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EXPANSIVE CEMENTS AND EXPANDING CONCRETE

by

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SYNOPSIS

A critical literature review of expansive cements and expanding concrete is presented. The history of expansive cements, expanding concrete properties, tests on structural elements made of expanding cement and other engineering applications are discussed.

An extensive list of references is given, including both published and unpublished work.

1. INTRODUCTION

Concrete has long established its undisputed place among engineering structural materials. However, its deficiencies are well known. It is weak in tension and, on shrinkage, it can crack and may deteriorate. Shrinkage, which is due to the colloidal nature of the material,(1)* can not as yet be stopped, but its undesirable consequences, e.g., cracking, can be prevented by prestressing so that the subsequent shrinkage will only reduce the initially induced compressive stresses and will not permit the development of tensile stresses. The development of prestressed concrete, by conventional mechanical means, has its origin in France and it is probably no pure coincidence that chemical prestressing seems to share the same origin.

There are a number of substances which, upon their delayed hydration, can cause destructive expansion when present in excessive amounts in concrete. Examples are hard-burned lime and magnesia. A disruptive reaction is caused when concrete is attacked by aggressive waters containing sulfates. In 1892, Michaelis(2) was the first to attribute this disruption to the reaction between the tricalcium aluminate and the sulfates and the formation, with increase in volume, of a compound $3CaO \cdot A\ell_2O_3 \cdot 3CaSO_4 \cdot 30H_2O$, which he designated the "cement bacillus" due to its destructive action and to its rodlike shape. Two years earlier, in 1890, Candlot(3) found that the reaction product of tricalcium aluminate and calcium sulfate in an aqueous solution was a

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*Numbers in parentheses refer to references in bibliography.

hydrated salt of calcium sulfoaluminate (this has been referred to as "Candlot's salt".) Candlot reported the salt as $C_{5.5}\ A\ \bar{S}_{2.5} \cdot H_{59}$ (see Appendix I for standard abbreviations) but later research suggested the identity of the two compounds found by Michaelis and Candlot and the correctness of Michaelis' formula. This compound has the same composition and structure as the naturally occurring mineral ettringite, reported by Lehman in 1847.

The transition from science to engineering was made when it was first suggested to use the disruptive expansive component and put its forces to work towards chemical prestressing. This is usually attributed to the French engineer Lossier(4,5,75,76) who claims that he was considering such use as early as 1925. However, Kühl(6) credits the earliest suggestion to Gutmann in 1920. The basis of chemical prestressing is the high energy of the expansion, i.e., its ability to perform work and expand under restraint. Without restrain, internal or external, there could be no chemical prestressing. A French patent was issued in February 1936 to Etablissements Poliet and Chausson, Paris, and during the late 1930's the French work predominated. Since about 1947, much work has been done in the U.S.S.R., particularly by Budnikov(7) and Mikhailov(8,9,10). In 1958, Klein and Troxell(11) published the discovery of a new compound, an anhydrous calcium sulfoaluminate. This new development led, in the U.S.A., to much research work, both basic and applied, and to field applications of a commercially produced expansive cement. A recent annotated bibliography(12) lists ten expansive cement patents (five American, two Russian, two British and one Japanese) dating from 1937 to 1964.

In the U.S.A., two types of expansive cement are distinguished:

(a) "shrinkage - compensating cement," aimed mainly to minimize the development of cracks due to drying shrinkage in concrete, and

(b) "self-stressing cement," causing greater expansion and aimed at structural chemical prestressing.

At present, in the U.S.A., the "shrinkage-compensated cement" is the only one available commercially, produced by a few portland cement manufacturing companies under license. It is claimed(13) that up to 1965 approximately 50,000 cubic yards of expanding concrete have been placed since the expansive cement became available in 1963. The economic potential of this cement can be illustrated by an example. At present, to prevent shrinkage cracks, there are about 340 contraction joints per mile of California(14) concrete pavements, at a cost of $1,200.

The use of "self-stressing" cement is still mainly experimental. A basic study of expansive cement and expanding concrete is being carried out at the University of California at Berkeley. This four year research program is being supported by National Science Foundation Grants. Research in this area is also being carried out at other educational institutions and by the Portland Cement Association. Work is proceeding on a wide spectrum, from basic physico-chemical investigations to material properties and structural applications.

This literature survey attempts to summarize critically some of the chemistry of expansive cements, relevant aspects of crystal growth under pressure, various properties of expanding concrete, the experimental structural tests reported so far and the known engineering applications. A number of good review papers have been published.(15,16,17,18,19,20)

2. CHEMISTRY OF EXPANSIVE CEMENTS

We shall only consider cements in which the expansive mechanism is based on the crystal growth of calcium-sulfoalminate hydrates. There are expansive cements with other mechanisms, e.g., based on aluminum and hydrogen gas or on the rusting of fine iron particles.

(a) C-A-$\bar{S}$-H System

Lerch, Ashton and Bogue(21) (1929), Jones(22) (1938), Kalousek, Davis and Schmertz(23) (1949), Eitel(24) (1957) and many others have investigated the C-A-$\bar{S}$-H system but even to date information available on the structure of these hydrates, as defined by X-ray diffraction spectra, differential thermal analyses and optical constants is still limited.

There seems to be general agreement that there are two sulfoaluminate hydrates of calcium, which can be present in portland cement hydrates:

(a) the monosulfate hydrate of composition $C_4A\bar{S}H_{12}$ and possessing a hexagonal, plate-like layer structure

(b) the trisulfate hydrate of composition $C_6A\bar{S}_3H_{32}$ (the "cement bacillus," "Candlot's salt" or ettringite mentioned above) and needle-shaped crystals.

The number of water mols shown in the above compositions, 12 and 32, are the usually given figures. However, there is a wide range of reported values, which probably depend on the circumstances investigated. For the monosulfate, Mehta(25) identified a compound with 18 mols and he also quotes (20) an identification by Turrizziani and Schipps of 14-15 mols. For the trisulfate, Henkel and Rost (26) quote more than three investigations with 30 to 31 mols, three with up to 33.26 mols and one with 32 mols. In addition Henkel and Rost (26) claim that there is also present a low-water form of the trisulfate hydrate with 10-12 mols of water. They state that it was discovered by Michaelis (2) and investigated by many workers.

Which is the sulfoaluminate hydrate responsible for expansion under restraint? There is general agreement (10,19) with Michaelis' conclusion that it is the trisulfate hydrate $C_6A\bar{S}_3H_{32}$. According to Ferrari (27), the formation of expansive sulfoaluminate occurs only in a medium maintained supersaturated with calcium ions. The formation of the monosulfate hydrate under restraint is believed (10,25) not to produce expansive forces, probably due to its layer structure. The presence of monosulfate hydrate in concrete, however, represents a potential danger of disruptive expansion on exposure to sulfate waters and the resulting transformation of monosulfates into trisulfates. Exception to the above views were expressed by Chatterji and Jefferey, (28) who believe that, in solutions saturated with CH, it is the monosulfate hydrate that is responsible for the expansion phenomena. Also, Henkel and Rost (26) concluded that the cause of the destructive attack on concrete by sulfates is the low-water trisulfate $C_6A\bar{S}_3H_{12}$. They used crystalline optics for identification and this is not as accurate as X-ray diffraction.

Before proceeding to a description of the French, Russian and American expansive cements some general comments should be made. In all these cements the expansion is probably due to the formation of the same trisulfate hydrate. Therefore, the origin of the ions forming the trisulfate hydrate is irrelevant from the point of view of the final product. In other words, there may be many combinations of materials in the cement resulting in the same final product. However, there are other considerations which, from the engineering point of view, can make the difference between success and failure of expanding concrete.

One basic aspect of the successful use of expanding concrete is what might be called the "strength-expansion" relationship. This is the relation between the rates of strength increase (hydration of tobermorite) and expansion (formation of trisulfate hydrate.) Since expansion is a damaging disruptive action, the concrete must be able, to achieve desirable mechanical properties, to gain simultaneously in strength and also to repair, internally, the damage caused by the expansion. The rate of expansion is also most important. Consider the extreme cases. If the full expansion is reached instantaneously at mixing, there would be no damage since the tobermorite structure is yet to form, but, for the same reason, no prestress could be achieved. If the expansion does not take place till concrete is fully hardened and no more tobermorite can form, the disruption can not be repaired, the damage to mechanical properties can be very serious but prestress can arise. Obviously, an optimum rate of expansion and strength-expansion relation is desirable as the practical compromise. Different expansive cements vary in the difficulty and expense required to achieve the ideal optimum conditions. The degree of control that can be achieved is a measure of successful application.

(b) French Cement

Initially, the expansive cement produced in France, by the Lafarge Company, was a mixture of portland cement, gypsum and aluminous cement. It was not successful since the reaction was erratic and difficult to control.

Later developments, due to Lossier(4,5,75,76) used a mixture of portland cement, a ground clinker, and a stabilizer. The clinker was obtained by burning a mixture of gypsum, bauxite, and chalk. The stabilizer was blast furnace slag and was used to control the reaction by combining with the CH, reducing its concentration in the liquid phase and thus affecting the formation of the trisulfate hydrate. The cement was not successful due to difficulties in controlling and reproducing the expansion.

Originally the ground clinker was thought to be an anhydrous calcium sulfoaluminate. However, in 1952, Lafuma(29) reported X-ray analysis of the clinker and found no anhydrous calcium sulfoaluminate (ternary C-A-$\bar{S}$ compound) but only a mixture of free anhydrous $C\bar{S}$, calcium aluminates (mainly C_5A_3) and C_2S (dicalcium silicate). Based on later information, the temperature of the clinker burning must have been below that required for the formation of the ternary compound. However, Lafuma concluded wrongly that no benefit is achieved by separate clinker burning and that the same trisulfate hydrate expansion characteristics could be obtained by using a mixture of high-alumina cement and calcium sulfate.

(c) Russian Cement

A number of expansive cements have been used in Russia(7,8,9,10). Cements with low expansion had an expansive component made of a mixture of calcium aluminate hydrate, obtained by the interaction of lime on aluminous cements or on calcined clay, and gypsum. In 1957, Mikhailov(9) reported cements with high expansion used for chemically prestressed concrete. These were made by intergrinding aluminous cement, portland cement and either gypsum or plaster of Paris. Because of the large amount of A ions available immediately on hydration, the monosulfate ($C_4A\bar{S}H_{12}$) is formed rather than the trisulfate ($C_6A\bar{S}_3H_{32}$). Thus the initial expansion is low and the danger of disruption exists on recrystallization to the trisulfate hydrate. Mikhailov achieved proper control by using a very low water-cement ratio (e.g., 0.16), permitting the material to harden at ambient temperature for up to 24 hours and then subjecting the hardened structure to hydrothermal treatment for some 6 hours at about 70-100° C. This treatment is effective because with the disruptive formation of the trisulfate hydrate, the hydration of the portland cement is accelerated, due to the high temperature, and a balanced strength-expansion relation is achieved. However, the type of hydrothermal treatment required limits the structural use of this expansive cement.

(d) University of California Cement

In 1958, Klein and Troxell(11) reported on clinkers prepared by firing mixtures of calcium hydroxide or calcite, gypsum, and bauxite or aluminum sulfate, at about 1350° C (2460° F). The examination of the clinkers gave a hitherto unknown X-ray diffraction pattern and they deduced that the clinker contained a new compound, an anhydrous calcium sulfoaluminate. Based solely upon the oxide compositions, they estimated the probable composition of the calcium sulfoaluminate to be $C_5 A_2 \bar{S}$ or $C_9 A_4 \bar{S}$. Other conclusions of Klein and Troxell were:

(a) Satisfactory strength and expansive characteristics were obtained in an expanding concrete using expansive cement made up of 20% of the above expansive clinker plus 5% slag plus 75% portland cement.

(b) Free CaO was essential for the expansion to take place.

(c) Portland cements high in alkalies and C_3A did not produce sufficient expansion. The reason is apparently(20) that the trisulfate hydrate does not form in media containing high concentrations of aluminate ions.

In 1961, Fukuda(30) reported the preparation of clinkers made by firing mixtures of bauxite, lime and gypsum at 1350° C (2460° F) and claimed to have identified $C_4 A_3 \bar{S}$ as the major constituent in all of them.

In 1962, Halstead and Moore(31) confirmed the existence of $C_4 A_3 \bar{S}$ as the sole ternary compound occurring in the system $C\text{-}A\text{-}\bar{S}$ below 1350° C. Its structure has body-centred cubic symmetry and was deduced to be of the "ultramarine" type. It reacts with water to form predominantly calcium aluminate hydrate since its content of alumina is in excess of that required to form trisulfate or monosulfate hydrate. In order to convert the whole of the alumina to trisulfate hydrate, some 6% of extra lime and 20% of extra calcium sulfate would be needed.

The literature also contains other, earlier, references to an anhydrous calcium sulfoaluminate but wrongly identified as to composition. Thus, according to Henkel and Rost,(26) "along with others, A. Zinzen(32) (1943) assumes in high temperature melts, such as boiler slags, a water-free calcium alumino-sulfate having the formula $C_6\ A\ \bar{S}_3$, whose hydration into a 'cement bacillus' may cause the breakdown of the boiler slag." Also, according to Halstead and Moore(31), "in 1957 Ragozina(33) described the synthesis of a compound to which she assigned the composition $C_{2.6-4.6} \cdot A_{1.6-3.6} \cdot \bar{S}$. This material was prepared by firing mixes of C_3 A and $C\bar{S}$, as gypsum, at 1200° C and dissolving out free lime and excess $C\bar{S}$ with 1N-acetic acid."

The new compound $C_4\ A_3\ \bar{S}$ is of interest not only because it is the sole ternary compound in the C-A-$\bar{S}$ system, its stability is unusual and its structure is of a rare type. Klein and his co-workers recognized the importance of $C_4\ A_3\ \bar{S}$ as a convenient material to achieve the desired control in expanding concrete, since the rate of formation of the trisulfate hydrate will be affected by the availability of aluminate ions which can now come from the slow hydration of the $C_4\ A_3\ \bar{S}$ compound.

The production of the expansive cement at the University of California, at Berkeley (U.S.) is shown in a flow chart in Fig. 1. A separate expansive

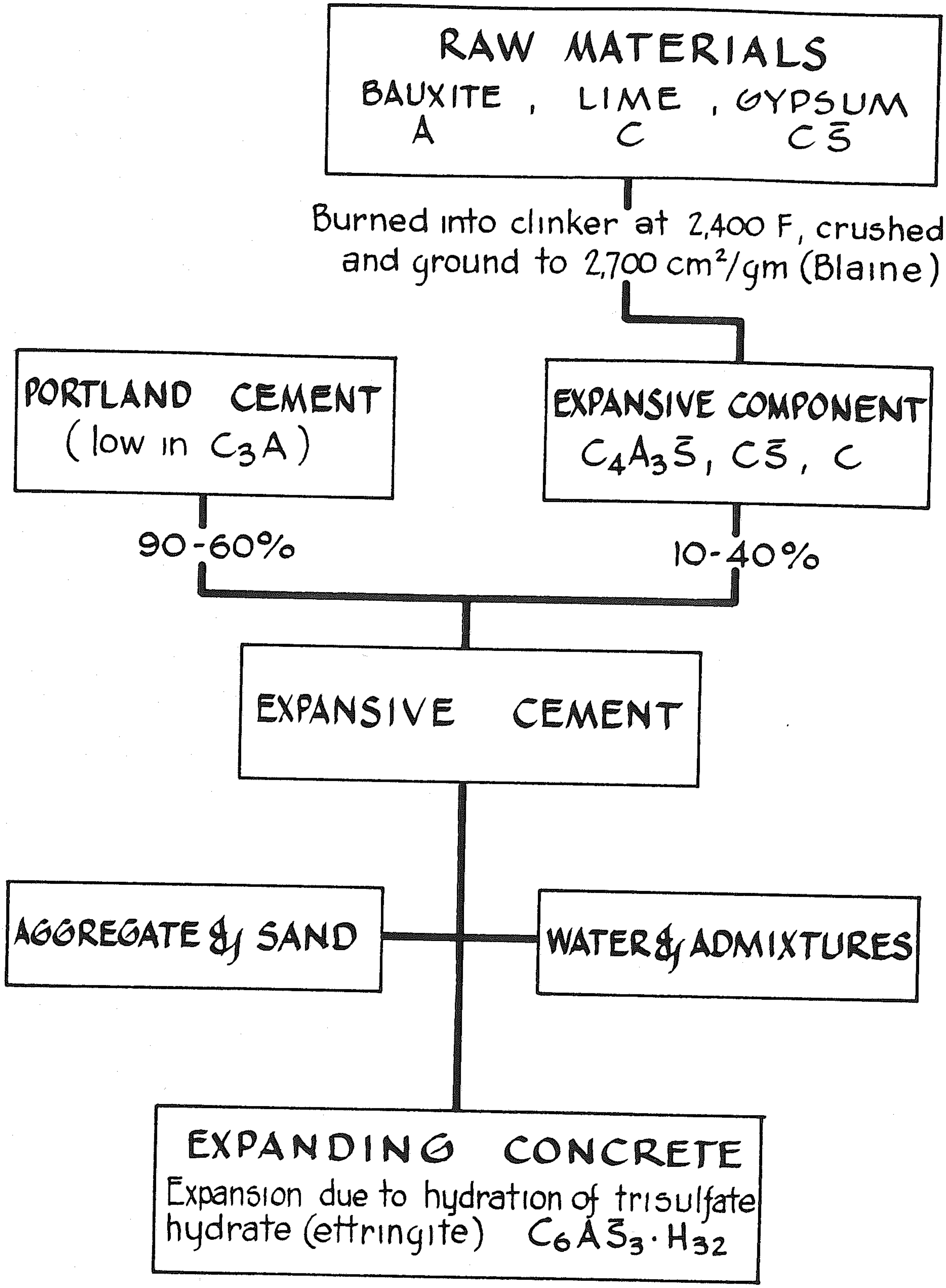

FIG. 1 FLOW CHART OF U.C. EXPANDING CONCRETE

component is burned, ground and blended with portland cement in proportions from 10 to 40% by weight of total cement. The overall procedure is the same as used by Lossier's French cements but the details differ. For example, no blast furnace slag is necessary as stabilizer. The development of an integral expansive cement, not requiring separate burning of the expansive component, can be foreseen in the future.

The two types of expansive cements mentioned earlier differ in their amount of the expansive component and, consequently, in their expansion potential. Thus

(a) "shrinkage-compensating cement" contains about 10-15% E.C. (expansive component) and has a free expansion potential as high as 0.2%.

(b) "self-stressing cement" contains about 25-40% E.C. and has a free expansion potential of about 3-6%.

(e) $C_4 A_3 \bar{S} - C\bar{S} - C - H$ System

The effect of variation in the composition of the expansive component upon products of hydration, in the above system, and upon the expansion characteristics of the hydrates has been studied and reported by Mehta(20,25), Mehta and Klein(34) and Mehta and Pirtz.(19)

Eleven mixes were made with different combinations of $C_4 A_3 \bar{S}$ (10-100%), $C\bar{S}$ (0-60%) and C (0-45%) and mixed with water (40%). At different ages (6 to 72 hours), the products of hydration were studied by powder X-ray diffraction analysis. Changes in specific volume and in the chemically combined water were also investigated. The products of hydration were mainly trisulfate hydrate, monosulfate hydrate and calcium hydroxide (CH). The mixes giving the maximum observed amounts of these products are shown in Table 1.

Table 1. Some Products of Hydration in the $C_4A_3\bar{S}$-$C\bar{S}$-C-H system (Mehta and Klein(34))

	Compound Composition			Molar Ratios		Maximum Observed
No.	$C_4A_3\bar{S}$	$C\bar{S}$	C	$C/\bar{S}$	$A/\bar{S}$	Product of Hydration
K	45	45	10	2.0	0.50	Trisulfate Hydrate
R	45	10	45	8.0	1.5	Monosulfate Hydrate
N	10	45	45	3.5	0.2	Calcium Hydroxide

Some of the conclusions reached(19,20,25,34) were:

(a) It has been definitely established that the compounds in the ternary system, useful to produce expansion, all contain all three phases, $C_4A_3\bar{S}$, $C\bar{S}$ and C. None of the compositions along the edge lines, or corners of this system are capable of producing expansion in concrete.

(b) The products of hydration were formed under conditions of nominal restraint. It was concluded that there is no direct correlation between the increase in specific volume, as measured, and the magnitude of observed expansion. Independent of the increase in specific volume, the expansion, under the nominal restraint, depends on the products of hydration. The pastes with predominantly trisulfate hydrate and CH showed large expansion while those with predominantly monosulfate hydrate and CH did not.

(c) The trisulfate hydrate existed as the stable phase in the pastes for a wide range of compositions of $C_4 A_3 \bar{S}$, $C\bar{S}$ and C. It formed within 6 hours after addition of water in the hydration of all compositions except when the $C/\bar{S}$ molar ratio was about 4.0 or more and the $A/\bar{S}$ molar ratio exceeded 1.0.

(d) There is evidence that the expansive reaction is a liquid-solid state reaction.

3. CRYSTAL GROWTH UNDER PRESSURE

Since, in chemically prestressed expanding concrete, restraint is a vital ingredient, we are dealing with a phenomenon of crystal growth under pressure. Basic physico-chemical knowledge of this phenomenon could help us to understand, control and predict related behavior of expanding concrete. Unfortunately not very much of direct applicability is available in the literature.

Becker and Day(35,36) appear to have been the first to demonstrate conclusively that a crystal can grow and lift an object when loaded with a weight.

According to the Riecke principle,(37) a crystal under linear pressure has a greater solubility or lower melting point than an unstressed one. According to Correns,(37) the pressure which a growing crystal can exercise depends upon the supersaturation of the surrounding medium. Thus we would expect the expansion under stress, in expanding concrete, to depend on the state of supersaturation. Also, restraint and pressure will cause not only visco-elastic deformations but will interact with and influence the very cause of expansion, the crystal growth.

Hansen(36) discussed crystal growth as a source of expansion in portland-cement concrete and described the work of Taber,(38) who undertook an extensive early investigation of the behavior of crystals under different environments. Some of Taber's conclusions were:

(a) A crystal will grow in a direction in which external forces oppose growth if the surface on which the forces are acting is in contact with a solution that is supersaturated with respect to it. As the solubility increases with pressure, for any increase in the forces opposing growth a corresponding increase in the concentration of the solution is necessary for the growth to continue.

(b) If a crystal is subjected to a greater pressure in one direction than in others, the surfaces that are under the lesser pressure will tend to limit the concentration of the solution and prevent it from becoming supersaturated with respect to the surfaces that are rendered more soluble because of the greater pressure. In order that growth may also take place on the surface subjected to the greater pressure, some means must be provided for maintaining the supersaturation of the solution in contact with them.

(c) In porous materials the solutions in subcapillary pores are not readily nucleated. Therefore the solution in such pores will be supersaturated with respect to crystals growing in larger pores. Hence the crystals in the larger pores can receive material from the smaller pores, can grow at the interface between the liquid and the crystal and can exert pressure during this growth.

Now, what is the applicability of the above to expanding concrete? Certainly, in concrete, we have a wide range of capillary and subcapillary pores and the suggested mechanism of growth under pressure would be applicable.

But there is a need for much more basic and applied research on crystal growth under pressure, to be able to answer some puzzling questions.

Consider two cylinders of expanding concrete, of equal expansive potential, one free to expand in all directions and the second restrained in the longitudinal direction. What effect does the longitudinal restraint have on the transverse expansion? Using a continuum mechanics reasoning we would expect an "inverse" relation between longitudinal and transverse strains. A longitudinal shortening will create, due to Poisson's effect, a transverse elongation. However, do crystals, growing under pressure, behave in this way? There is scattered experimental evidence in the literature that, the reverse of continuum mechanics expectation, a "direct" relation exists between longitudinal and transverse expansions.

According to Klein, Karby and Polivka(39) (1961): "There is an inherent and as yet unexplained characteristic of expansive concretes of great significance in future applications. Although precise determinations have not yet been made at the University of California, there is considerable evidence that in small specimens with symmetrical restraint the expansive movements in all directions are substantially equal per unit length of concrete even under uniaxial restraint, where the cross section normal to the axis of restraint is under adequate compression. This characteristic is cited repeatedly in Russian literature on expansive cements, although without pertinent supporting data. The ability to achieve three-dimensional restraint of concrete with uniaxial steel restraint could open realms of application not possible with mechanical prestressing, even though the expansion may not be equidimensional in relatively large specimens." Work carried out since 1961 has not realized the promise of achieving three-dimensional concrete restraint with uniaxial steel restrain but the reference to the "direct" relation is interesting. The effect of size is probably due to curing differences.

Al-Jashami(40) (1964) tested two spirally reinforced columns made of expanding concrete. The longitudinal reinforcement was bounded along its length and had end restraining plates. Electric resistance strain gages were placed on the spiral and longitudinal steel at three levels along the column length, at mid-height and at 1/6th of the length from each end. The average strains due to chemical prestress are shown in Table 2, at 37 days, just before testing. The spiral strains at mid-height are much larger than at either end. This is probably due to the additional lateral restraint provided by the end plates. What is relevant to our discussion here is the significant increase, an average of 15%, of the longitudinal strain at mid-height. Again, this is an indication of the "direct" relation.

Further evidence of the "direct" relation is given by Aroni.(74) A comparison was made of the expansion of similar specimens, some with uniaxial, longitudinal, restraint only and others with triaxial, longitudinal and lateral, restraint made with three different aggregate types, namely expanded shale, crushed granite, and river gravel. When the lateral restraint was absent an increase was also observed in the longitudinal

TABLE 2. AVERAGE STRAINS, DUE TO SELF-STRESS, BEFORE TESTING OF SPIRALLY REINFORCED COLUMNS
(Al Jashami (40))

	Column Y (Element 15 in Table 6)				Column Z (Element 16 Table 6)			
LOCATION	LONGIT. STRAIN (%)	SPIRAL STRAIN (%)	LONGIT. STRAIN (RATIO OF TOP STRAIN)	SPIRAL STRAIN (RATIO OF TOP STRAIN)	LONGIT. STRAIN (%)	SPIRAL STRAIN (%)	LONGIT. STRAIN (RATIO OF TOP STRAIN)	SPIRAL STRAIN (RATIO OF TOP STRAIN)
TOP	0.138	0.069	1.00	1.00	0.135	0.053	1.00	1.00
MIDDLE	0.1575	0.1055	1.14	1.53	0.156	0.120	1.16	2.26
BOTTOM	0.138	0.0705	1.00	1.02	0.127	0.055	0.94	1.04

expansion of up to 25%. A significant difference was found between the aggregate types, the expanded shale showing no such increase.

In view of the above, it seems desirable for future research work to investigate more closely this question and the whole problem of crystal growth under pressure. In the meantime we must be cautious in applying the continuum mechanics reasoning to expanding concrete. This was done, for example in the work by Bertero and Iragorry Montero.(41) They observed an average maximum longitudinal expansion of 0.15% in control specimens with uniaxial external longitudinal restrain (specimen 14 in Table 3). At the same time, structural elements of the same material, cross-section and amount of longitudinal steel (elements 12 and 13 in Table 6) gave an average maximum expansion of 0.184%. The structural elements were of different length, the longitudinal steel was bonded and had end restraints and the elements had closely spaced ties providing additional lateral restraint. The increase in longitudinal expansion was explained by the statement(41) that "this difference was expected because the existence of closely spaced ties in the structural elements did not allow the concrete to expand freely in the lateral direction and therefore forced it to expand more longitudinally." It seems that other reasons for their observation might have been the difference between external and bonded steel as well as the difference in length.

4. PROPERTIES OF EXPANDING CONCRETE

The factors controlling the properties of expanding concrete are numerous and their influence complex. So far, they have been studied in an experimental, empirical way mostly one at a time. Future research work will have to establish most of the interactions between them. In the meantime care is necessary in interpreting and extrapolating any set of data and in comparing different test results.

Most of the tests reported here have been performed at the University of California, at Berkeley, with U.C. expansive cement. The only exceptions are some tests carried out at the Portland Cement Association Laboratories, with U.C. expansive cement(42,43) and with Russian type cement,(42) made by mixing portland cement with gypsum and calcium aluminate cement. Also, one investigation(45) was carried out at the Cement and Concrete Association Laboratories, in England, using expansive cement made and supplied by U.C. Most of the tests were with cements of the "self-stressing" type. A summary of the various specimens used in the different investigations is given in Table 3. The properties and factors investigated with each specimen type are indicated as well as the type of restraint and aggregate used.

We shall next discuss a number of important properties and subsequently consider separately some of the factors affecting them.

TABLE 3. SPECIMENS USED IN EXPANDING CONCRETE INVESTIGATIONS

Nº	REFERENCE NUMBER	SIZE OF SPECIMEN	TYPE OF RESTRAINT	LOCATION OF RESTRAINING STEEL	AGGREGATE TYPE	PROPERTIES AND FACTORS INVESTIGATED
1.	18, 52	2×2×9¼ IN.	U	EXTERNAL	RIVER GRAVEL	STRENGTH, C_3A CONTENT, FINENESS OF EXPANSIVE COMPONENT
2.	18, 47, 48	6×6×19¼ IN. (CENTRAL HOLE)	U	EXTERNAL	RIVER GRAVEL CRUSHED GRANITE	DEGREE OF RESTRAINT, EFFECT OF AGE, TYPE OF AGGREGATE
3.	53, 39 18	2×2×12 IN.	U F	EXTERNAL	RIVER GRAVEL	CREEP, CURING TEMP., AMOUNT OF EXPANSIVE COMPONENT, WATER-CEMENT RATIO
4.	49	12×12×110 IN. 6×6×9¼ IN.	U U	EXTERNAL "	RIVER GRAVEL "	SIZE EFFECT
5.	39	3×6 IN. CYLINDERS	F B	EXTERNAL	RIVER GRAVEL	CURING CONDITIONS, AMOUNT OF EXPANSIVE COMPONENT, COMPRESSIVE STRENGTH
6.	42	3×6×15 IN. SLABS	B	INTERNAL (WELDED WIRE FABRIC)	RIVER GRAVEL	FREEZE-THAW RESISTANCE
7.	43	1×1×6 IN. 2×2×10 IN.	F U	INTERNAL WITH END RESTRAINT	NATURAL SAND	COMPOSITION, SIZE EFFECT, WATER CEMENT RATIO

TABLE 3. CONT'D SPECIMENS USED IN EXPANDING CONCRETE INVESTIGATIONS

Nº	REFERENCE NUMBER	SIZE OF SPECIMEN	TYPE OF RESTRAINT	LOCATION OF RESTRAINING STEEL	AGGREGATE TYPE	PROPERTIES AND FACTORS INVESTIGATED
8.	45	1¾×1¾×3⅞ IN 1¾×1¾×8 IN	U	EXTERNAL	RIVER GRAVEL	EXPANSION HISTORY, WATER-CEMENT RATIO, COMPRESSIVE STRENGTH
9.	40	6×12 IN CYLINDERS	T	EXTERNAL	RIVER GRAVEL	TRIAXIAL RESTRAINT
10.	50	3×2·6×6 IN	U	EXTERNAL	RIVER GRAVEL	COMPRESSIVE STRENGTH
11.	50	3×5·4×21 IN (CENTRAL HOLE)	U	EXTERNAL	RIVER GRAVEL	MODULUS OF RUPTURE
12.	50	3×5·15×21 IN	U	INTERNAL WITH END RESTRAINT	RIVER GRAVEL	MODULUS OF RUPTURE

TABLE 3. CONT'D SPECIMENS USED IN EXPANDING CONCRETE INVESTIGATIONS

Nº	REFERENCE NUMBER	SIZE OF SPECIMEN	TYPE OF RESTRAINT	LOCATION OF RESTRAINING STEEL	AGGREGATE TYPE	PROPERTIES AND FACTORS INVESTIGATED
13.	41	4×2⅞×12 IN.	U	EXTERNAL	RIVER GRAVEL	COMPRESSIVE STRENGTH
14.	41	4×2⅞×45 IN.	U	EXTERNAL	RIVER GRAVEL	MODULUS OF RUPTURE
15.	51	4⅞×6×16 IN.	U	INTERNAL WITH END RESTRAINT	RIVER GRAVEL	COMPRESSIVE STRENGTH, MODULUS OF RUPTURE, SONIC MODULUS OF ELASTICITY, CEMENT CONTENT, AMOUNT OF EXP. COMPONENT
16.	52	2×2×12 IN. 6×18 IN. CYL. (CENTRAL HOLE)	F U	EXTERNAL	RIVER GRAVEL	CREEP
17.	74	2×2×11½ IN. 6×18¾ IN. CYL. (CENTRAL HOLE)	F U T	EXTERNAL	RIVER GRAVEL, CRUSHED GRANITE, EXPANDED SHALE	TYPE OF AGGREGATE, TYPE OF RESTRAINT

NOTES:

F = FREE, NO RESTRAINT
B = BIAXIAL RESTRAINT
U = UNIAXIAL "
T = TRIAXIAL "

(a) Expansion

Since it is the expansion which distinguishes expanding concrete from conventional material, its importance is obvious. Expansion is both an important property, determining the prestress developed in the concrete under restraint, and also a factor influencing other properties, strength properties in particular. The "strength-expansion" relation has already been discussed earlier.

The potential expansion magnitude depends on the composition of the expansive cement (chemical composition, percentage of expansive component and its age) and the concrete mix details (cement content, type of aggregate.) The actual expansion achieved at any given age depends on the rate and history of expansion. These in turn are functions of the expansive cement (fineness in particular), curing conditions (temperature, availability of water) and type and degree of restraint provided (uniaxial, biaxial or triaxial.) Curing can be both external and internal by creating a reservoir of available water using porous aggregate. Also, in addition to possible external restraint, internal, local restraint most probably exists in free specimens. The differences in local curing and restraint could be the main reason for observed shape and size effects and possible disruptive forces.

The large water retention (11% by weight) of the expanded shale aggregate used in the tests reported by Aroni(74) was shown to produce higher early age expansion and to result in continued expansion for some time after exposure to drying.

The length of time to reach the completion of the expansion process, even under water curing, depends on many variables. In Klein, Karby and Polivka's tests(39) the expansion appeared to be practically complete after seven days. On the other hand, Dewar(45) observed, with both free and uniaxially restrained specimens, little or no expansion during the period from four to seven days, whereas after this period expansion was again observed and continued for nearly two months. It is interesting to note that during the four to seven days, when virtually no expansion took place, there was, nevertheless, a measured increase in weight indicating water absorption and possible production of the trisulfate hydrate. Dewar refers to the observed behavior as a two-stage expansion process and states that "the reasons for its occurrence have not as yet been explained satisfactorily."

The expansion curves obtained by Aroni(74) on free specimens seemed to be composed of a number of S-shape curves fitted together. Based on this experimental evidence Aroni proposed a "multiple-stage expansion" theory.

He stated that "it is evident that expansion is not uniform, it goes through a number of stages (S-shape segments) of positive and negative second derivatives (rates of change of expansion rate.) Initially, after placing, the expansion accelerates with respect to the formation of the strength-giving tobermorite gel. As the tobermorite structure is developed, the internal flow of water becomes restricted and the formation of the trisulfate hydrate is slowed down. Also the growing expanding crystals find themselves under increasing local restraint from the tobermorite structure.

All this leads to the reduction of the rate of change of the expansion rate and a flattening off of the expansion curve (top part of the S-shape segment.) The increasing pressure, from the locally restrained crystals, on the relatively weak tobermorite structure, still not fully developed, leads to its eventual disruption. This results in a revitalized expansion due to both the temporary decrease of the local restraint and the better flow of water in the disrupted structure. However the hydration of tobermorite is also accelerated and the structure tends to repair itself (autogenous healing) thus once again reducing the rate of expansion. Expansion thus occurs in a number of stages or cycles each tending to produce an S-shaped expansion-time curve. The greater the availability of internal water, the larger the expected number of such stages. This is demonstrated by the described expansion curves.

It is believed that this 'multiple-stage expansion' behavior is basic to expanding concrete and is therefore present in restrained specimens as well as externally free ones. With external restraint, however, the expansion-time curve is governed by additional factors which overshadow the S-shape tendency. Nevertheless, an examination of the plotted curves, for restrained specimens, still shows some signs of such a tendency."

Klein and Bresler(17) distinguish, in expanding concrete, three stages of expansion, each with its particular important features:

(a) first stage, from 1 to 24 hours. The specimen is still in the mold, much chemical reaction takes place, water loss has to be prevented and precise measurements of expansion and prestress are difficult.

(b) second stage, from 2 to up to 60 days. This is from time of demolding to an essential termination of expansion, preferably uniformly throughout the specimen. It is during this stage that the rate and amount of expansion has a major influence on the concrete mechanical properties.

(c) third stage, normal service conditions, usually air storage. Shrinkage takes place, depending on many factors (relative humidity, previous curing, details of expansive cement and mix proportions, type of aggregate.)

A regression computer program was used by Aroni(74) to obtain analytic expressions for expansion-time curves of best fit, minimizing the sum of squares of the differences between experimental and analytical values. Equations, in terms of exponentials and polynomials, are given for both longitudinal and lateral expansions of free and triaxially restrained specimens. The selection of a suitable analytic expression, before the calculation of the unknown coefficients by regression, is still a matter of judgement.

Theoretical work, aimed at predicting restrained expansion, was described by Bertero and Bresler.(46) A highly idealized model was used, in view of the yet inadequate understanding of the physico-chemical processes involved. The calculated strain (expansion) was assumed to be given by the free expansion (as determined from tests) plus elastic strain plus creep (based on an assumed variation with time of the modulus of elasticity and of the creep compliance.) The theoretical assumptions included a linear stress-deformation (elastic and creep) relation and the important simplification of neglecting the effect of the stress history on the modulus of elasticity, the creep compliance and the free expansion. Nevertheless, certain significant correspondence was observed between the calculated and the experimental(47) expansion under uniaxial restraint versus time curves.

(b) Mechanical Properties

Compressive Strength. -- Many references (18,39,41,43,45,47,48,49,50,51,52,74) report compressive test results of expanding concrete, either as a primary or a secondary property under investigation. The many factors influencing strength will be discussed separately but of major interest here is the effect of expansion. In the previous "strength-expansion" relation discussion mention was made of the disruptive effect of expansion. We might expect that the larger the expansion, the lower the specimen strength. This is usually the case but there are exceptions, probably due to interactions not yet fully investigated.

Bertero and Polivka(47,18) compared compression test results of similar specimens, same age, water curing and mix details, but different uniaxial restraint. The longitudinal expansion decreased with increase in restraint for the full range investigated. The lateral expansion behaved similarly except for the largest restraint when an increase in lateral expansion was observed. At the same time the compressive strength, which kept increasing with the degree of restraint, decreased significantly at the largest restraint. This was explained as being due to the damage caused by the increased lateral expansion. A much more striking example are the tests reported by Bertero.(48)

He tested similar, uniaxially restrained and water cured, specimens at different ages. In one series, the compressive strength was about 4,500 psi, at 5 and 12 days and dropped to 2,400 psi at 60 days. The drop of strength, which started at ages of 12-16 days, was explained by the considerable increase in the rate of the transverse expansion observed at the same time. This lateral expansion exhibited a two-stage behavior, similar to the one observed by Dewar.(45) Since more than 50% of the total expansion occurred after 12-16 days, when the material was fully hardened, its disruptive effect is not surprising. A similar tendency is seen in Wong's tests(50) (3690 psi at 7 days, 3340 psi at 21 days.) However, among four similar uniaxially restrained specimens, tested at the same age by Bertero and Irragory Montero(41) and discussed by Bertero and Bresler,(46) one developed a somewhat higher level of longitudinal expansion and prestress and at the same time had a higher compressive strength.

Aroni(74) suggested that the availability of internal water, as in the case of lightweight expanded shale aggregate, will lead to a relative improvement of strength. In terms of the "multiple-stage expansion" theory, this leads to a greater number of stages, i.e., to a structure which has the ability to "repair itself" a greater number of times. Differential expansion will also be reduced. Aroni showed a comparison of compressive strengths of similar triaxially restrained specimens made with expanding concrete and portland cement concrete and with three types of aggregates. The relative strength of expanding concrete with the lightweight aggregate was significantly higher than that with crushed granite or river gravel.

Modulus of Elasticity. -- Lum(51) reported sonic measurements of modulus of elasticity. The value of E was found to increase with age and richness of mix and decrease with expansion. This relation was similar to the compressive strength. The same detrimental effect of expansion on E was reported by Dewar.(45) The above mentioned Wong's specimens had an E of 2.00×10^6 psi at 7 days and 1.75×10^6 psi at 21 days.

Aroni(74) reported average values of secant modulus for expanding concrete specimens of 2.5, 3.7, and 3.95×10^6 psi with expanded shale, crushed granite, and river gravel aggregate respectively. These values were lower than for similar portland cement concrete specimens. A comparison was also given of E values for triaxially and uniaxially restrained specimens with the restraint removed just before testing. The beneficial effect of triaxial restraint was reflected in increases of E between 22 and 35% for all three aggregate types.

Stress-Strain Relation. -- Systematic studies of stress-strain relations, in both compression and flexure, have not yet been reported. However stress-strain curves in compression are given in some of the studies reporting on compressive strength. The two types of compression tests that can be performed, with and without prior release of the prestress, are discussed by Bertero and Bresler.(46) They also suggest the use of stress-accrued strain curves. Accrued strain is defined as the total strain of the specimen history, including expansion and shrinkage under restraint, strain on possible release and subsequent external loading to failure.

Such curves are advantageous in presenting the full stress-strain history of specimens.

Aroni's(74) compression tests indicate that the strain at failure depends on the type of aggregate used but is not affected by the restraint history (uniaxial or triaxial.)

Creep. -- Creep studies have been reported by Klein and Bertero.(53) They conclude that expanding concretes in the range studied (uniaxial restraint, water cement ratio of 0.31, 25% and 30% expansive component, maximum expansion below 0.6%) may be expected to exhibit ultimate creep strains due to external load equal to or considerably less than those of conventional concretes subjected to the same ratio of sustained stress to ultimate stress at the same ages of loading. The reason for the lower creep deformations of expanding concrete is their "pre-creep" characteristic, i.e., the existence of creep during the expansion and self-stressing period.

Another example of pre-creep can be found in tests conducted by Klein, Karby and Polivka.(39) They subjected uniaxially restrained set of specimens to a curing cycle, at 70° F, of fog curing up to 7 days, drying at 50% relative humidity to 70 days and fog curing again till 90 days. The original dimensions and stress conditions were regained during the second fog curing period, indicating that practically all of the creep had been taken out of the concrete.

Anastas(52) performed creep tests of uniaxially restrained specimens, stored under water and subjected to sustained loads at 28 days, corresponding to 0.20, 0.30, 0.40 and 0.50 f'_c. He concluded that no proportionality exists between creep and load and that the superposition theorem cannot be applied to creep of expanding concrete under any stress level. The observed creep diminished rapidly and practically stopped after about 9 days. This was much faster than with normal concrete.

Bond Strength. -- Very low bond strength was observed by Tso(54) in three beam tests of expanding concrete (elements 9, 10 and 11 in Table 6) with only longitudinal high-tensile steel. This was probably due to lateral expansion with lack of lateral restraint. We would anticipate high bond strength when lateral restraint is available, particularly for circumstances when frictional phenomena predominate. Polivka and Klein(55) performed pull-out tests on a 0.196 in. dia. high-tensile strength wire grouted inside a 1-1/4 in. dia. steel tube. The bond strength was 1.52 and 2.20 times higher for grouts containing 20% and 30% expansive component respectively compared with a grout containing 20% fly ash and 0.03% aluminum powder. Here frictional phenomena predominated. Bertero and Iragorry Montero(41) observed very good bond between high-strength deformed reinforcement and expanding concrete (elements 12 and 13 in Table 6). This was attributed to the lateral restraint offered by the closely spaced ties.

Durability: Freeze-Thaw and Surface Scaling. -- Freeze-thaw durability tests were conducted at the P.C.A. Laboratories(42) on 112 slabs (specimen 6, Table 3). The variables included:

(a) Four types of expansive cements:

U.C. shrinkage-compensating cement with 12% E.C.

U.C. self-stressing cement with 20% E.C.

Comparable two kinds of Russian-type cement

(b) Four levels of internal biaxial restraint (0-1.77% steel)

(c) Two curing methods: water curing and curing by sealing with polyethylene sheets.

The conclusions reached were:

1. The behavior of U.C. and Russian-type cements was similar.

2. The shrinkage-compensated concretes behaved better than the self-stressing ones. The shrinkage-compensated concretes were subjected to 300 cycles of freezing and thawing in water or 300 cycles of de-icer application without serious deterioration. The self-stressing concretes were susceptible to large volume changes. They required considerable reinforcement to withstand 300 freeze-thaw cycles and none of them survived 300 cycles of de-icer application.

3. Increase in restraint improved the freeze-thaw resistance.

(c) <u>Factors Influencing Properties</u>

(1) <u>Factors Related to the Cement</u>

<u>C_3A Content.</u> -- Restrained expansion is inversely proportional to the C_3A content of the portland cement.(18) The reason is the increased solubility of the trisulfate hydrate and the increased tendency of the formation of non-expansive monosulfate hydrate with increase of C_3A. Klein and Troxell(11) found no significant expansion with a portland cement having a C_3A content of 12%.

Proportion of Expansive Component (E.C.). -- Generally speaking, expansion increases with increase in the amount of expansive component, but not proportionately.(18,39) There is probably an interaction with the degree and type of restraint and some contradictory evidence exists, particularly for uniaxial restraint. Thus, Klein, Karby and Polivka(39) report a drop of self-stress from 760 psi to 500 psi when the percentage of E.C. was increased from 23 to 26%. Lum(51) reports drops in compressive strength and modulus of rupture with increase in percentage of expansive component. However, according to Klein, Karby and Polivka(39) the compressive strength of unrestrained concretes appears to be related primarily to the expansion rather than to the amount of expansive component used, provided the cement factor and water-cement ratio are kept constant.

Fineness of Expansive Component. -- The fineness of the expansive component is a major factor influencing the expansion characteristics of the concretes,(18,11) particularly restrained expansions which are of primary interest. On the other hand, the fineness of portland cement had no effect on expansion.(18) An increase in fineness of expansive component increases the rate of trisulfate hydrate formation and therefore the rate of early age expansion. However, for the same reason, the final total expansion and the potential prestress level are reduced.

Age of Expansive Component. -- Aging of expansive component tends to reduce the expansive potential. The expansive cements are in general more sensitive than portland cements to atmospheric effects such as moisture and carbon dioxide.(17) Bertero(48) presents some experimental evidence showing reduction of early age expansion with increase in age of expansive component.

(2) Factors Related to Mix Details

Water Cement Ratio. -- The evidence(39,42,43) is that in general the lower the water-cement ratio of the mix, the greater the expansion of expanding concrete, but in some cases the relationship between water-cement ratio and expansion is not clearly defined. Water-cement ratios as low as 0.16 are reported to be used in Russia in pneumatically applied expanding mortars. A number of reasons have been suggested for the observed behavior:

(1) Low water-cement ratio will increase the concentration of calcium ions which is conducive to trisulfate hydrate formation and consequent expansion.

(2) Low water-cement ratio will result in low permeability which impedes entrance of water particularly after 24 to 48 hours. This will reduce expansion and, if this factor is operative, it is apparently not as strong as the factors leading to increased expansion.

(3) High water-cement ratio will increase the internal water-filled volume. This will provide more room for internal expansion and will reduce the observed external expansion.

(4) High water-cement ratio might be expected to increase the amount of trisulfate hydrate formed during storage in the molds, and consequently to decrease the amount formed during subsequent curing in water and therefore the expansion.

How is strength of expanding concrete affected by water-cement ratio? Low water-cement ratio leads to high expansion and this will tend to decrease the strength. However, apparently,(39) the decrease is more than compensated by the higher strength of the tobermorite gel. Thus strength of expanding concrete also increases with decrease of water-cement ratio.

Cement Content. -- Test results(39) indicate that an increase in cement content increases the expansion and self-stress of uniaxially and biaxially restraint specimens. Increase in cement content was also shown(51) to increase both the compressive strength and modulus of rupture of specimens with uniaxial restraint.

Type of Aggregate. -- In conventional concrete, the effect of aggregate is well known. In expanding concrete, two important additional effects are present:

(1) the stiffness, size, shape and surface texture of the aggregate influence the internal restraint against expansion.

(2) the porosity and water retention capacity of the aggregate (such as lightweight expanded shale aggregate) could significantly affect the internal curing and therefore the expansion and shrinkage characteristics.

Polivka and Bertero(18) describe comparative tests of uniaxially restrained concrete specimens with two types of coarse aggregate of two maximum sizes. Table 4 shows the average test results.

Table 4. Effect of Shape and Size of Aggregate on Expansion and Compressive Strength of Uniaxially Restrained Concretes (Polivka and Bertero(18))

Mix Details: 6-1/2 scy; E.C.: 2 scy, 31%.

Aggregate		Expansion, percent		Compressive Strength psi
		Longitudinal	Lateral	
Type	Max Size, In.	Average of three specimens (sp.2, Table 3) with 1.76% steel, after 28 days of moist curing.		
River Gravel (rounded)	5/8	0.21	0.98	2510
	1	0.29	1.26	2040
Crushed Granite (angular)	5/8	0.24	0.72	3340
	1	0.28	0.80	3150

The following conclusions were reached:

(1) The two concretes containing the angular crushed granite aggregate exhibited similar longitudinal expansions but lower lateral expansions than those of the corresponding concretes with rounded river gravel. This was attributed to the higher internal restraint provided by greater bond developed by the angular and rough textured crushed aggregate.

(2) The crushed granite concretes had considerably higher compressive strength than the river gravel ones. This was attributed to the smaller lateral expansion.

(3) The concretes with 1 in. maximum size aggregate had higher expansions and lower strength. This could be due to the lower surface area of the aggregate and thus lower internal restraint.

The investigation of the effect of aggregate type on the properties of expanding concrete was the principal aim of Aroni's(74) tests. He used three types of aggregates: lightweight expanded shale (maximum size 5/8 in.), crushed granite, and river gravel (both with maximum size 3/4 in.). The experimental results demonstrated the advantageous use of high porosity aggregate. The internal distribution of water caused the early-age expansion to be a higher percentage of the eventual total expansion than with regular types of aggregate. This, together with a probable reduction of moisture and expansion gradients in the specimens and better internal curing, increased the compressive strength of the material. Also, better behavior on drying was observed, the internal water reservoir causing continued expansion and compensating for more of the shrinkage. However, for the application under discussion, the low strength of the lightweight aggregate and its brittle type of failure were undesirable features. The ideal aggregate was thought to be one with high porosity and water retention coupled with the strength and stiffness of normal aggregate. Lightweight aggregate has also been used with shrinkage-compensating expansive cement.(58)

Additives. -- Expanding concretes exhibit more rapid setting characteristics than do corresponding plain concrete mixes. Polivka and Bertero(18) report initial and final set of expanding concrete was reached about 25% earlier than for portland cement concrete. They also give test results with three types of set-retarding admixtures, all of which increased the time of set beyond the plain concrete values. Apparently the use of these set-retarding admixtures had no significant influence on the concrete expansion characteristics. According to Klein, Karby and Polivka,(39) a retarding water-reducer appeared to affect the expansive characteristics of the concrete only through the resulting reduction of water-cement ratio, which improved the strength and to some degree the expansion.

An interesting observation was made(14) on an experimental pavement section at Antelope Valley Freeway, in California. Half of the units in the test section had air-entrainment and half did not. The average observed distance between cracks for the air-entrained units was 69% of the distance for the non-air-entrained units. Apparently, under the conditions existing for this test pavement, the air-entrained concrete may have had substantially greater drying shrinkage.

(3) Factors Related to Hardening Conditions

Curing. -- The significance of curing, the supply of water for hydration, is well established in portland cement concretes. In expanding concrete it is of even greater importance. Here we are dealing with two hydration reactions, the formation of the strength-giving tobermorite and of the expansion-causing trisulfate hydrate, and these are affected differently by curing conditions, temperature and relative humidity. The trisulfate hydrate formation is the more rapid of the two and curing conditions can be used as a meansof control in the "strength-expansion" relation. The Russian hydrothermal curing, mentioned earlier, represents an example of this. It has also been suggested(39) that an effective method of controlling the rate of expansion in concretes where high expansions are desired is to restrict early curing, prevent evaporation with a membrane cover, and allow time for development of adequate strength to resist the expansion on subsequent curing.

The effect of curing temperature on the rate and magnitude of expansion in both free and uniaxially restrained specimens has been investigated(53) and reported.(18) The results are shown in Table 5.

Table 5. Effect of Curing Temperature on Expansion Characteristics of Concretes
(Polivka and Bertero,(18) Klein and Bertero(53))

Details: 9-3/4 scy; E.C. = 25%; water-cement ratio = 0.30; water curing; Specimen 3 of Table 3

Curing Temperature °F	Free Specimens		Uniaxially Restrained, Steel Ratio = 1.1%	
	Maximum Stabilized Expansion			
	Days of Curing	Percent	Days of Curing	Percent
45	200	0.78	150	0.43
70	16	2.17	17	0.42
100	12	2.72	25	0.26

The temperature of curing is seen to have a significant influence on both the rate and magnitude of expansion. The higher the temperature, the higher the rate of expansion, for both free and restrained specimens, and the higher the total free expansion. For uniaxially restrained specimens, the total expansion, and therefore the self-stress, decreases with increase in curing temperature. The reason lies in the different rates of expansion with different temperatures. With a high temperature, there is very rapid early expansion, while the concrete is still plastic, and consequantly less total maximum expansion. The compressive strength probably increases with the curing temperature, being inversely proportional to late expansion. Some optimum temperature probably exists with respect to the magnitude of self-stress and the compressive strength.

The beneficial effect of internal curing with lightweight aggregate in Aroni's tests was mentioned earlier.

<u>Degree and Type of Restraint</u>. -- Restraint is vital for expanding concrete, first to limit the disruptive expansion and, secondly, to utilize the expansion and induce prestress. The amount of restraint is a crucial design parameter. It determines the stress induced and the effective strength of the concrete. The type of restraint, uniaxial, biaxial or triaxial is also most important. Most of the investigations to date used uniaxial or biaxial (mostly in slabs) restraint. The only specimens reported with triaxial restraint have been some cylinders reported by Al-Jashami(40) (specimen 9 of Table 3), and Aroni's(74) specimens (specimen 17 of Table 3.) A number of structural elements, pipes,(56) beams with stirrups(41) and columns,(40) were designed with triaxial restraint.

The investigation of uniaxial restraint by Bertero and Polivka(47,18) was mentioned in discussing compressive strength. The amount of restraining steel ranged from 0.22 to 6.47 percent and the conclusion was reached that there exists an optimum percentage of longitudinal restraint as far as the mechanical properties of the concrete are concerned. This optimum was estimated to be between 1 and 2 percent. Also, it was concluded that for successful use of expanding concrete in some structural elements, triaxial restraint is necessary.

This conclusion is now generally shared. Bertero(48) demonstrated the damaging effect of lateral expansion and the large deterioration in strength that can occur with age. Anisotropy, non-homogeneity and detrimental lateral expansion(49) increase with the size of specimen. Except for two dimensional elements, like slabs and shells, where biaxial restraint might be satisfactory, triaxial restraint will be required to eliminate or minimize the above detrimental features.

There is probably an optimum triaxial restraint in the complex relation of expansion-strength-restraint and this will have to be investigated in future work.

There is a certain difficulty, requiring care, in the determination of mechanical properties of triaxially restrained expanding concrete. To determine the compressive strength of the material, the triaxial restraint has to be removed before testing and the danger exists of damaging the material in the process. While no conclusive evidence exists, there are some indications for it in the experimental work described by Aroni(74). Also, Al-Jashami(40) obtained an ultimate strength of 6,350 psi for a released, triaxially restrained cylinder, while the shell in the companion columns (elements 15 and 16 in Table 6) failed at 6,500 and 6,660 psi. The difference is made more significant by the fact that the shell was only uniaxially restrained. No evidence exists for damage on release of uniaxially restrained specimens.

Size Effect. -- Since curing normally proceeds from the surface of a specimen, the size of the specimen, which determines the ratio of surface area to volume, would be expected to influence the history of expansion and therefore the mechanical properties. In large specimens a moisture gradient will be set up and therefore an expansion gradient, which may originate internal stresses detrimental to the mechanical properties of the concrete.

This was confirmed experimentally by Bertero(49) in tests on uniaxially restrained specimens. He found that:

(1) Size affects the history but not the ultimate magnitude of longitudinal expansion, but it affects both the ultimate magnitude and history of lateral expansion.

(2) An increase in size

- decreases initial expansion and increases later lateral expansion
- increases the time needed for expansion stabilization
- increases the anisotropy and non-homogeneity of the material.

Monfore(43) also found a decrease, at an early age, of the expansion of mortar prisms with increase in size.

Most of the detrimental effects mentioned above should be eliminated with triaxial restraint and internal curing provided by a porous aggregate.

5. STRUCTURAL TESTS

A total of eighteen tests on structural elements made of expanding concrete have been reported in the literature(40,41,50,54,56,57). These include tests on four pipes, four slabs (two one-way, two two-way), five beams, two frames, two columns and one hyperbolic paraboloid. The details of the structural elements and the test results are summarized in Table 6. All elements were made with expanding concrete containing U.C. expansive cement. All tests were performed at the University of California.

Elements 1 and 2 exhibited some objectionable bond slip phenomena, particularly at the top of the pipes. This was removed by the provision of end anchorage and expanded-metal rings at the ends, and no such difficulty occurred in elements 3 and 4.

Element 5 is a one-way slab, only lightly reinforced, with conventional steel and without end anchorage in the transverse direction. It was apparent, however, that the bond in the transverse direction did reduce to some extent the expansion in that direction. The transverse expansion was only about 60% greater than the longitudinal one, whereas other test data, with longitudinal restraint only, indicate transverse expansions 80-100% greater than those in the direction of restraint.

Element 6 is a two-way slab with equal reinforcement both ways. The observed prestress was also equal in the two directions and a highly symmetrical crack pattern was obtained at failure.

TABLE 6. CHEMICALLY PRESTRESSED STRUCTURAL ELEMENTS

Nº	REFERENCE NUMBER	ELEMENT	DESCRIPTION	STEEL REINFORCEMENT	STEEL ANCHORAGE	% STEEL	W/C BY WT.	% E.C.	CEMENT CONTENT SCY.	AGGREGATE TYPE	CURING	AGE AT TEST DAYS	MAX. CONCRETE COMPRESSIVE PRESTRESS PSI	CONCRETE COMPRESSIVE PRESTRESS AT TEST PSI	MAX. STEEL STRESS PSI	STEEL STRESS AT TEST PSI	PREDICTED CRACKING LOAD LB	ACTUAL CRACKING LOAD LB	PREDICTED ULTIMATE LOAD LB	ACTUAL ULTIMATE LOAD LB
1.	56	PIPE	INTERNAL DIAM.: 12" WALL THICKNESS: 1.4" LENGTH: 36"	CIRCUMFERENTIAL: 1/8" WIRE, 1" PITCH LONGITUDINAL: 12 AT 1/4", 7 WIRE STRANDS	INTERNAL BOND	0.89 0.68	0.325	30	10	RIVER GRAVEL 1/2" MAXSIZE	0-30 DAYS: FOG 30-90 DAYS: 70 F. 50% RH	90	*1 700 (820)	*2 530	*3 78,000 (122,000)	—	*4 300 psi	*5 360 psi	24,000	*6 23,800
2.	56	PIPE	INTERNAL DIAM.: 12" WALL THICKNESS: 2" LENGTH: 36"	CIRCUMFERENTIAL: 1/4" WIRE, 3" PITCH LONGITUDINAL: 6 AT 3/8", 7 WIRE STRANDS	"	0.88 0.65	0.31	30	10	"	"	90	*1 900 (500)	*2 810	*3 104000 (80,000)	—	—	—	55,200	*6 48,500
3.	56	PIPE	INTERNAL DIAM.: 14" WALL THICKNESS: 2" LENGTH: 36"	CIRCUMFERENTIAL: 0.196" WIRE, 2½" PITCH LONGITUDINAL: 12 AT 1/4" WIRE + SIX Nº 3 RODS	END ANCHORAGE FOR 1/4" DIAM. STEEL. ALSO EXPANDED METAL RINGS AT BOTH ENDS.	0.60 0.55	0.31	30	10	"	0-20 DAYS: FOG 20-60 DAYS: 70 F. 50% RH	60	*1 600 (370)	*2 490	*3 100,000 (66,000)	—	*4 270 psi	*5 220 psi	59,500	*6 53,600
4.	56	PIPE	"	CIRCUMFERENTIAL: 0.196" WIRE, 2½" PITCH LONGITUDINAL: 12 AT 1/4" WIRE	END ANCHORAGE FOR 1/4" DIAM. STEEL	0.60 0.55	0.31	30	10	"	0-50 DAYS: FOG 50-100 DAYS: 70 F. 50% RH	100	*1 640 (500)	*2 530	*3 90,000 (108,000)	—	*4 310 psi	*5 250 psi	65,000	*6 63,500
5.	56 54	SLAB	8'×5'-3¾" 3" THICK	LONGITUDINAL: HIGH-TENSILE 0.276" DIAM. AT 6¾" CENTERS TRANSVERSE: 3/8" DIAM M.S. BARS AT 24" CENTERS	LONGITUDINAL: EXTERNAL ANCHORAGE BOTH ENDS TRANSVERSE: INTERNAL BOND	0.3	0.30	22.2	9	RIVER GRAVEL 3/4" MAX.SIZE	0-14 DAYS: FOG 14-40 DAYS: 70 F 50% RH 40-45 DAYS: FOG	45	375	355	—	*7 120,000 (37,000)	280 psf	300 psf	390 psf	400 psf
6.	56	SLAB	6'-6" × 6'-6" 2" THICK	TWO-WAY EQUAL REINF. 1/4" HIGH TENSION STRANDS AT 3" CENTERS	END ANCHORAGE FOR EACH WIRE	0.59 EACH WAY	0.30	22.2	9	"	0-35 DAYS: FOG 35-90 DAYS: 70 F 50% RH	90	680	560	—	95,000	225 psf	260 psf	472 psf	460 psf
7.	50	SLAB	6'-0"×6'-0" 3" THICK	ONE-WAY REINF. 3/8" HIGH TENSION DEFORMED BARS AT 5.15" CENTERS.	END ANCHORAGE BY WELDING BARS TO END ANGLES	0.715	0.305	30	10	"	0-18 DAYS: WATER 18-23 DAYS: 70 F 50% RH	23	—	410	—	58,000 (yield point)	*8 446 psf	320 psf	551 psf	580 psf
8.	50	SLAB	"	TWO-WAY EQUAL REINF 3/8" HIGH TENSION DEFORMED BARS AT 5.15" CENTERS EACH WAY	"	0.715 EACH WAY	0.305	30	10	"	0-18 DAYS: WATER 18-84 DAYS: 70 F 50% RH	*9 24	—	389	—	58,000 (yield point)	*10 360 psf	*11 ABOVE 680 psf	830 psf	*11 ABOVE 680 psf
												*12 84	—	389	—	58,000 (yield point)	*8 10,220	7,000	6,900	8,750
9.	54	BEAM	CROSS-SECTION: 6"×6" LENGTH: 3'-4"	TWO 0.357" DIAM. HIGH TENSION RODS CENTRALLY PLACED	EXTERNAL ANCHORAGE BOTH ENDS	0.6	0.45	25	10.25	"	0-14 DAYS: FOG 14-44 DAYS: 70 F 50% RH 44-92 DAYS: WATER	92	900	570	—	93,000	6,460	8,000	13,920	*13 11,800
10.	54	BEAM	"	ONE 0.357" DIAM. HIGH TENSILE ROD CENTRALLY PLACED.	"	0.3	0.45	25	10.25	"	0-14 DAYS: FOG 14-79 DAYS: 70 F 50% RH	79	420	280	—	91,000	5,580	5,000	9,500	*14 6,350
11.	54	BEAM	"	ONE 0.357" DIAM. HIGH TENSILE ROD ECCENTRICALLY PLACED 1" FROM BOTTOM AT CENTER OF BEAM	"	0.3	0.45	25	10.25	"	0-14 DAYS: FOG 14-65 DAYS: 70 F 50% RH	65	940	690	—	96,000	8,200	8,000	14,720	*13 11,500
12.	41	BEAM	CROSS-SECTION: 2⅞"×4" LENGTH: 3'-8"	FOUR 1/4" DIAM. DEFORMED BARS (Y.P. = 50,000 psi). 12-GAGE WIRE STIRRUPS AT 1½" ALONG FULL LENGTH.	END ANCHORAGE BY WELDING EACH BAR TO END BEARING PLATES	1.74	0.277	33.3	12	RIVER GRAVEL 1/4" MAX SIZE	0-15 DAYS: FOG 15-46 DAYS: 70 F 50% RH	46	860	679	—	39,000	*15 10.80 Kip-in.	16.76 Kip-in.	19.3 Kip-in.	22.5 Kip-in.
13.	41	BEAM	"	"	"	"	"	"	"	"	"	46	860	622	—	35750	*15 9.80 Kip-in.	14.01 Kip-in.	19.3 Kip-in.	20.60 Kip-in.
14.	41	FRAMES (TWO)	PORTAL MADE UP OF THREE ELEMENTS CONNECTED BY HIGH-STRENGTH STEEL BOLTS. OVERALL: HEIGHT 42" & SPAN 44"	INDIVIDUAL ELEMENTS (BEAM & TWO COLUMNS) SAME AS IN Nº 12 & Nº 13 ABOVE							0-15 DAYS: FOG 15-41 DAYS: 70 F 50% RH	*16 41	INDIVIDUAL ELEMENTS SIMILAR TO Nos 12 & 13 ABOVE				—	*17 2,140 2,140	2,120	2,270 2,400
15.	40	COLUMN	OVERALL DIAM: 12" CORE DIAM: 9" HEIGHT: 60"	LONGITUDINAL: SIX 1/2" DIAM. DEFORMED BARS (Y.P. = 66,000 psi) SPIRAL: 1/4" DIAM. WIRE (Y.P. = 60,000 psi AT 0.2% OFFSET, PITCH = 1.0")	END ANCHORAGE BY BOLTS TO 1¼" THICK END PLATES.	*18 1.04 (2.22)	0.328	30	6	RIVER GRAVEL 3/4" MAXSIZE	FOG AT 70 F	37	—	LATERAL 367	—	*18 55,000 (34,000)	—	*19 6,500 psi	—	*20 12,350 psi
16.	40	COLUMN	"	"	"	"	"	"	"	"	"	37	—	LATERAL 391	—	*18 54,000 (36,200)	—	*19 6,660 psi	—	*20 11,910 psi
17.	57	HYPERBOLIC PARABOLOID SHELL	8' SQUARE PLAN 1½' RISE 0.8" THICK	EQUAL REINF. BOTH WAYS. 0.105" DIAM. HIGH STRENGTH STEEL AT 3" CENTERS	END ANCHORAGE	0.43 EACH WAY	0.30	25	10	RIVER GRAVEL 1/4" MAX. SIZE	0-50 DAYS: WATER 50-100 DAYS: 70 F 50% RH	100	475	420	—	94,000	*21,22 1,051 psf	*21 330 psf	—	*21 570 psf

Table 6. Chemically Prestressed Structural Elements

Notes

*1 Maximum circumferential prestress at center (Maximum longitudinal prestress at center)

*2 Average effective circumferential prestress

*3 Maximum circumferential prestress in steel, at center (Maximum longitudinal prestress in steel, at center)

*4 Theoretical internal pressure at leakage (corresponding to assumed tensile strength of 0.10 f'_c)

*5 Observed internal pressure at leakage

*6 Observed ultimate load in a center-load test (three-edge bearing test)

*7 Longitudinal (Transverse)

*8 Based on control specimen tests

*9 Tested with the four corners held down and loaded with a uniform upward pressure

*10 Theoretical value, based on the cracking moment (from control tests) and the theoretical moment assuming no elastic support from edge beams.

*11 The behavior of the slab was still elastic at an equivalent uniform load of 680 psf, when the pressure bag leaked and the pressure was reduced to zero.

*12 Tested with the load applied at two opposite corners and supported at the remaining two corners.

*13 Sudden failure by slippage of longitudinal reinforcement in end anchorages. Steel in elastic range.

*14 Sudden failure by steel fracture at end anchorage.

Table 6 (Con't)

Chemically Prestressed Structural Elements

*15 Based on modulus of rupture from control specimens.

*16 Tested under a vertical load at the center of the beam and a horizontal load at the top of the column. The two loads were increased equally from zero to collapse.

*17 Observed loads at which large deflections began to develop.

*18 Longitudinal (Spiral)

*19 Concrete stress at the spalling of the shell

*20 Concrete stress at the maximum load on the restrained core

*21 Equivalent live load

*22 Based on membrane theory

Elements 7 and 8 present a direct comparison between uniaxial and biaxial restraint in slabs. It is interesting to note the 5% higher prestress in the uniaxially restrained slab. In other words, when the transverse direction is restrained, the longitudinal strain also decreases. This is another bit of evidence for the "direct" relation discussed earlier. Element 7 (uniaxial) exhibited excessive lateral expansion close to its center (away from the confinement of the edge beams) which resulted in bond failure between the aggregate and mortar and indicates the necessity for biaxial restraint. The difference between the predicted and actual cracking loads in this element might be due to a number of reasons. The 320 psf figure is the load at the end of the linear portion of the load-deflection curve. This might not actually correspond to the first crack. The yielding of the steel began at 420 psf. Also the predicted 446 psf is based on control specimens and no allowance was made for possible effects of the shape. In element 8 (biaxial), the predicted cracking load was based on a conservative theoretical assumption. In the second test the low observed cracking load might have been initiated by existing very small cracks. Also, the predicted values were based on control specimens.

In beam elements 9, 10 and 11, the low ultimate load was due to end anchorage failures. Element 11 is interesting since it is the only structural element with eccentric reinforcement. Eccentric prestress was observed and higher cracking and ultimate loads resulted.

Elements 12 and 13 are duplicate beam elements with relatively high steel ratios of high-strength deformed reinforcement. The discrepancy in cracking moments might be again due to differences between the modulus of rupture values of the control specimens and of the material in the beams (which had the additional restraint due to lateral stirrups.) Comparison with an ordinary reinforced concrete element showed the advantage of inhibiting the formation of cracks by prestressing. The maximum compressive strains at failure were 0.00331 and 0.00381 and a satisfactory maximum rotation at the center of 0.18 radians was observed.

Elements 14 (two frames) showed good agreement between experimental and theoretical load-deflection curves and collapse loads. It was concluded that the tests demonstrated the practicability of chemically prestressed frames with high-strength deformed bars, provided the steel percentage is at least 1.74. The applicability of conventional plastic collapse theory was also demonstrated. It should be pointed out, however, that these were only small, model beams and frames and size effect problems might still arise with full-scale elements.

Elements 15 and 16 are triaxially restrained, short (length to diameter ratio of five,) duplicate columns. A third column was tested, identical in detail but made with portland cement concrete. The behavior of the elements, under load, followed the expected behavior of spirally reinforced columns. A load was first reached, the yield load of the column, at which the shell outside the spiral spalled off. In the expanding concrete elements the shell concrete, which is restrained only longitudinally, would be expected to have a lower strength than the triaxially restrained core concrete. After the spiral spalls off, there is usually first a drop of load followed by a significant increase, depending on amount of spiral steel, as the core concrete takes more axial stress due to the increasing lateral pressure. Finally a maximum ultimate load is reached.

In these tests, electrical resistance strain gage readings were obtained, at the different stages, on both the longitudinal and spiral steel. It was thus possible to calculate the lateral stress as well as the longitudinal stress carried by the concrete. A straight line relation between the two was proposed in 1928, following research at the University of Illinois.(59,60) The equation is

$$f_1 = f'_c + K f_2 \tag{1}$$

where,

f_1 = axial stress

f'_c = compressive strength of concrete under axial load only

K = numerical factor

f_2 = lateral stress

The value of K in the Illinois tests was 4.1. Table 7 shows the values of K calculated at ultimate load for elements 15 and 16 (columns Y and Z respectively) and for the companion portland cement concrete column (column X). The difficulty in calculating K is in the selection of the appropriate value of f'_c to use in Equation (1).

TABLE 7. CALCULATION OF COEFFICIENT K (eq (1)) (Elements 15 and 16, Table 6, and Portland Cement Column (40))

COLUMN	CONCRETE COMPRESSIVE STRENGTH PSI	CONCRETE AXIAL STRESS AT SHELL SPALLING PSI	CONCRETE AXIAL STRESS AT ULT. LOAD PSI	CONCRETE LATERAL STRESS AT ULT. LOAD PSI	AGE AT TESTING DAYS	COEFFICIENT K
X PORTLAND CEMENT	*1. 9,385	9,220	12,760	865	54	4.09 *2. 3.90 *3.
Y ELEMENT 15	*4. 6,350	6,500	12,350	1,060	37	5.50 *5. 5.65 *6.
Z ELEMENT 16	*4. 6,350	6,660	11,910	1,060	37	4.95 *7. 5.25 *6.

NOTES:

*1. AVERAGE FROM COMPANION CYLINDERS

*2. CALCULATED WITH f'_c = SHELL SPALLING STRESS = 9,220 PSI

*3. CALCULATED WITH f'_c = 9,385 PSI

*4. TEST VALUE OF A COMPANION TRIAXIALLY RESTRAINED 6"×12" CYLINDER, FULLY RELEASED AT TESTING. (POSSIBLE DAMAGE ON RELEASE DISCUSSED EARLIER)

*5. CALCULATED WITH f'_c = SHELL SPALLING STRESS = 6,500 PSI

*6. CALCULATED WITH f'_c = 6,350 PSI

*7. CALCULATED WITH f'_c = SHELL SPALLING STRESS = 6,660 PSI

Note the higher axial strength at shell spalling of the plain concrete in comparison with that of the expanding concrete elements. However, the ultimate loads of the expanding concrete elements were not much lower. This is also reflected in their higher values of *K*. The reasons why expanding concrete columns should have higher values of K are not yet clear. This might be related to the internal local disruptions that could exist in such elements.

Element 17 is a hyperbolic paraboloid shell with equal reinforcement each way. The efficiency of the chemical prestressing was demonstrated by its behavior during loading. Even after unloading from 460 psf, all cracks closed and no further damage was observed during reloading to the same level. A considerable discrepancy is observed between predicted and actual cracking loads. This was probably due to the existence of moments, which are neglected by the membrane theory.

The following general conclusions seem justified:

(1) Test results indicate a wide range of structural applications possible with expanding concrete. It appears that the strength and behavior of structural elements, of the types described above, can approximately be predicted by conventional elastic and plastic theories and principles of prestressing.

(2) One important and difficult problem, due to the multitude of factors involved, is to predict and to design for the desired material properties of the structural members. Some standardization of the types of control

specimens is necessary and also more research into the problem of size and shape effects with expanding concrete elements.

(3) The need exists for more research into problems of eccentric prestressing and structural bond and for more full-scale structural tests.

(4) For the many reasons given earlier, future emphasis in research on expanding concrete structural elements should be placed on triaxial restraint and more uniform internal curing, possibly with lightweight aggregates.

6. ENGINEERING APPLICATIONS

In the U.S.A., expansive cement is at present being produced commercially by a number of cement companies (Kaiser Cement and Gypsum Co., Medusa Portland Cement Co. and Penn-Dixie Cement Co.) under license from the patent holding company, the Chemically Prestressed Concrete Corporation. The current (1965) price of expansive cement, is about 40% above the normal portland cement.(67) The expansive cement available commercially, under the name of Chem Comp, is of the shrinkage-compensating type. The manufacturers recommend the following specification to be used: "As a condition of acceptance, the average increase in length of three or more concrete specimens made with Chem Comp cement shall be not less than 0.1% nor more than 0.2%." The "self-stressing" cement, under the name of Chem Stress, is available on special order.

A great number and variety of construction jobs have already used the Chem Comp cement. The Engineering Data Book of C.P.C. (Chemically Prestressed Concrete Corporation)(61) lists over fifty applications. These and others have been described in various detail in technical, semi-technical and newspaper and magazine articles.(13,14,16,61,62,63,64,65,66,67,68,69,70,71) They include topping on structural slabs and prestressed beams, grade slabs, driveways, factory and parking slabs and decks, folded plate roofs, a swimming pool and highway pavements. Some of the parking decks made of Chem Comp cement have been subsequently post-tensioned by conventional means. It is claimed that the use of Chem Comp cement has resulted in the elimination of other waterproofing measures, the reduction or elimination of contraction joints and that very few or no shrinkage cracks at all have been observed in these structures. In some cases, companion sections made with conventional portland cement were available and these exhibited the usual shrinkage cracking.

On the other hand, very few field applications of self-stressing, Chem Stress, cement are reported. These include experimental slabs, experimental highway pavements and sections of a house.

Apparently, the very first field use(13) of expansive cement, with 14% expansive component, was in a reinforced, folded plate roof on a savings and loan building in Yuba City, California. The roof was placed in March 1963, covered an area of 100 ft. x 43 ft. and varied in thickness between 3 to 6 in. It had a fairly high percentage of steel and no expansion or contraction joints were used. A saving of $1,000 has been estimated(67) as a result of not needing a waterproof coating. Being the first, the description of this roof has been repeated many times.(13,16,61,62,63,65,67,69) Its performance has apparently been satisfactory. Some five hairline cracks were observed and attributed to finishing operations. One leak developed, in an area in which there were no cracks, and was blamed on a small region of high porosity.

Another example, in which overall saving is claimed,(13) is a grade slab, made with Chem Comp, 4 in. thick, reinforced with No. 3 bars on 16 in. centers each way and covering 22,000 sq. ft.(61) No contraction or expansion joints were used and this resulted in the overall saving. The slab is reported to have only one 3 in. long crack.

An important possible future use of expanding concrete is for highway pavement construction. Two experimental sections were installed in 1963 in California. One was in the Antelope Valley Freeway near Palmdale and the other on U.S. Highway 99, near Lodi.(13,14,16,61,63,72,73) The expansive cement used was of the shrinkage-compensating type with only 15% expansive component. The pavements contained no steel reinforcement and the only restraint was by the subgrade friction and end anchors. This experiment did not prove successful. The cracking in the expanding concrete was not less than that of the control sections made of normal portland cement concrete. It becomes obvious that for successful shrinkage compensation of highway pavements, a larger percentage of expansive component and some reinforcement will be required.

Three chemically prestressed pavements have been reported. These included a small slab field study, a large full-scale experimental field concrete pavement and an experimental section in the Connecticut highway system. A summary of all three is presented by Rice and Simms.(71) Some details are given below.

(1) Slab study.(71) Size: 3 ft. by 12 ft. by 5-1/2 inches. Cement: 25% expansive component. Steel: 1/4 in. high tensile wire at 2 in. centers both ways, giving 0.45% steel in each direction. Positive end anchorage provided for the steel. Concrete: 7-1/2 scy, water-cement ratio 0.40.

Curing: under water for 10 days.

Maximum expansion at five days: steel 0.14%, concrete 0.27%. Maximum prestress: steel 48,000 psi, concrete 210 psi. The values indicated are at the middle of the slab. At the edges the stresses were only about 55% of those at the middle. The stress level in the slab remained essentially constant up to 30 days when readings were discontinued.

(2) Blakeslee Slab. (cast June 1963)(16,61,64,71)

This pavement slab was constructed without joints at the prestressed concrete plant of C. W. Blakeslee and Sons, Inc., in Hampden, Connecticut.

Size: 13 ft. 4 in. by 185 ft. 6 in. by 6 in. Cement: Chem Stress (percentage of expansive component not stated.) Steel: longitudinal - twelve 1/2 in. dia. and six 3/8 in. dia., 7 wire, high tensile strands with end anchorage (0.23 percent steel), transverse - 1/2 in. dia. deformed bars on 8 in. centers (0.53 percent steel). Concrete: 10 scy, water-cement ratio 0.32. Slump 3-4 inches. Retarding and air entraining agents (4 oz./sk. Plastiment; 0.25 oz./sk. Darex). Average strength of 4 in. dia. cores were 2,880 psi and 3,025 psi at 7 and 28 days respectively. Curing: through the use of a water soaker for 7 days.

Overall expansion at 3 days was 0.27 percent. No change in expansion was observed between 5 and 14 days. The expansion corresponds to a prestress in the concrete of 150 psi. Concrete and steel expansions were in close agreement, indicating no apparent slippage. Finishing difficulties due to rapid stiffening of the concrete presented the only construction problem.

The slab is in use as a casting bed at the plant. At an age of 23 months it was reported to be crack-free with a good surface appearance and no wear or deterioration has been observed.

(3) Connecticut Highway (cast September 1963)(16,44,61,67,68,71,77)

The Blakeslee slab served as a pilot test for three experimental sections of the Connecticut Highway. These experimental sections, free of joints, were constructed of expanding concrete on the Connecticut State Route No. 2 near Glastonbury.

Size: Three sections, each 24 ft. 6 in. wide, 500 ft. long and 6 in. thick. Construction and expansion joints were provided between the sections. The concrete was placed on a base of 2 in. screenings and 1 in. bituminous concrete. Between the bituminous concrete and the expanding concrete there were two layers of polyethylene sheeting to minimize subgrade friction. Cement: 35% expansive component. Steel: longitudinal - twenty-one 1/2 in. dia., 7 wire, high tensile strands with end anchorage (0.175 percent steel); transverse - 7/8 in. dia. deformed bars on 24 in. centers and hooked at each end (0.42 percent steel). Concrete: 8-1/2 scy, water-cement ratio 0.425-0.465 by wt. Air entraining and set-retarding agents were used. Average strength of 4 in. dia. cores were 4,030 and 4,590 psi at 7 and 28 days respectively. Curing: fog spray for 24 hours and polyethylene sheeting for 7 days.

Construction difficulties were encountered due to a very rapid loss of workability. Some concrete had to be removed and three adjustments had to be made in the mix proportions (water was increased, maximum aggregate size reduced, aggregate proportions and amount of retarding agent changed.) The difficulties were magnified due to a lack of adequate vibration equipment during placement. The first slab placed experienced a possible compressive failure at approximately the mid-point of the slab that became evident shortly after final finishing. It was theorized that possibly excessive retardation could have kept the strength-gaining character of the portland cement lagging behind the stress build-up due to expansion and that the failure occurred during early stages. The retarding agent was therefore subsequently reduced to 4 oz./sk.

Strain measurements on the steel and dimension changes of the concrete in both directions were recorded. The average longitudinal expansion of the concrete was only 0.10 percent (concrete stresses of 25 to 100 psi) and the average transverse expansion approximately 0.05 percent (concrete stresses of 140 psi). The low longitudinal expansion was attributed to subgrade friction and to the larger early age expansion with increase in water. It was noted that length changes measured adjacent to sections of concrete placed at low slump indicated expansion nearly double of those measured in sections where high slump concrete was placed.

Inspection at the age of one year found the pavement to contain a reasonably large number of drying shrinkage cracks. Most of the cracks ran across the entire width of the pavement and were most prevalent in the center area of each of the slabs. In the middle third of the slabs, cracks were found as close as 10 to 20 feet apart. At the slab ends few cracks were noticed. The cracking pattern indicates that the prestress was less effective in the center sections of the slabs than at the ends. Measured expansions confirm this conclusion.

The Connecticut Highway pavement demonstrated some of the field difficulties involved and the need of further research work to minimize the concrete stiffening problems and achieve the desired uniform expansion and prestress. The 500 ft. slabs might have been too long. An optimum length for chemical prestressing may lie somewhere between 185 ft. (the Blakeslee slab) and 500 ft.

7. CONCLUSIONS

(1) The development of the U.C. cement represents a significant step in the future of expanding concrete. Further physico-chemical basic research could probably improve the details of manufacture and might throw light on the phenomena of crystal growth under restraint involved here.

(2) There is need for much more research to investigate interactions between factors influencing the properties of expanding concrete and pursue the desired beneficial effects of restraint in all directions and more uniform curing.

(3) The chemically prestressed structural tests described provide much information and promise for future applications. However, here again the need exists for additional work, testing and development. Eccentric prestress, structural bond, behavior under triaxial restraint are but a few of the problems requiring answers. The need exists for full-scale experiments with adequate research instrumentation and measurements over long periods of time to determine long range behavior and possible dangerous areas of deterioration with time.

(4) The potential of expanding concrete for engineering applications is great; shrinkage compensation, waterproofing, concrete repair, low creep applications, prestressed pavements and structural elements are some important uses. The successful use of shrinkage compensated expanding concrete seems to have been already established. The field use of chemically

prestressed concrete still presents problems. The many requirements(17,56) include adequate workability, adequate data for accurate design, proper conditions to ensure termination of expansion at a predetermined age and its maintenance at a suitable level under subsequent drying shrinkage and creep, adequate anchorage and bond, etc.

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APPENDIX I. CHEMICAL NOMENCLATURE

1. Standard Abbreviations

$C = CaO$

$\bar{S} = SO_3$

$S = SiO_2$

$A = A\ell_2O_3$

$H = H_2O$

$F = Fe_2O_3$

2. Compounds

Calcium Hydroxide:

$Ca(OH)_2 = CH$

Gypsum:

$CaSO_4 \cdot 2H_2O = \underline{C\bar{S}H_2}$

Anhydrous Calcium Sulfoaluminate:

$$4CaO \cdot 3A\ell_2O_3 \cdot SO_3 = Ca_4A\ell_6O_{12}SO_4 = \underline{C_4A_3\bar{S}}$$

Calcium Aluminate Trisulfate or Trisulfate:

$$3CaO \cdot A\ell_2O_3 \cdot 3CaSO_4 = \underline{C_6A\bar{S}_3}$$

Calcium Aluminate Monosulfate or Monosulfate:

$$3CaO \cdot A\ell_2O_3 \cdot CaSO_4 = \underline{C_4A\bar{S}}$$

Trisulfate Hydrate (Ettringite):

$$3CaO \cdot A\ell_2O_3 \cdot 3CaSO_4 \cdot 32H_2O = C_3A \cdot 3C\bar{S} \cdot 32H = \underline{C_6A\bar{S}_3H_{32}}$$

Monosulfate Hydrate:

$$3CaO \cdot A\ell_2O_3 \cdot CaSO_4 \cdot 12H_2O = C_3A \cdot C\bar{S} \cdot 12H = \underline{C_4A\bar{S}H_{12}}$$