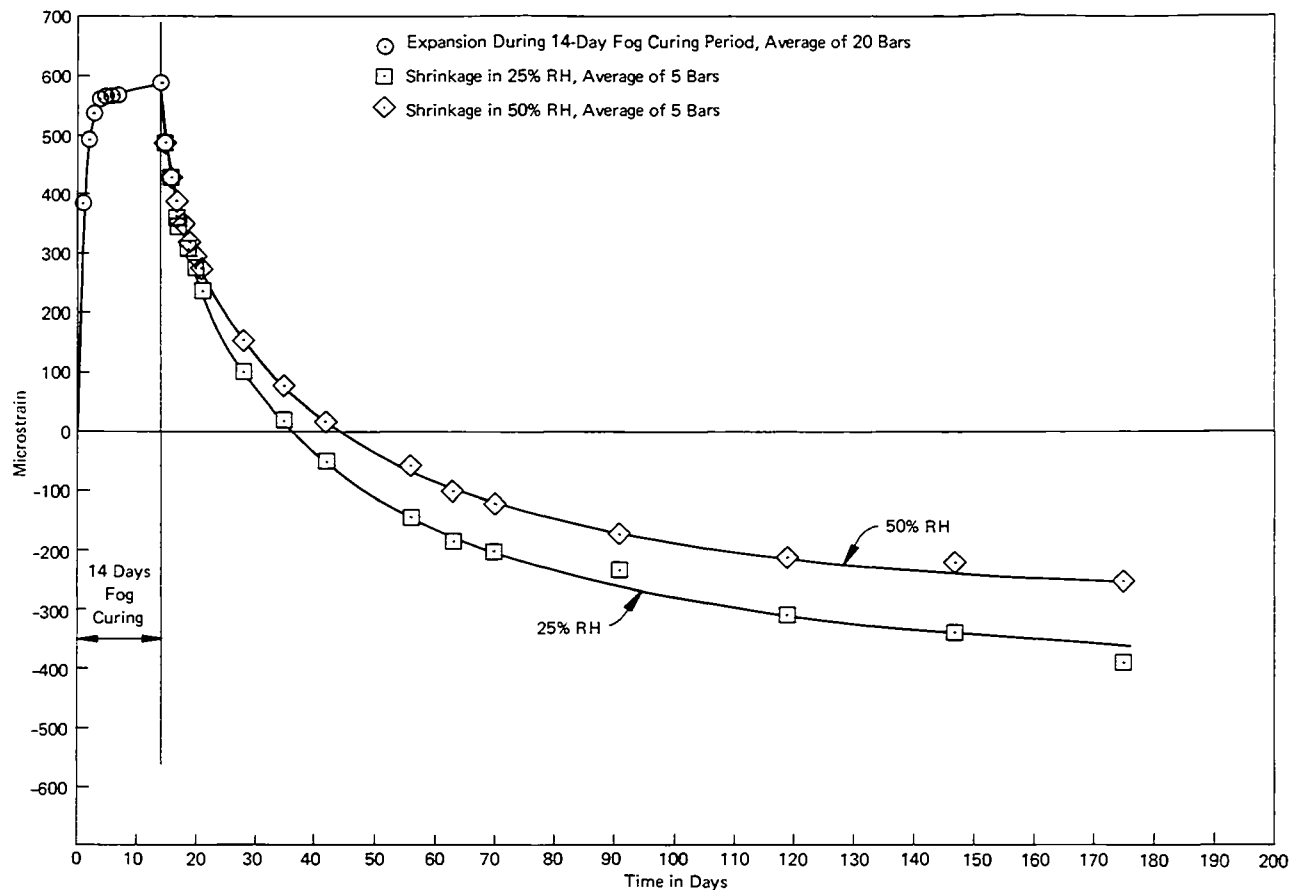
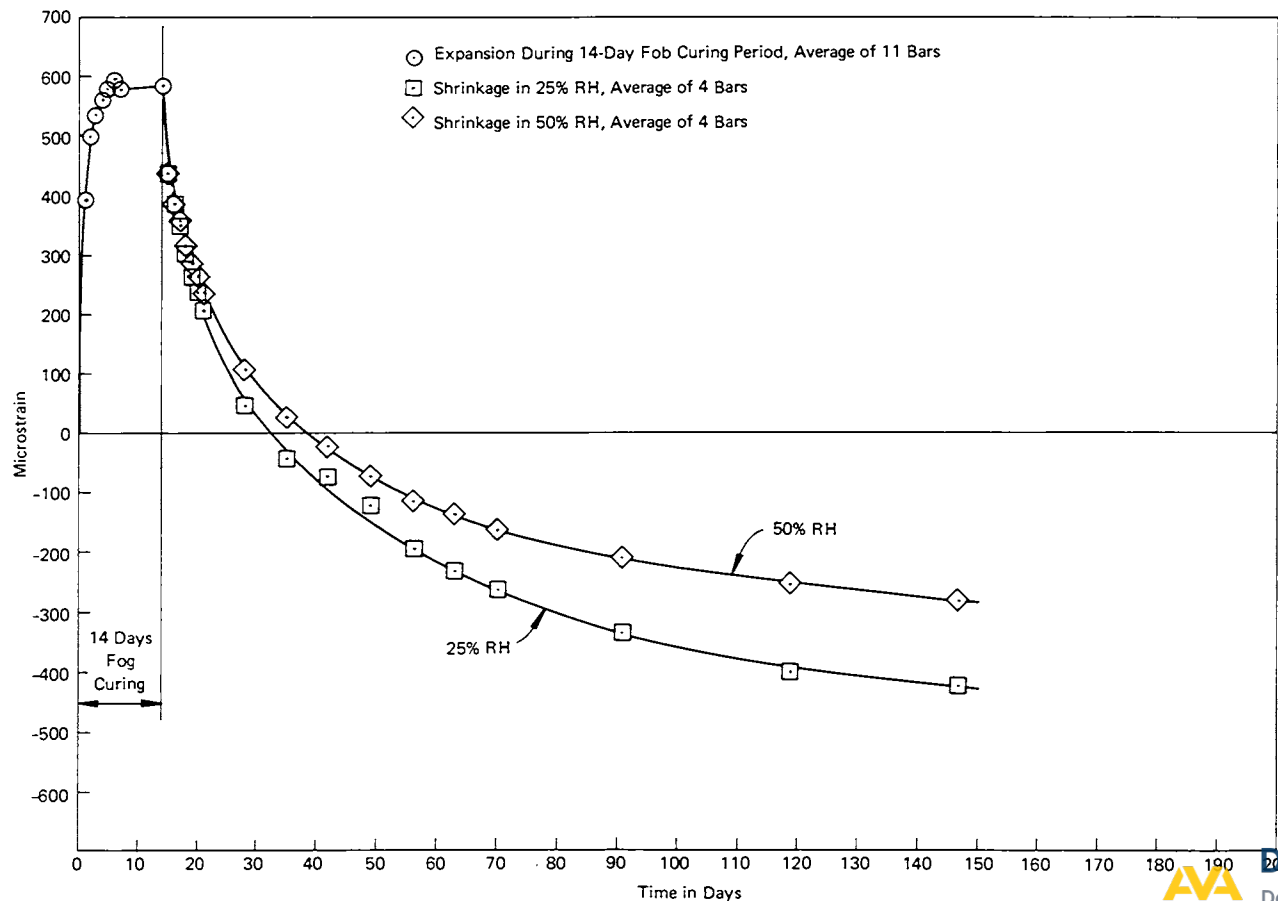


Fig. 9--Expansion and shrinkage of bars, $p=0.19\%$, 7.5 SC

Fig. 10--Expansion and shrinkage of bars, $p=0.19\%$, 7.5 SCA

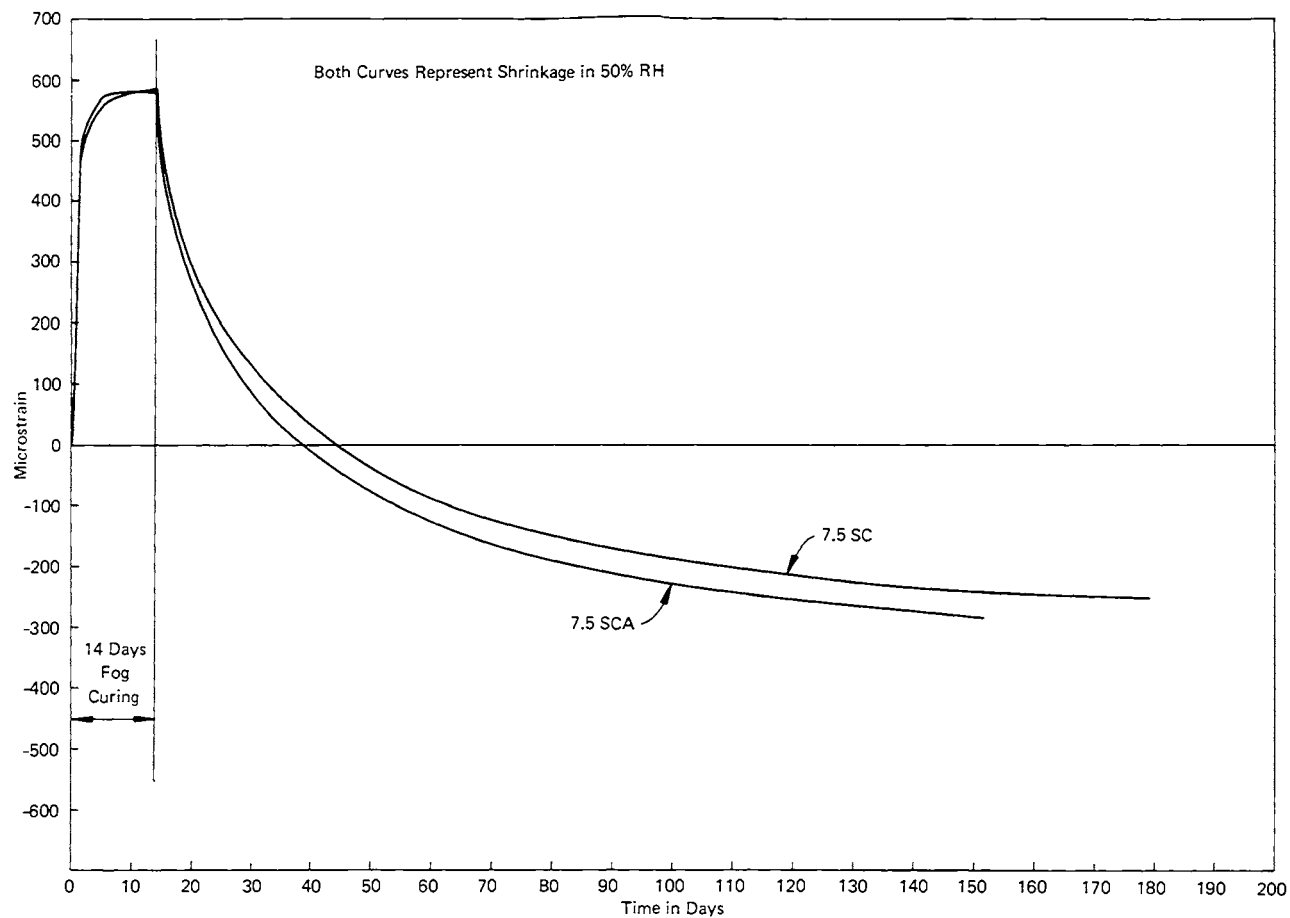


Fig. 11--Comparison curves for bars, $p=0.19\%$, 7.5 SC and 7.5 SCA

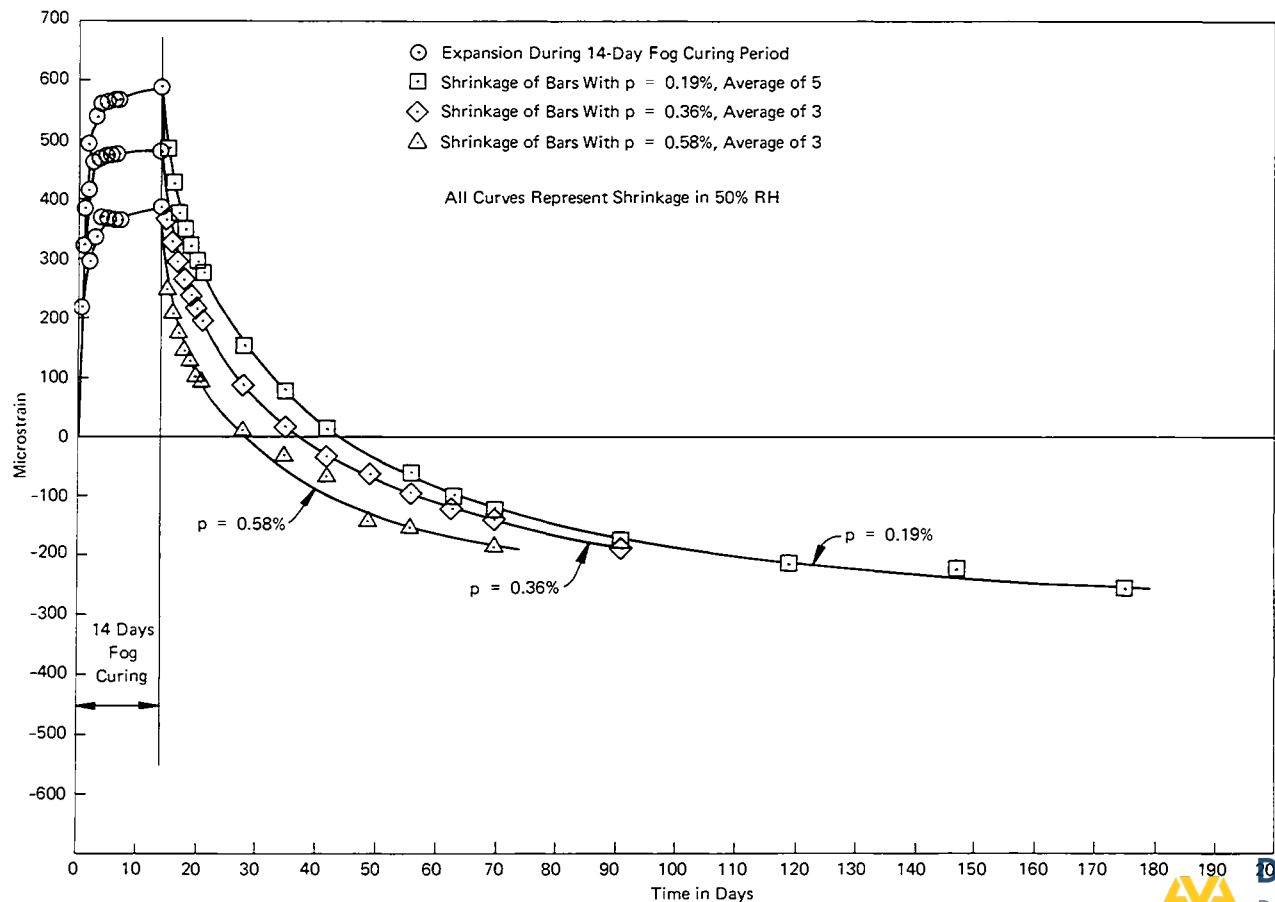


Fig. 12--Effects of restraint on expansion and shrinkage of bars, 7.5 SC

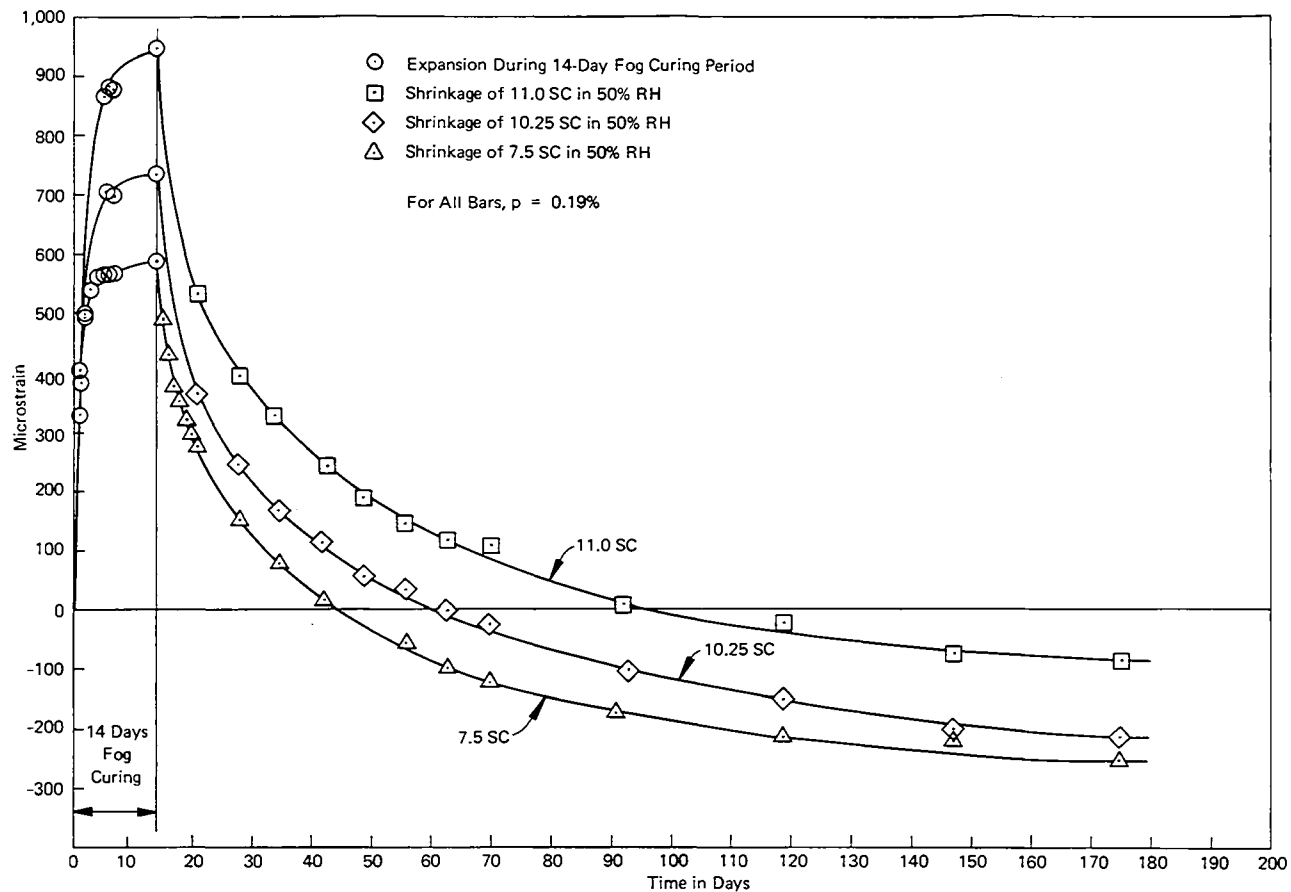


Fig. 13--Effects of cement content upon expansion and shrinkage of bars, 7.5 SC

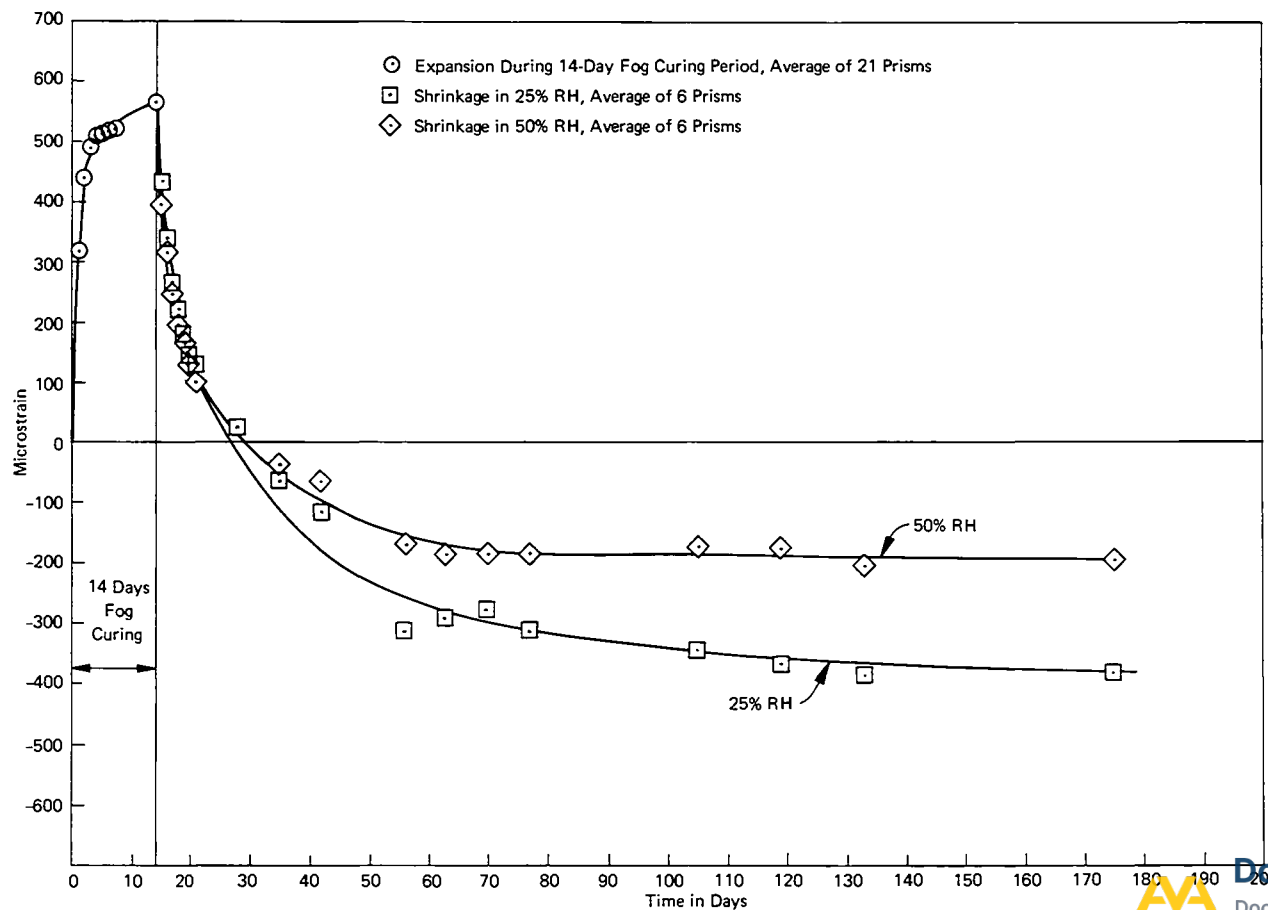


Fig. 14--Expansion and shrinkage of 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms, $p=0.30\%$, 7.5 SC

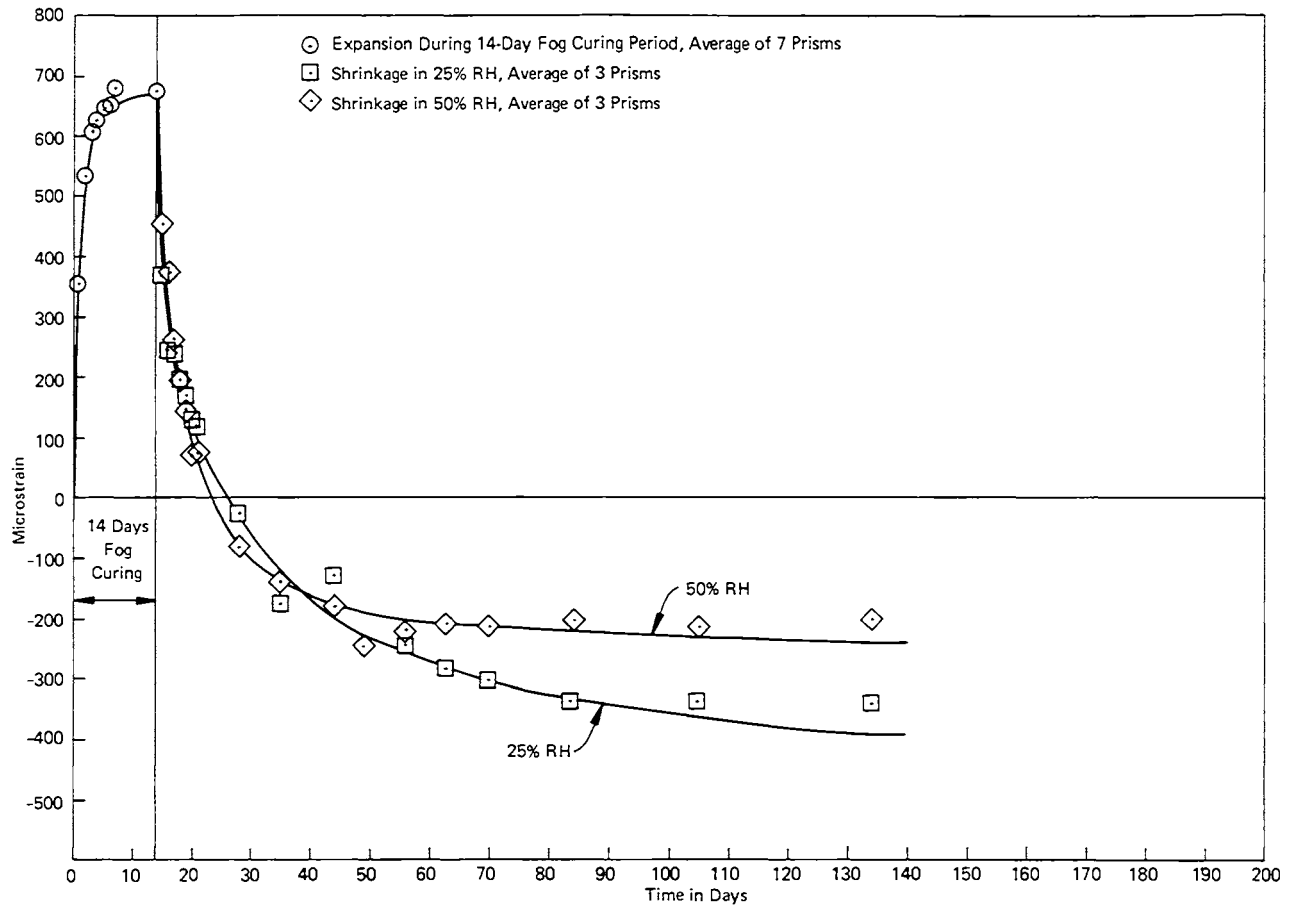


Fig. 15--Expansion and shrinkage of 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms, $p=0.30\%$ 7.5 SCA

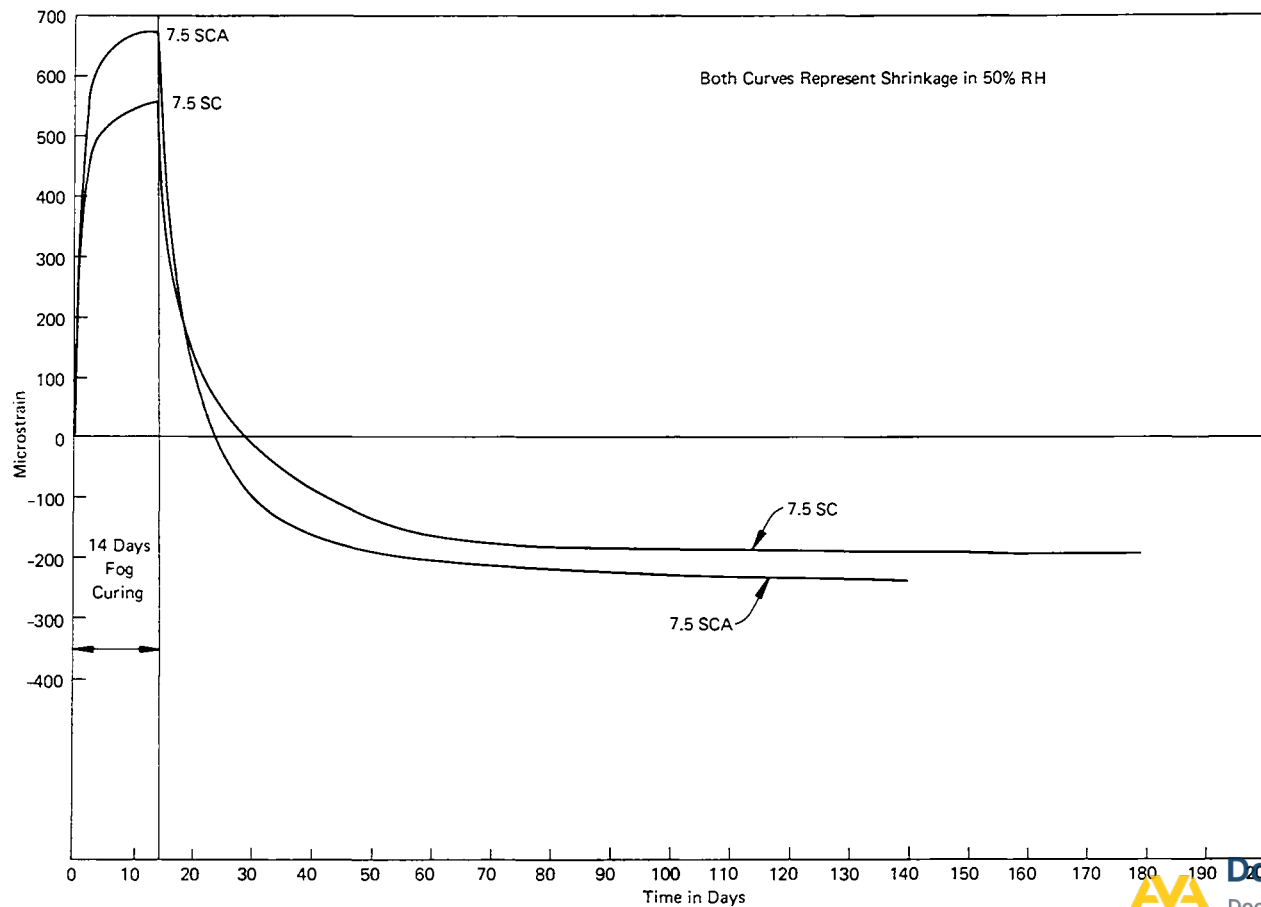


Fig. 16--Comparison curves for 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms, $p=0.30\%$, 7.5 SC and 7.5 SCA

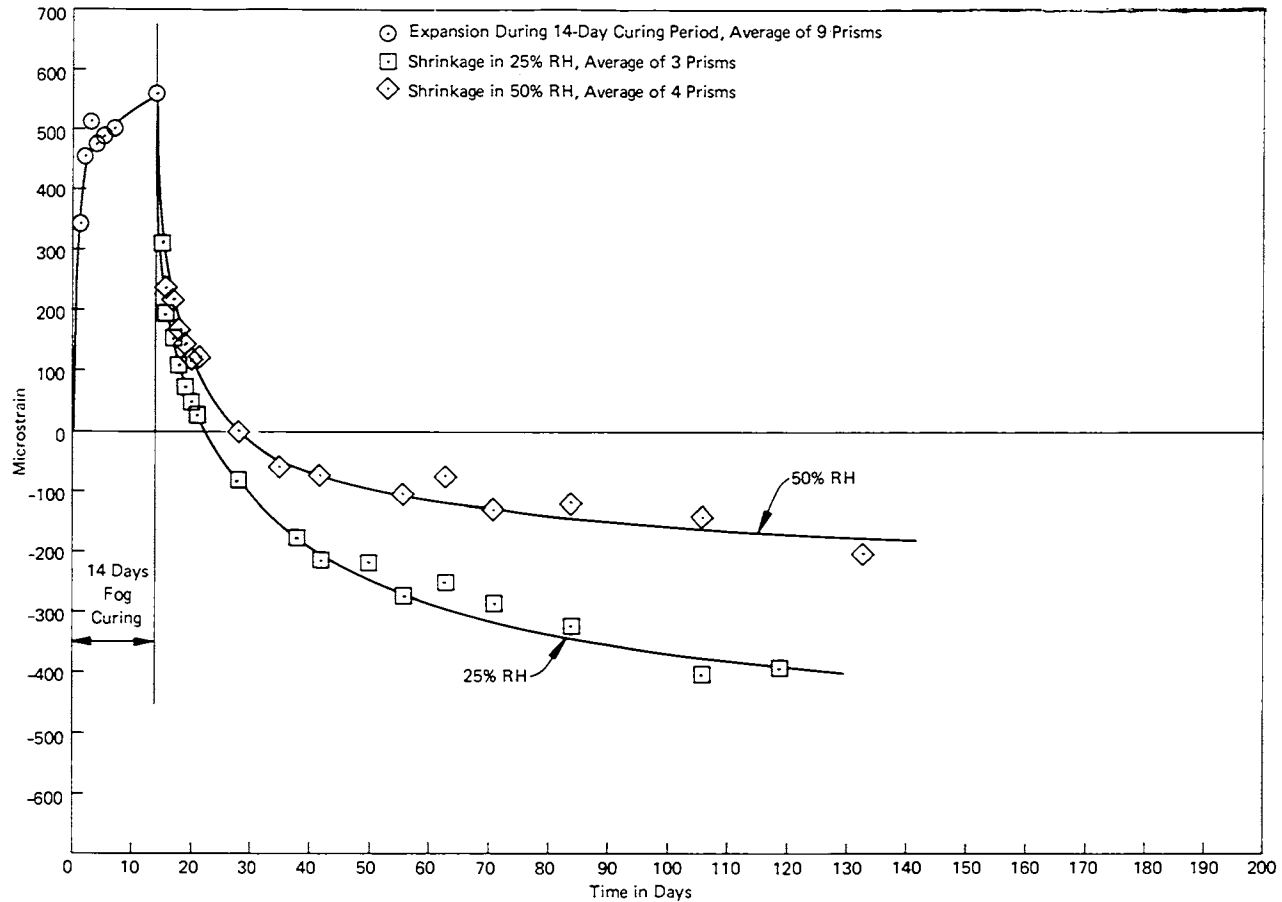


Fig. 17--Expansion and shrinkage of 2 in. by 5 in. (5.1 cm by 12.7 cm) prisms, $p=0.30\%$, 7.5 SC

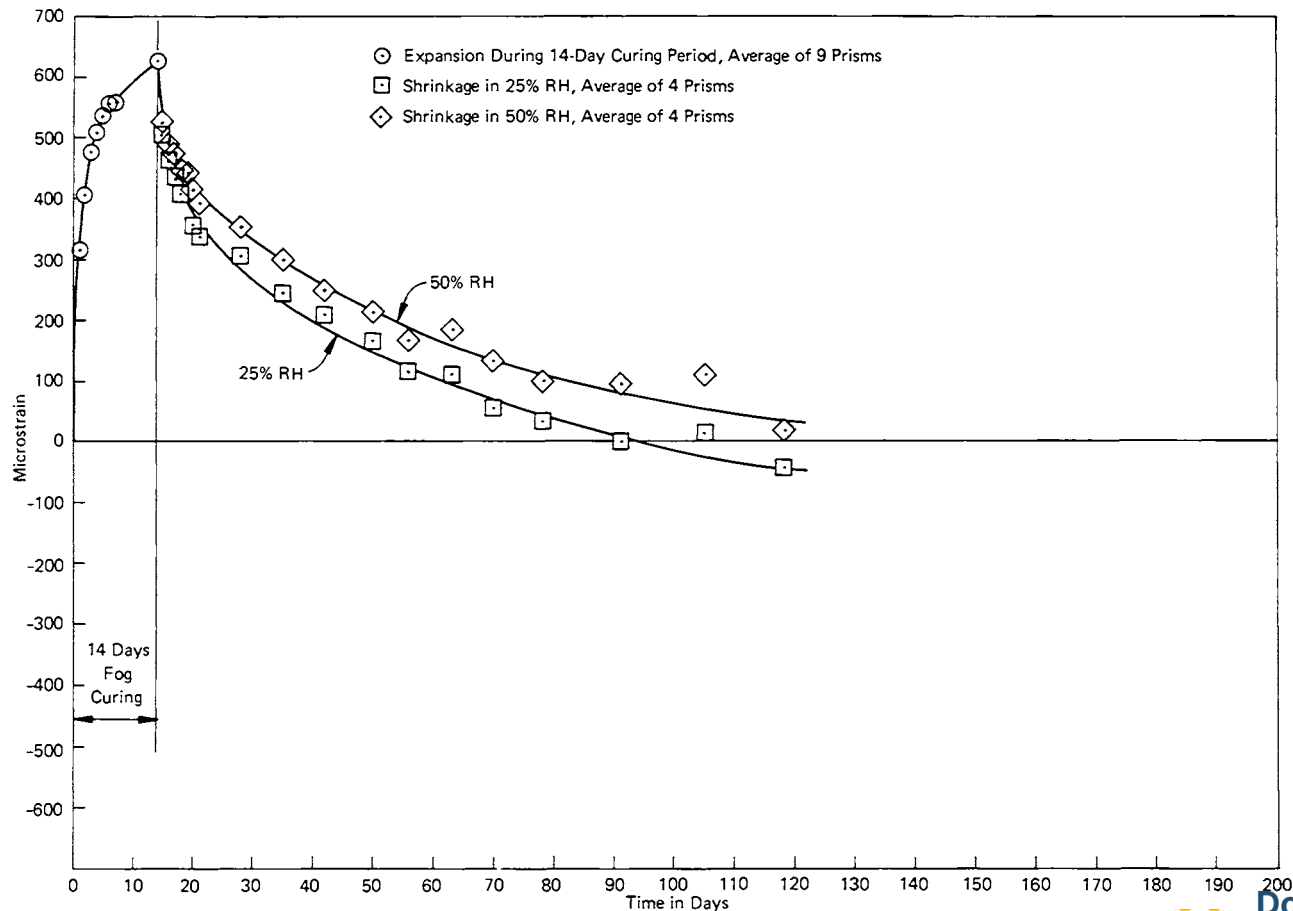


Fig. 18--Expansion and shrinkage of 4 in. by 5 in. (10.2 cm by 12.7 cm) prisms, $p=0.30\%$, 7.5°C

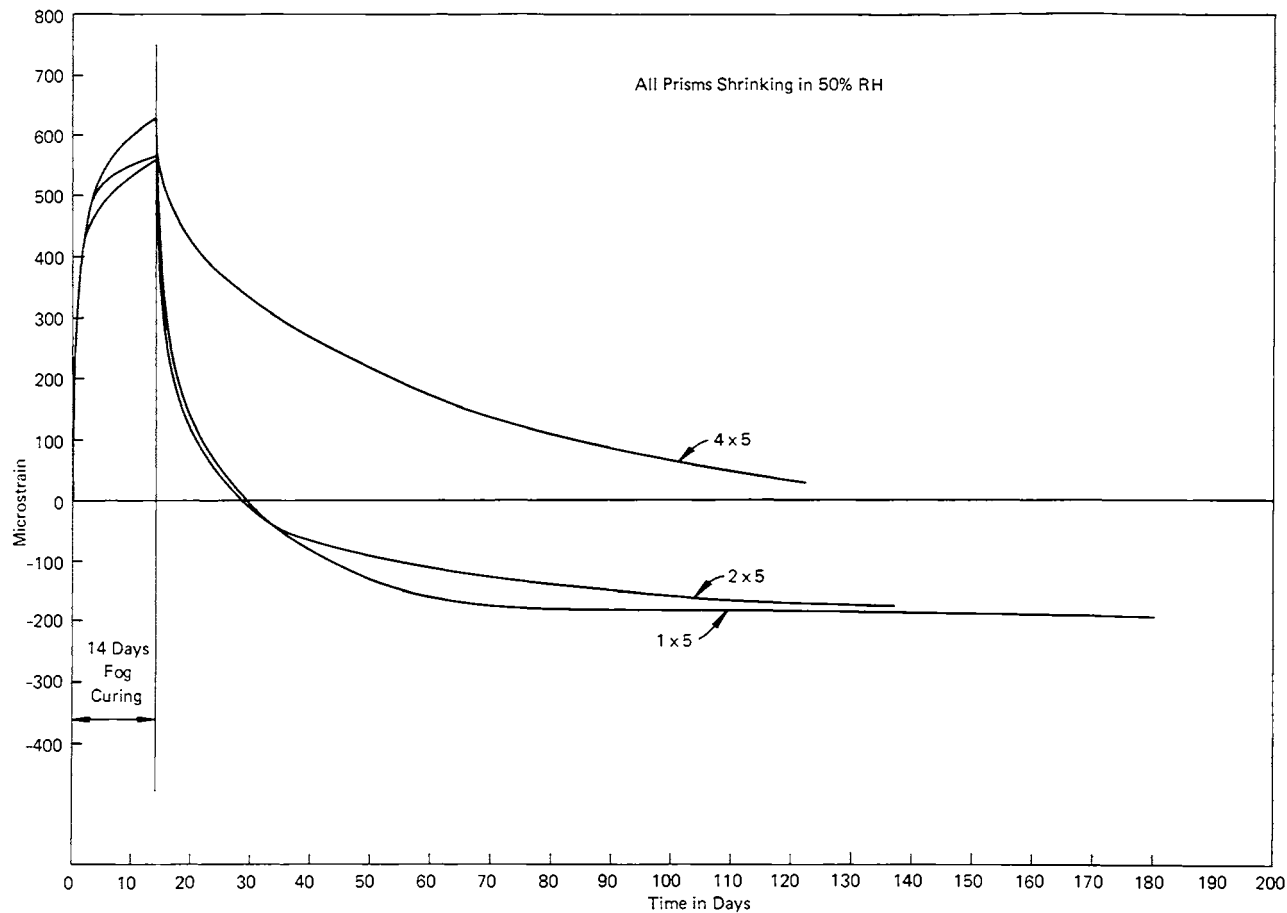


Fig. 19--Effects of prism size on expansion and shrinkage, $p=0.30\%$, 50% RH, 7.5 SC

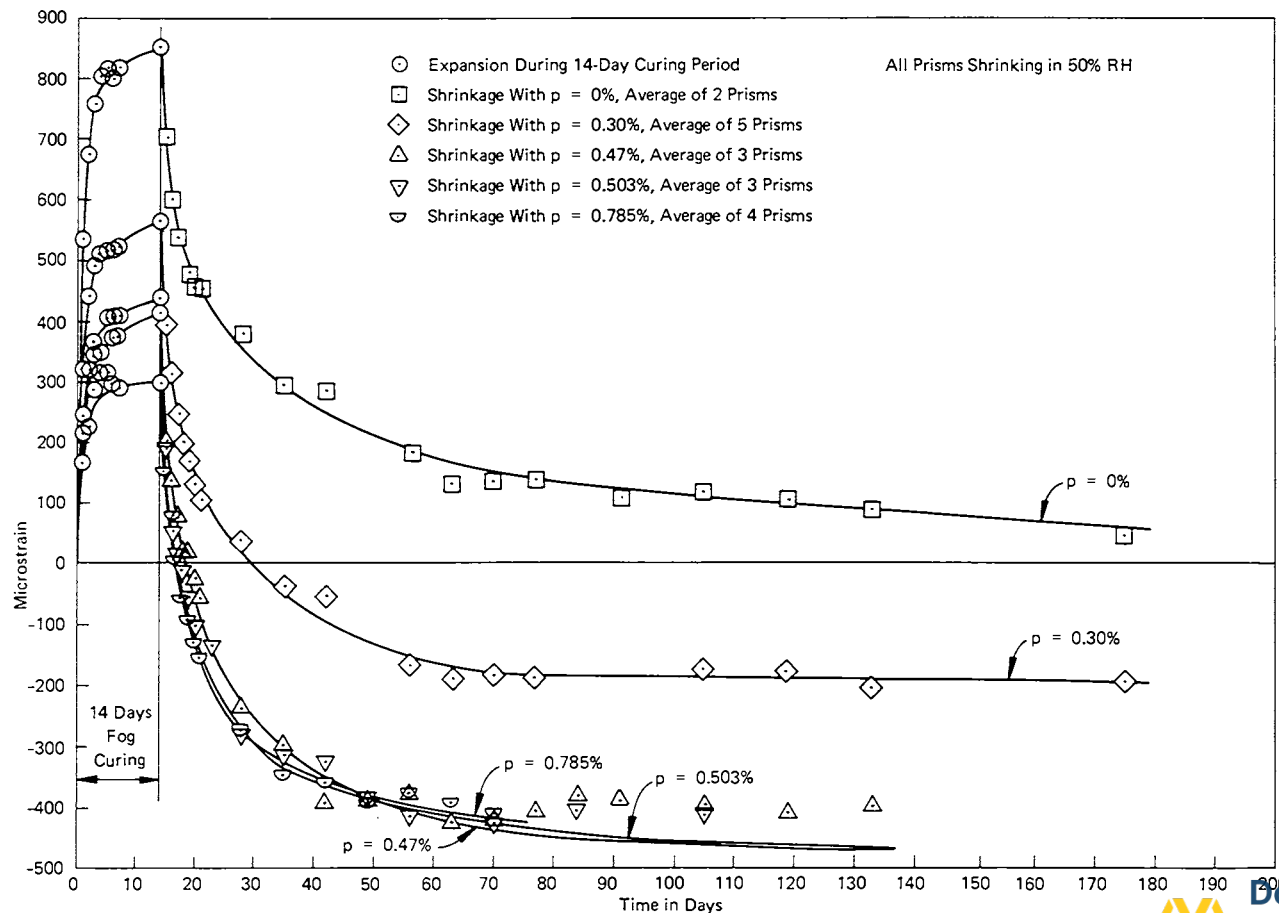


Fig. 20--Effects of restraint on expansion and shrinkage of 1 in. by 5 in. (2.5 cm by 12.7 cm) prisms, 7.5 sc