SCHOOL OF CIVIL ENGINEERING

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EXPANSIVE (SELF-STRESSING) CEMENTS: IN REINFORCED CONCRETE

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PURDUE UNIVERSITY
INDIANA STATE HIGHWAY COMMISSION

Final Report

EXPANSIVE (SELF-STRESSING) CEMENTS: IN REINFORCED CONCRETE

TO: H. L. Michael, Director
Joint Highway Research Project

December 8, 1978

Project: C-36-58D

FROM: C. F. Scholer, Research Associate
Joint Highway Research Project

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Attached is a Final Report on the JHRP-HPR Research Project titled "Expansive (Self-Stressing) Cements: In Reinforced Concrete". It has been authored by the principal investigators, Prof. C. F. Scholer and Prof. E. C. Ting, along with the graduate students who completed portions of this project as part of their graduate studies. H. Gowda's work was previously published as an interim report. The studies of V. A. Harris and D. H. Wagner are included in this report as Part II and III respectively.

The findings of this investigation show promise of a controlled expansive cement concrete but also show serious problems associated with the material. The great sensitivity of the plastic concrete to temperature effects, the sacrifice of strength for expansion and the low durability reduce the applicability of the material. If a better material becomes available which is able to eliminate these problems, further development of self-stressing cement concrete would be justified.

The Report is submitted as fulfillment of the objectives of the research project.

Respectfully submitted,

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16. Abstract

This four part report describes the evaluation of a highly expansive cement for structural applications in which three dimensional restraint was developed by the use of reinforcing steel cages or steel pipe in a column application.

The first part summarizes the evaluation of the expansive, Type M cement. Mechanical properties as well as structural behavior of a reinforced concrete beam were evaluated.

Part II and III are detailed descriptions of the evaluation of columns with conventional tied or spiral reinforcement and with steel tubes filled with the concrete. Included is a study of the effects of air entrainment on expansion and of durability of such self stressed units in freezing and thawing. Poor durability was found in this exposure. A study of the time of set properties of the expansive cement concrete was also included. The concrete's time of set was extremely sensitive to temperature.

Part IV is a critical evaluation of the investigation's results with recommendations for future utilization of highly expansive (self-stressing) cement concrete. The first requirement for future development should be an improvement in the compressive strength capability of expansive cement in order to obtain efficient utilization of the restraint steel.

17. Key Words
Concrete; Pre-stressed; Expansive Cement; Self Stressing; Columns

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EXPANSIVE (SELF-STRESSING) CEMENTS: IN REINFORCED CONCRETE

by

C. F. Scholer, E. C. Ting, H. Gowda, V. A. Harris, D. H. Wagner

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This report contains four parts:

Part I summarizes the results of the Interim Report (JHRP-76-24) which describes the evaluation of expansive cements by means of free expansion of mortar bars, effects of longitudinal as well as triaxial restraints on self stress development and mechanical properties, effects of eccentric restraints. It also includes the structural behavior of self- stressed concrete beams and discussions concerning possible savings with effective use of expansive cement for practical designs.

Part II of the report describes the study of the structural behavior of reinforced concrete beams and columns using tied or spiral reinforcements as lateral constraint for expansive cement concrete. A study of the effects of air entrainment on the expansion, strength and durability properties and a study of the time of set properties of expansive cement concrete are also included.

Expansions were monitored using electrical resistance strain gages mounted on the steel reinforcing of 5 x 5 x 21 inch specimens. In each of the twelve specimens tested for expansion the longitudinal reinforcement consisted of eight #3 deformed reinforcing bars. In six of these twelve specimens the lateral reinforcement was a tie arrangement. In the other six a double spiral helix was used as lateral restraint. In four of the twelve, air was entrained in the concrete so its effects on expansion could be monitored. After expansion the four air entrained and four of the non-air entrained 5 x 5 x 21 inch specimens were tested as

columns with concentric loadings. Five specimens made with portland cement were also tested as controls. The four remaining expansive 5 x 5 x 21 inch specimens were tested as beams with single point loadings on 19 inch spans. Three portland cement concrete specimens were also tested as controls.

The freeze-thaw durabilities of three 3 x 4 x 16 inch expansive specimens were investigated. These specimens were triaxially reinforced. Seventy-one cycles of freezing and thawing were performed on these specimens as well as on three air entrained portland concrete specimens with reinforcing arrangements identical to those of the expansive specimens.

Time of set tests were performed on expansive cement mortars and compared to times of set of portland cement mortars prepared under the same conditions.

Results of the expansion tests indicated that when air is entrained in the expansive concrete the expansion is markedly reduced. The highest self-stresses due to expansion occurred in the specimens with no air entrainment and with spiral helices as the lateral reinforcement. In all cases the quality of the cover concrete on the expansive specimens was very poor and its ability to provide adequate protection for the reinforcing steel against corrosion was questionable.

The column tests showed that the compressive stress carrying capacity of the expansive concrete was no better than that of the portland cement concrete, however, the ductility of a typical expansive specimen was much higher. The air-entrained expansive specimens were much weaker than the other specimens, yet the ductilities were still high.

The beam tests showed the tied expansive specimens as being more ductile and attaining slightly higher ultimate moments than the tied portland cement concrete specimens. The spiral specimens made with expansive cement concrete exhibited more ductility but lower ultimate moments than the spiral specimens made with portland cement concrete.

The freeze-thaw durability of the expansive concrete, even with air entrainment, was very poor.

The times of set of the expansive cement mortars were extremely fast compared to those of the portland cement mortars. Set times were able to be altered by changing mix temperature and/or water to cement ratio.

In part III, structural steel tubular columns filled with expansive concretes and others similarly filled with portland cement concrete are examined and compared. The effects of curing temperatures on the expansions and self-stressing of the composite columns were observed for several weeks. The objective was to compare the structural performance of columns cast with expansive concrete against those cast with the portland cement concrete. The column tests were for short columns loaded in axial compression.

At the earlier ages expansion was faster than at the latter. Quite noticeably the higher the curing temperature, the faster and greater the expansive growth. Expansion, self-stress and strength were all found to vary directly with the curing temperature.

The strength and stiffnesses of expansive concrete filled columns was significantly greater than for the columns filled with normal portland cement concrete. Of the expansive columns, the group cured at the low temperature of $50^{o}F$ was the only one weaker than the normal columns. Yet it was still stiffer in the lower working levels of loading. In addition the expansive concrete filled columns increase the maximum allowable strain prior to buckling.

CHAPTER 1

INTRODUCTION

Self-stressing cements have long been recognized as materials which might be beneficial for some reinforced concrete structures. It is a well-known fact that self-stressing can be achieved if the cements have the potential of producing large expansions and if the concretes are restrained properly by steel reinforcement. Such a combination can produce compressive stresses in the concrete and high tensile stresses in the steel.

Unrestrained expansion of concrete produces a volumetric strain which is of sufficient magnitude to be disruptive and results in poor mechanical properties, such as low compressive strength and low modulus of elasticity. The basic function of restraint in self-stressing concrete is the utilization of potential expansion to produce a favorable state of stress in the concrete. This state of stress is beneficial in resisting the tensile stresses induced by externally applied loads and influencing the internal structure of concrete in a manner that favorably affects its properties. There is a loss of self-stress, due to creep and drying shrinkage, as in conventional prestressed concrete. However, the residual self-stress in a confined expansive concrete is significant from the structural point of view.

From the point of view of practical application of an expansive cement, it would be logical to investigate the potentialities of the cement in developing adequate states of self-stress when expansion is restrained. For simplicity, most of the previous studies of the structural elements were restrained uniaxially. From the results of uniaxially restrained structural elements, it was found that the earlier microfissures, due to the expansions perpendicular to the direction of restraint, detrimentally affect the mechanical properties of the resulting concrete. More important, the mechanical properties of a structural concrete member using expansive cement have shown to be dependent upon the amount of restraint and the manner in which the structural member was restrained. Hence, a promising application of self-stressing cement concrete lies in the utilization of its three dimensional expansion to provide three dimensional prestressing of concrete and to achieve better mechanical properties.

The energy requirements for producing self-stressing cements are not greatly different than those for conventional hydraulic cements. Hence it would be logical to investigate the technological and economic benefits of making use of the self-stressed concrete structural members following the studies of restraintments and mechanical properties.

Objectives of the Study

The primary objective of this project was to gain a better understanding of the behavior of self-stressing reinforced concrete structures, in particular, the developments of prestress under various restraining conditions and the mechanical properties of the prestressed structures. A second objective was to investigate the possible technological and economic benefits in the effective use of expansive cement under simulated construction conditions.

To achieve the objectives, the following studies were made:

1. Developed expansive cements with different, yet consistent expansion properties. (See chapter 2).
2. Evaluated the effects of longitudinal and lateral restraints on self-stress development. Attempts were made to determine a possible optimum lateral/longitudinal reinforcement ratio. (See chapter 3).
3. Studied the mechanical properties of triaxially restrained cyclinders. (See chapter 3).
4. Studied the effects of eccentric longitudinal reinforcements in triaxially restrained conditions. (See chapter 4).
5. Studied the structural behaviors of triaxially restrained self-stressed concrete beam structures. (See chapter 3).
6. Studied the structural behaviors of triaxially reinforced concrete columns. (See chapters 6 thru 10).
7. Studied the structural behaviors of composite column made of steel tubes filled with expansive concrete. (See chapters 11 thru 14).

8. Studied the time of set of expansive cement and effect of curing conditions. (See chapter 8 and 13.
9. Studied the freezing and thawing resistance of self-stressed concrete members. (See chapter 8).

During Phase I of the project, work involved in items 1 to 5 were carried out by Dr. Hanume Gowda. Details of the test programs and results have been reported in an interim report (JHRP-76-24). A highlight is included in this chapter and a brief summary of the results are included in chapters 2 to 5.

Mr. V. A. Harris and Mr. D. H. Wagner have carried out the Phase II of the Project. Chapters 6-10 describes the test program and test results concerning reinforced concrete columns carried out by Mr. Harris under the supervision of Professors Scholer and W. F. Chen. Chapters 11-14 describe the details and results of tests for steel tube columns filled with expansive concrete. The tests were performed by Mr. Wagner under the supervision of Professors Scholer and Ting. The freeze and thaw properties and the tests for the time of set were jointly studied by Messrs. Harris and Wagner.

Highlight Summary of the Interim Report by H. Gowda

The Interim Report describes the evaluation of expansive cements by means of free expansion of mortar bars, effects of longitudinal as well as lateral restraints on self-stress development and mechanical properties, effects of eccentrical restraints, and effects of different curing conditions. It also describes the structural behavior of self-stressed concrete members and the determination of economic savings possible with the effective use of expansive cement under practical construction conditions.

The length change measurements of mortar bars were made in a length comparator. The expansive cement evaluated was a blend of portland cement, calcium aluminate cement, and gypsum. The calcium aluminate cement contents were varied from 10 to 25 percent and the gypsum contents were in the range of 10 to 20 percent. Even though the rate increased, the ultimate expansion decreased with the higher percentages of calcium aluminate cement. Two expansive cements, giving a medium and a high amount of expansion, were selected for the remainder of this investigation.

The self-stress development, longitudinal expansion, and lateral expansion measurements were made on 3 x 3 x 11 inch beams. Four different percentages of unbonded longitudinal steels were used. The studies were made on uniaxially restrained specimens, triaxially restrained specimens (with two different percentages of lateral steel and eccentrically restrained specimens). In triaxially restrained specimens the self-stress development was increased as the degree of lateral steel was increased. The lateral steel also prevented the longitudinal cracks which were observed in uniaxially restrained specimens. It was possible to achieve different magnitudes of self-stress at the top and the bottom of the beam by placing the longitudinal steel eccentrically.

Water curing was used and measurements were made at a regular interval of time. Studies were also made to find the effect of different curing temperatures in triaxially restrained beams with 0.56 percentage of lateral steel. In this case the beams were placed in a hot water bath at 80°C for six hours and then water cured. A lower self-stress development was measured for the 80°C water curing procedure. (Subsequent work reported in chapter 13 does not agree with this finding).

The 6 x 12 inch cylinders were used to study the mechanical properties of triaxially restrained self-stressed concrete. Five different percentages of longitudinal steel were used. The lateral steel percentage of 0.56 in the form of spirals was used in all the cylinders. The triaxially restrained and free expansion cylinders were tested in compression at the age of 28 days. The results indicated a significant improvement in mechanical properties of triaxially restrained specimen compared to the free expansion specimen.

The structural behavior of self-stressed concrete was studied by using 3 x 4 x 24 inch beams. Beams were tested in flexure using a third point loading. For comparison purposes, the ordinary reinforced concrete beams were also tested. The structural behavior of self-stressed concrete beam was similar to the ordinary reinforced concrete beams except less deflection and fewer cracks occurred in the self-stressed concrete beams. Using the equal deflection criteria, the savings in steel of self-stressed concrete compared to conventional reinforced concrete beams was calculated and found to be about 20 percent. This 20 percent reduction could have significant economic benefits.

Results of the free expansion studies indicate that for any given gypsum content, the rate of expansion was increased as the calcium aluminate cement was increased.

PART I:

DEVELOPMENT OF SELF-STRESS AND THE MECHANICAL PROPERTIES OF EXPANSIVE CONCRETE

CHAPTER 2 PROPERTIES OF THE EXPANSIVE CEMENT

Expansive cements have been studied during the past three decades in France, the Soviet Union and the United States. A cement with only a small expansive potential is marketed for the purpose of compensating for the shrinkage that results when concrete dries. The use of expansive cement for self-stress development in concrete has been used to a much more limited extent. Although expansive cements for shrinkage-compensation concrete are commercially available, it is still difficult to obtain cements for self-stressing purposes which require considerably larger expansions. Hence, it was necessary to develop expansive cements in the laboratory.

Since the magnitude of the expansion depends on the ingredients of the cement, it is also necessary to study the effects of ingredients on the expansion in order to use the self-stressing cement more efficiently. In this study, the portland cement, calcium aluminate cement, and gypsum were used as ingredients. The calcium aluminate cement contents were in the range of 10 - 25%, and gypsum content was varied from 10% to 25%. The product is generally referred as the Type-M expansive cement. Simple mortar bars were used to characterize the expansion properties.

Based on the conditions of tests and the materials employed in this investigation, the results indicate that the rate of expansion is increased as the amount of calcium aluminate cement content increases, but the magnitude of the expansion decreases. More importantly, a laboratory expansive cement was successfully developed which was reproducible and has controllable expansion characteristics.

Materials and Expansion Measurements

A Type I portland cement from a single clinker batch was used in this investigation. The calcium aluminate cement was obtained in one shipment. The gypsum (calcium sulfate, reagent grade) was received from a chemical house and was used for the work reported in the interim report. A commercial grade of gypsum was used for the work reported in Parts II and III of this report.

The fine aggregate used in casting mortar bars was a local glacial-alluvial sand with fineness modulus of 2.90. The aggregate was sieved and the material from each sieve was stored in a separate container. In order to eliminate any possible changes in the gradation the aggregate is batched for each mix by component fraction in accordance with accumulative batch weight formula (F.M = 2.90).

The self-stressing cement was prepared by mixing the ingredients (portland cement, calcium aluminate cement and gypsum) for 3 minutes at low speed with a mechanical mixer. This was done just before the preparation of mortar in order to eliminate any possible reaction due to the long period of storage. All mortar mixes were prepared using water to cement ratio of 0.4 and sand to cement ratio of 2. The ingredients of self-stressing cement were varied for each mix. After preliminary studies it was thought of investigating free expansion measurements by using calcium aluminate cement of 10, 15, 20 and 25% and gypsum content of 10, 15 and 20%.

The molds for casting test specimens and the length comparator for measuring length change were in accordance to the requirements of ASTM Specifications C 490. The mechanical mixer for mixing mortar was in accordance with ASTM Specification C 305.

Two mortar bars (x x 1 x 11 1/4 in.) were cast according to the requirements of ASTM C 157 from each mix of mortar. The casting of specimens took place within 10 minutes after the mixing was completed. As soon as casting was completed the specimens were covered with a plastic sheet and then with a damp cloth to prevent the evaporation of water from the specimens.

Specimens were removed from the mold about 18 hours after casting. At this time, the initial length measurements of mortar bars were made using the length comparator. The specimens were cured in a fog room and length measurements were made at 48 hour time intervals until constant readings were observed or until the specimens were cracked and broken.

Results

The expansion versus time curves for each mortar mix are given in Figures 2-1, 2-2, and 2-3. The results indicate that for any given gypsum content the rate of expansion increases as the calcium aluminate cement content increases, except for the specimens of 20 percent calcium aluminate cement at 10 percent gypsum. For this particular case, the reaction (thereby expansion) between calcium aluminate cement and gypsum was probably completed within the first 24 hours during which the specimens were in the mold, hence no expansion was measured.

From the curves it is also shown that for constant calcium aluminate cement content the rate of expansion increases as the gypsum content increases. For example, with a calcium aluminate cement content of 15 percent and gypsum contents of 10, 15, and 20 percent, at the age of 4 days expansions of 0.020 inch, 0.035 inch, and 0.040 inches were obtained respectively.

The appearance of the specimens at the end of the curing period falls into one of two basic categories: surface cracks were observed even in a low expansion specimen; and, in high expansion specimens large surface cracks and some spalling at the corners were observed. Self-destruction and in many instances, warping also occurred.

During the course of this study, it was interesting to note that the mortar specimens which warped, did so in the direction of the top side of casting regardless of the manner in which the specimens were kept in the process of curing. This means the larger expansion occurred in the bottom and lesser expansion occurred in the top portion of the as cast specimen. It was concluded that this differential expansion was due to the different water cement ratios at top and bottom side of the specimen. Due to the bleeding characteristics of mortar, the water cement ratio at top is higher than the water cement ratio at the bottom. To verify this, as soon as casting was completed, the specimen was covered with a glass plate and the mold was kept upside down. Some of the bleeding water came out of the specimen. After demolding the specimen was cured in the same way as other specimens. In this particular specimen no warping was observed. This indicates that extra care must be taken in the curing process of self-stressing concrete construction to avoid non-uniform expansions.

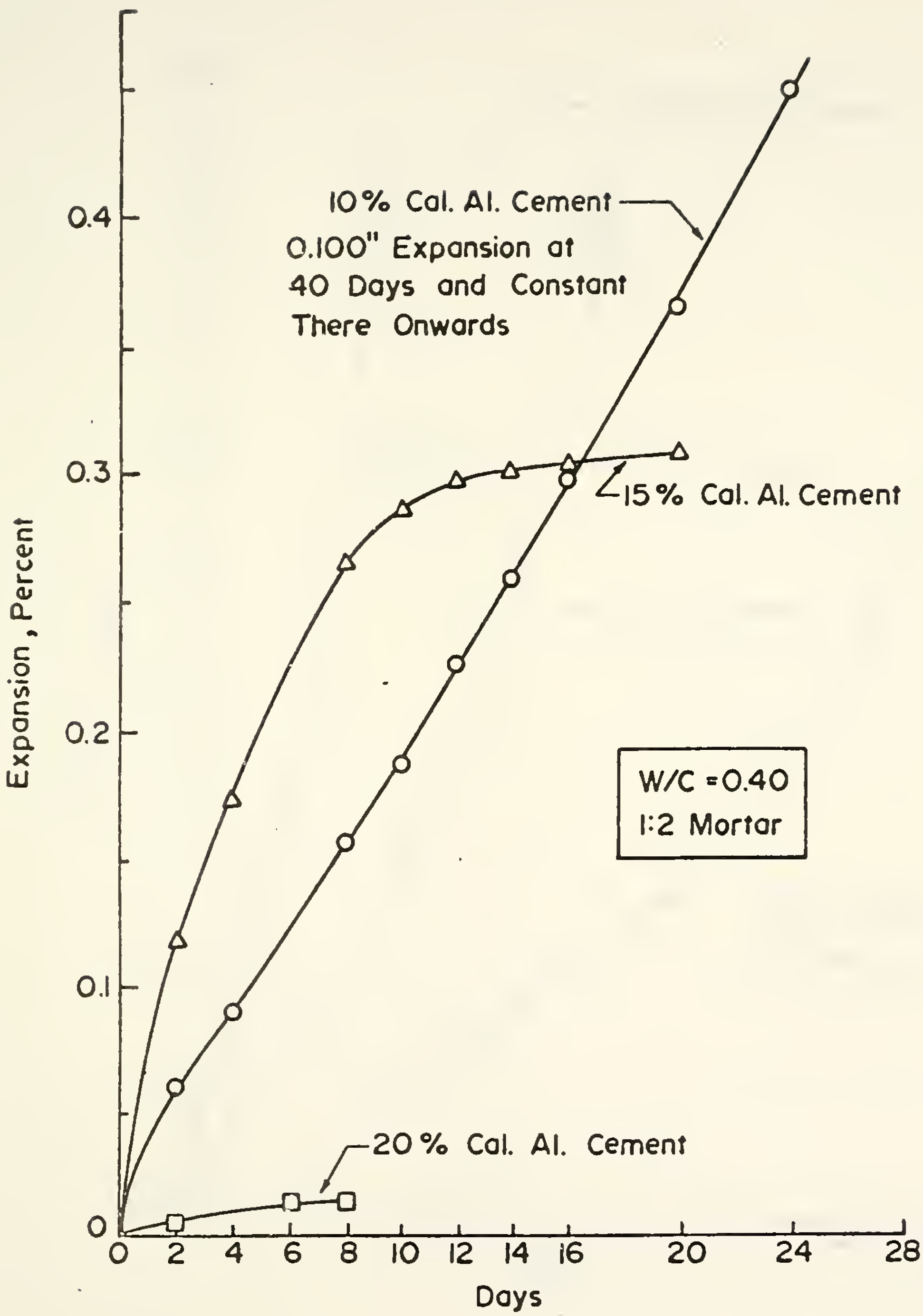


FIGURE 2-1 INFLUENCE OF CALCIUM ALUMINATE CEMENT AT 10 PERCENT GYPSUM.

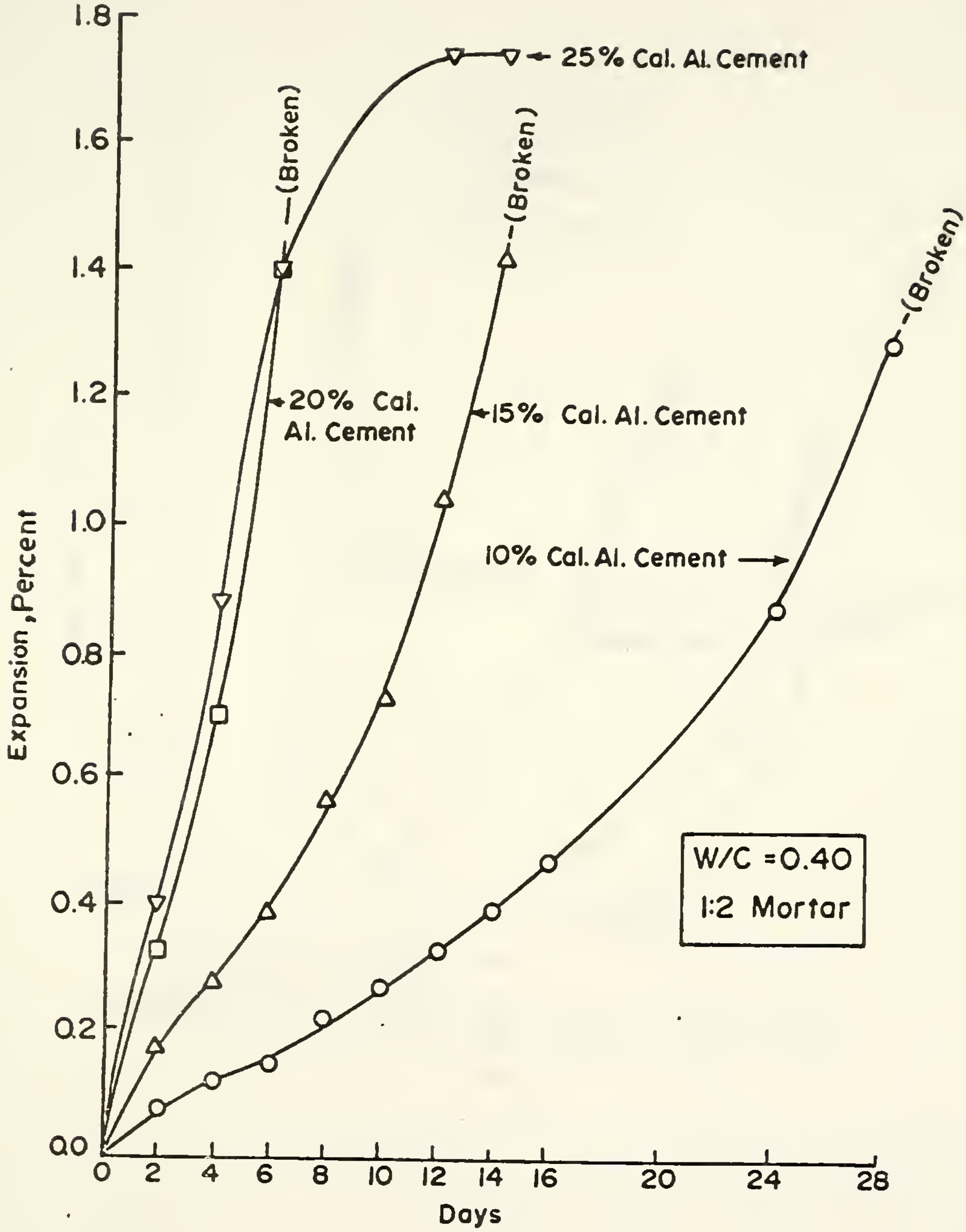


FIGURE 2-2 INFLUENCE OF CALCIUM ALUMINATE CEMENT AT 15 PERCENT GYPSUM.

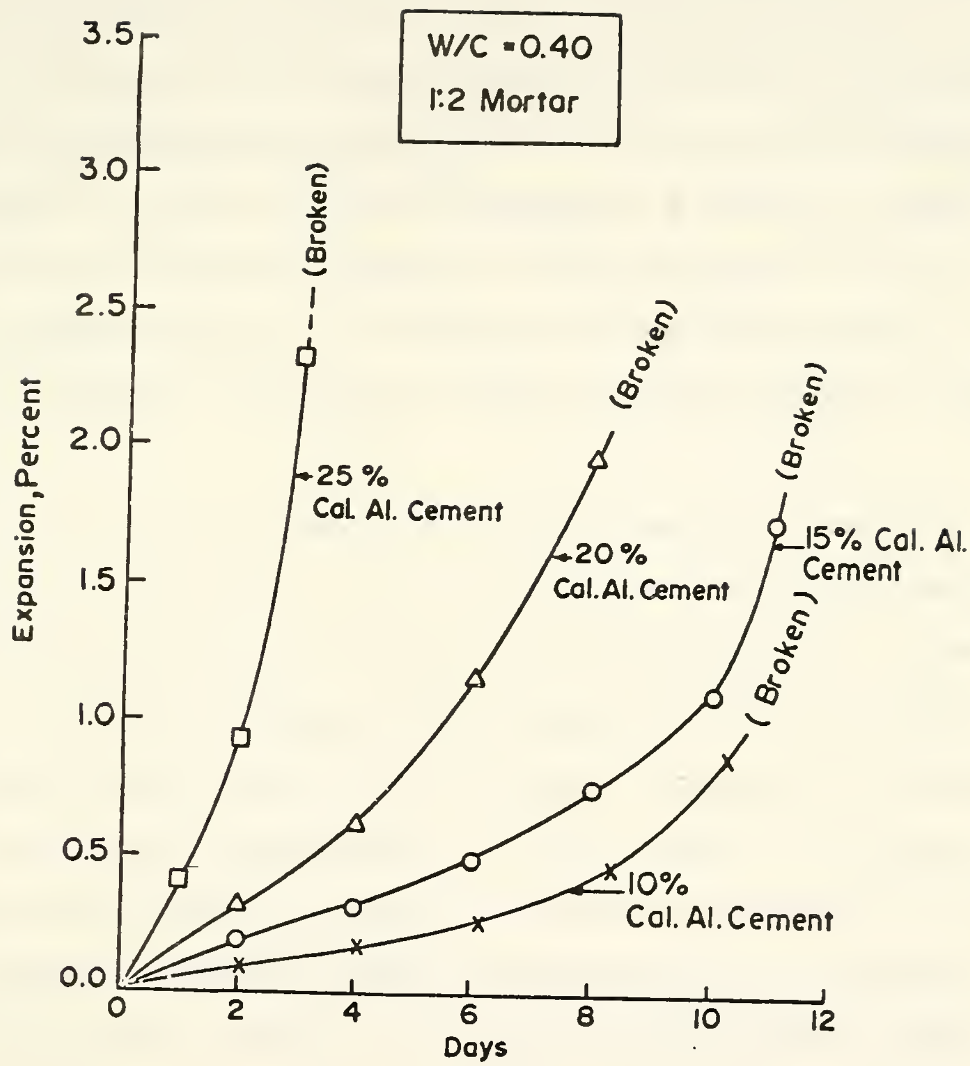


FIGURE 2-3 INFLUENCE OF CALCIUM ALUMINATE CEMENT AT 20 PERCENT GYPSUM.

CHAPTER 3 EFFECT OF RESTRAINTS ON SELF-STRESS DEVELOPMENT

The basic function of restraint in self-stressing concrete is the utilization of potential expansion to produce a favorable state of stress in the concrete. This state of stress is beneficial in resisting the tensile stresses induced by externally applied loads and in bringing about changes of internal structure of concrete in a manner that favorably affects its properties. There is a loss of self-stress, due to creep and drying shrinkage, as in conventional prestressed concrete. However, the residual self-stress is significant from the structural point of view.

The self-stress development of a structural concrete element containing self-stressing cement depends greatly upon the degree of restraint and the manner in which it was restrained. An objective of this investigation was to find the effects of various degrees of uniaxial restraint for different types of cement. Methods of mix design and test procedures were detailed in the interim report by H. Gowda. Briefly, the Type-M expansive cements described in Chapter 2 were used for the study. The mix design was developed to produce a concrete with 2 to 3 inches of slump. The schematic diagrams of a uniaxially restrained beam specimen and a tri-axially restrained beam specimen are shown in Figs. 3-1 and 3-2.

Uniaxial Restraints

Assuming the restraining rods were stressed within their elastic range, the steel stresses were calculated by multiplying the restrained expansion by the modulus of elasticity of the steel. Once the steel

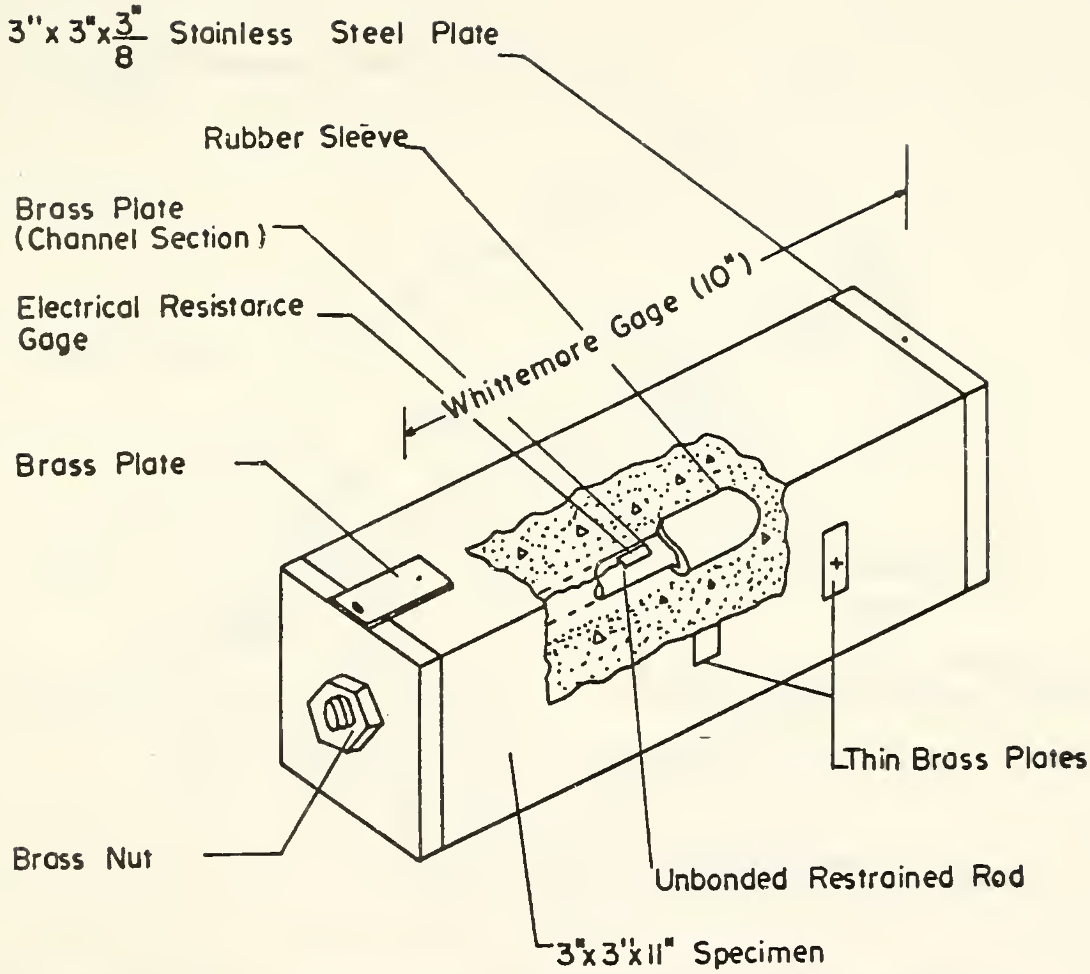


FIGURE 3-1 SCHEMATIC DIAGRAM OF UNIAXIALLY RESTRAINED BEAM.

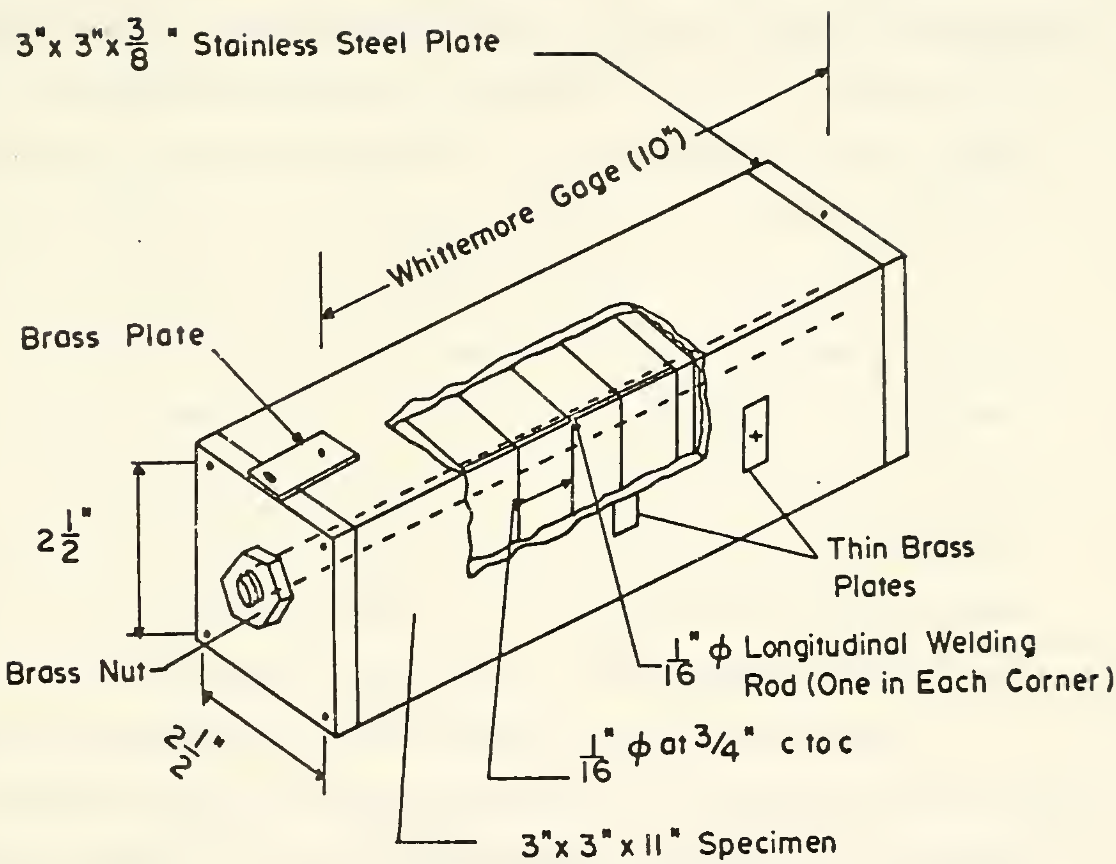


FIGURE 3-2 SCHEMATIC DIAGRAM OF TRIAXIALLY RESTRAINED BEAM.

stresses have been determined, the self-stress developed in the concrete was calculated by multiplying the steel stress by the ratio of steel to the area of concrete. The variations of these stresses with time are shown in Figures 3-3 and 3-4.

In a medium expansive concrete specimen (Figure 3-3), the self-stress developed rapidly in the first two days of curing. Then, the development was at a slower rate and almost terminated at 14 days. In the case of high expansive concrete (Figure 3-4), the increase was very fast within the first four days. It slowed down between two to six days. At 14 days, the self-stress increase was negligible.

The effects of longitudinal restraint shown in Figures 3-4 and 3-5 were different from results reported in the literature. In a similar study of the uniaxial restraints, it was reported that, the self-stress developed in the concrete increased as the steel percentage increased from 0.6% to 3.23% and decreased for the steel percentages higher than 3.23%. The present study showed that, as the percentage of steel increased, the self-stress in the concrete continuously increased.

During this study, random cracks in all free expanded concrete specimens were observed. For the medium expansive concrete, no longitudinal cracks were observed. However, for the highly expansive concrete specimens, the longitudinal cracks were observed at the age between 4 and 6 days. At the age of 6 days, these longitudinal cracks were clearly visible. Then, the lateral expansion started to increase rapidly without increase in longitudinal expansion. Comparison of lateral expansions of uniaxially restrained specimens (Table 3-1) of highly and medium expansive concrete specimens shows that about four times greater expansion exist

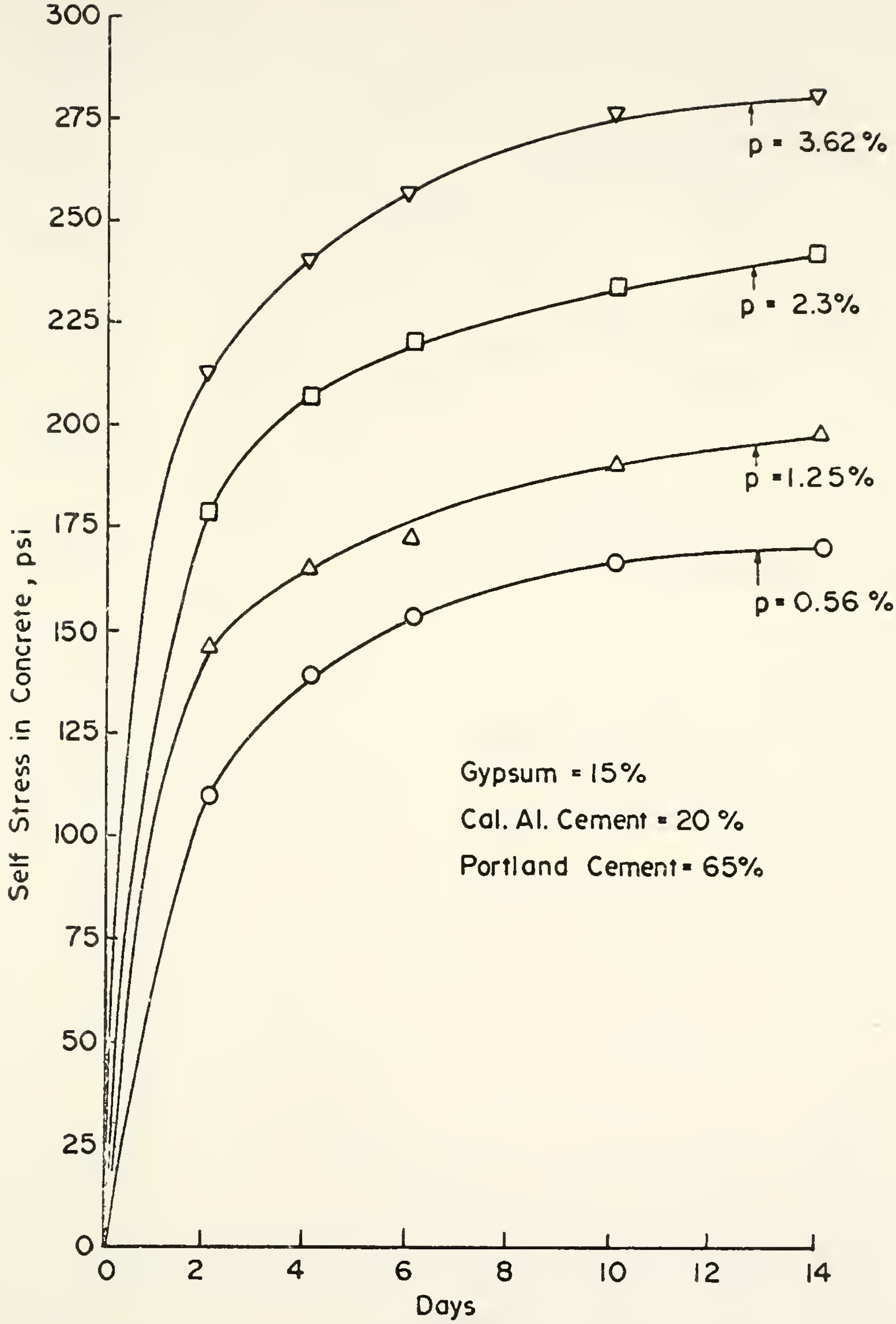


FIGURE 3-3 SELF-STRESS DEVELOPMENT IN UNIAXIALLY RESTRAINED MEDIUM EXPANSIVE CONCRETE SPECIMENS.

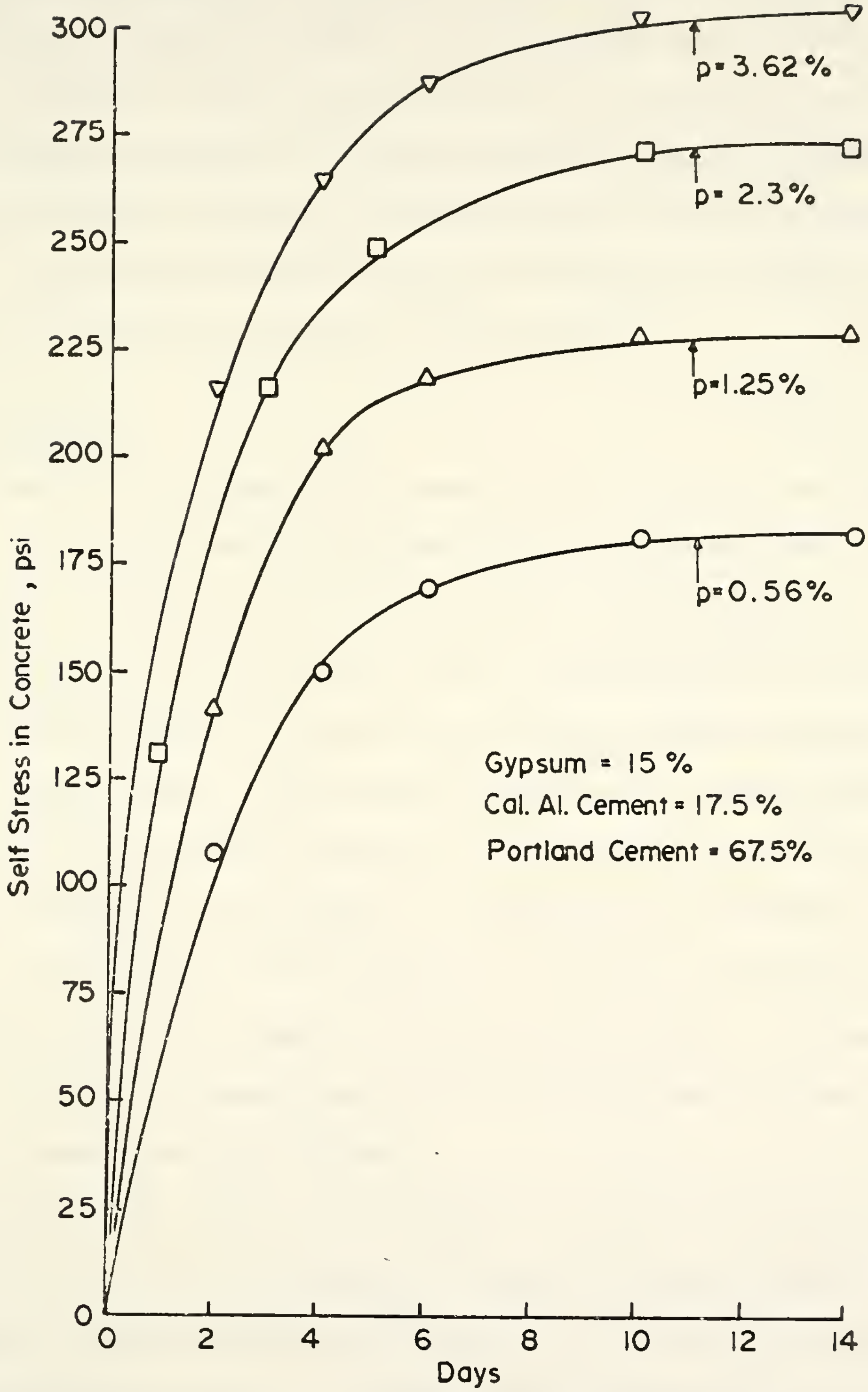


FIGURE 3-4 SELF-STRESS DEVELOPMENT IN UNIAXIALLY RESTRAINED HIGH EXPANSIVE CONCRETE SPECIMENS.

in highly expansive specimens. In addition, the increase of expansion of the concrete no longer contributes to the development of self-stress after the longitudinal cracks appeared. Thus, it is important to note that for efficient application of expansive cement to generate prestress in a concrete structural member, the structural member should be triaxially restrained. Limited restraints in one or two directions have shown to be ineffective in developing a desirable state of stress.

Triaxial Restraints

Three series of experiments were conducted for triaxially restrained specimens. In the first series, specimens were of medium expansive concrete with a lateral restraining steel of 0.56 percent. The specimens of the second series were of high expansive concrete with 0.56 percent of lateral steel. The third series of specimens were of highly expansive concrete percent of lateral steel. The self-stress developments of these series as functions of age are shown in Figures 3-5, 3-6, and 3-7. Each curve represents the average of the results of two specimens with identical restraints of reinforcement. The variation of test results between the two specimens was within a range of 10 percent in all cases. In these tests most of the self-stresses were developed in the first four days with little change after the sixth day.

Comparison of the curves given in Figures 3-5 and 3-6 with those shown in Figures 3-3 and 3-4 indicates that the self-stress developed in the triaxially restrained specimens was greater as compared to the uniaxially restrained specimens. The higher stress level was due to the existence of lateral stirrups in the triaxially restrained specimens. The stirrups restricted the concrete expansion in the lateral directions and therefore the specimen expanded more longitudinally.

Table 3-1. Lateral Expansions of Uniaxially and Triaxially Restrained Specimens

A. Medium Expansive Concrete

Type of Restraint	Longitudinal Steel, %	Lateral Expansions, % at 1/4 pt.	Lateral Expansions, % at 1/2 pt.
Uniaxial	3.62	0.57	0.68
	2.30	0.61	0.73
	1.25	0.69	0.80
	0.56	0.89	0.96
Triaxial with 0.56 percent of lateral steel	3.62	0.42	0.53
	2.30	0.58	0.66
	1.25	0.67	0.73
	0.56	0.78	0.86

B. High Expansive Concrete

Type of Restraint	Longitudinal Steel %	Lateral Expansions, % at 1/4 pt.	Lateral Expansions, % at 1/2 pt.
Uniaxial	3.62	2.90	3.10
	2.30	2.93	2.96
	1.25	3.26	3.90
	0.56	3.96	4.26
Triaxial with 0.56 percent of lateral steel	3.62	1.66	1.73
	2.30	1.76	1.86
	1.25	2.23	2.26
	0.56	2.46	2.60
Triaxial with 0.84 percent of lateral steel	3.62	1.20	1.26
	2.30	1.40	1.43
	1.25	1.66	1.80
	0.56	1.96	2.06

The lateral steel percentage was then increased from 0.56 to 0.84 in order to find the effect of lateral restraint on self-stress development. Results of this investigation (Figure 3-7) indicated an increase in self-stress as the lateral steel percentage increased. As before, the increment was due to the closely spaced stirrups which confined the concrete laterally thus resulting in larger longitudinal expansion and greater self-stressing.

Comparison of lateral expansions (Table 3-1) of uniaxially and triaxially restrained specimens of medium expansive concrete indicates the smaller lateral expansions for triaxially restrained specimens.

For highly expansive concrete specimens, the lateral expansion was decreased as the lateral steel increased from 0 to 0.84 percent. The decrease was measurable even with a small percentage of lateral steel (0.56 percent). Also, it was observed that the presence of this small amount of lateral steel was capable of preventing the longitudinal cracks by confining the concrete in the lateral direction. Thus, considerable improvement in the mechanical behavior was also found in triaxial restrained specimens.

Study of the Optimum Reinforcement Ratio

An interesting study, and possible of practical importance, which was proposed for investigation concerns with the existence of an optimum longitudinal/lateral steel ratio. If such an optimum ratio is within the practical design range, it should be of great interest to the users of the self-stressing concrete. The existence of an optimum ratio may be reasoned as follows: At lower percentages of lateral steel used, the self-stress development increases as the lateral steel percentage

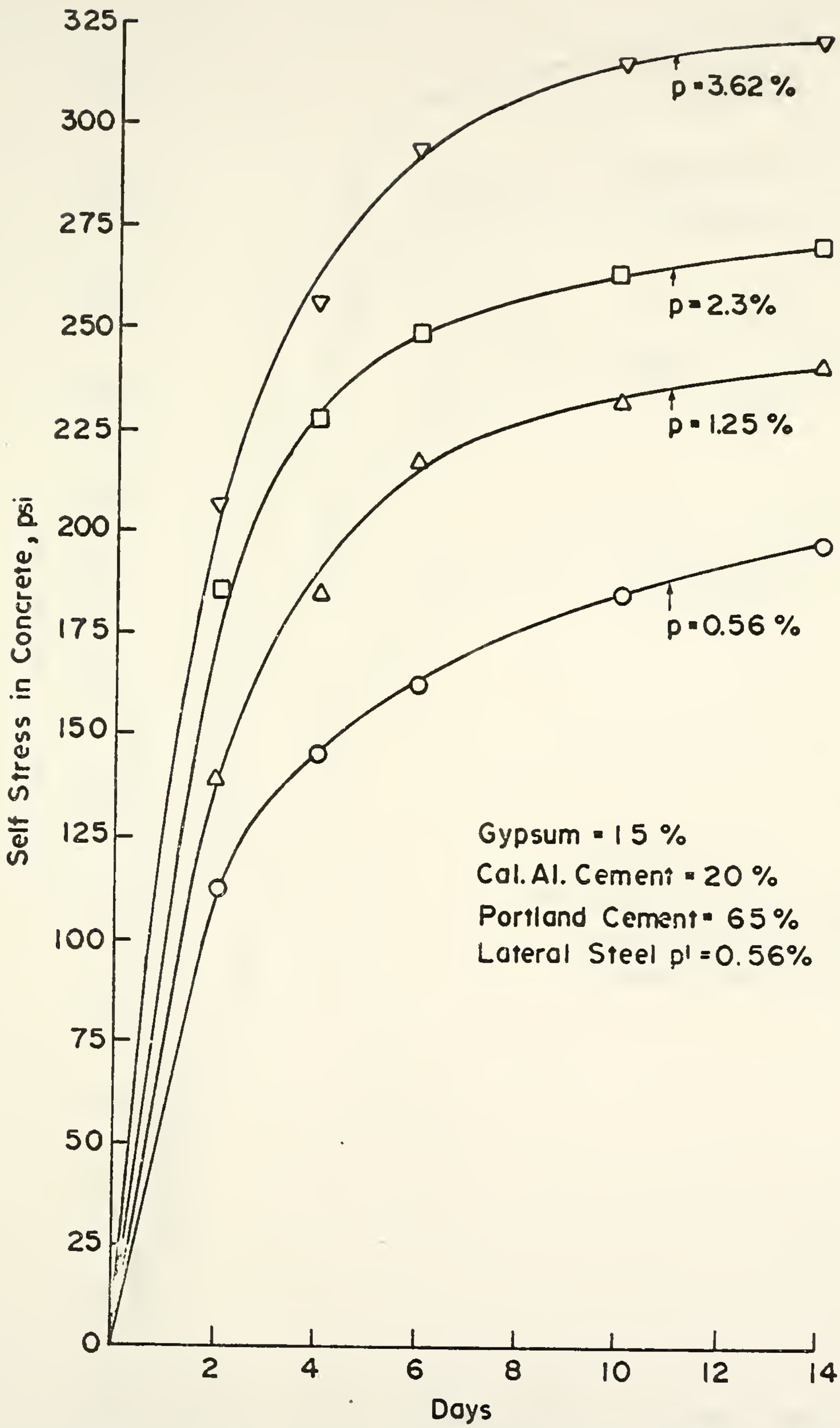


FIGURE 3-5 SELF-STRESS DEVELOPMENT IN TRIAXIALLY RESTRAINED MEDIUM EXPANSIVE CONCRETE SPECIMENS WITH P' = 0.56 %.

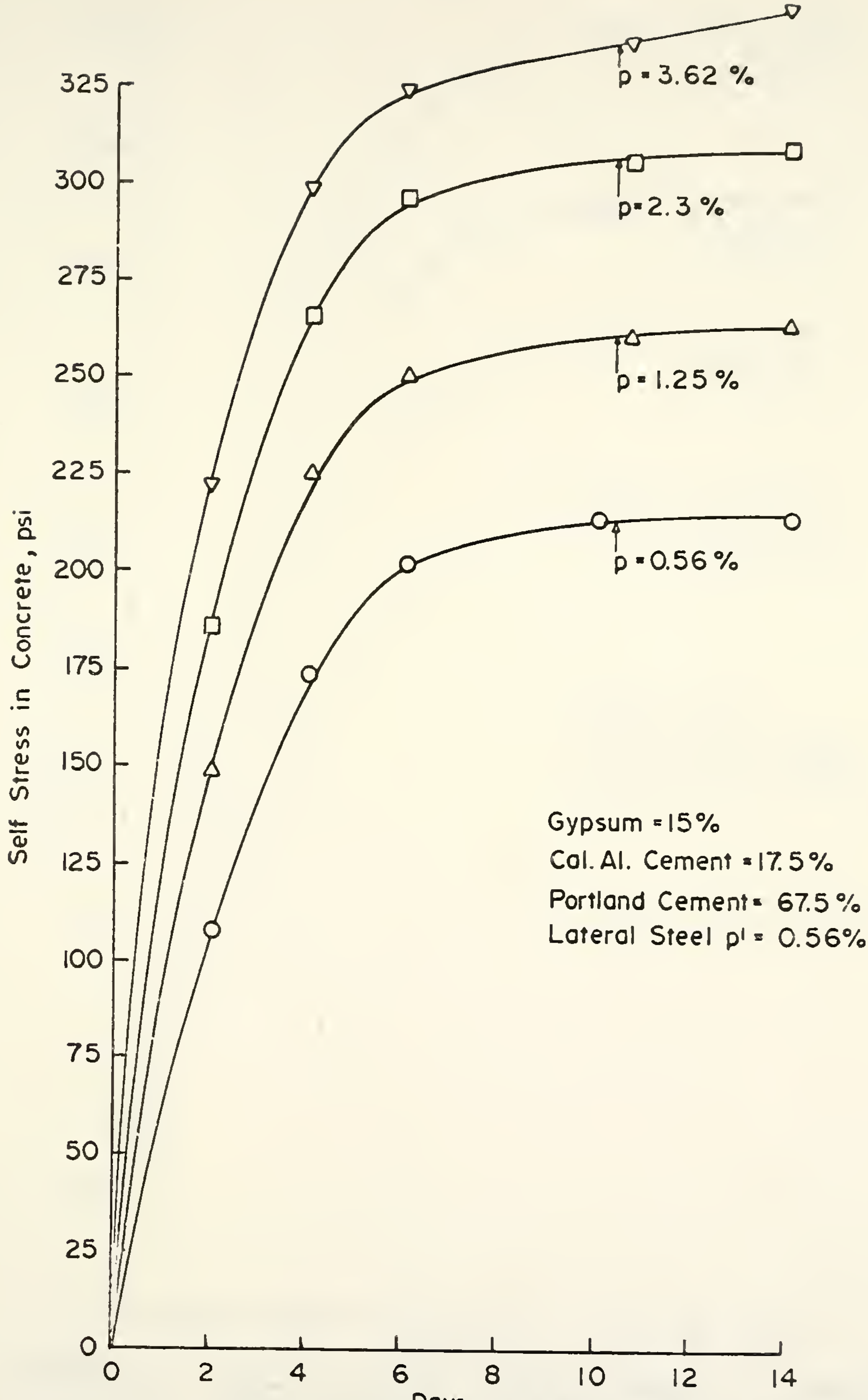


FIGURE 3-6 SELF-STRESS DEVELOPMENT IN TRIAXIALLY RESTRAINED HIGH EXPANSIVE CONCRETE SPECIMENS.

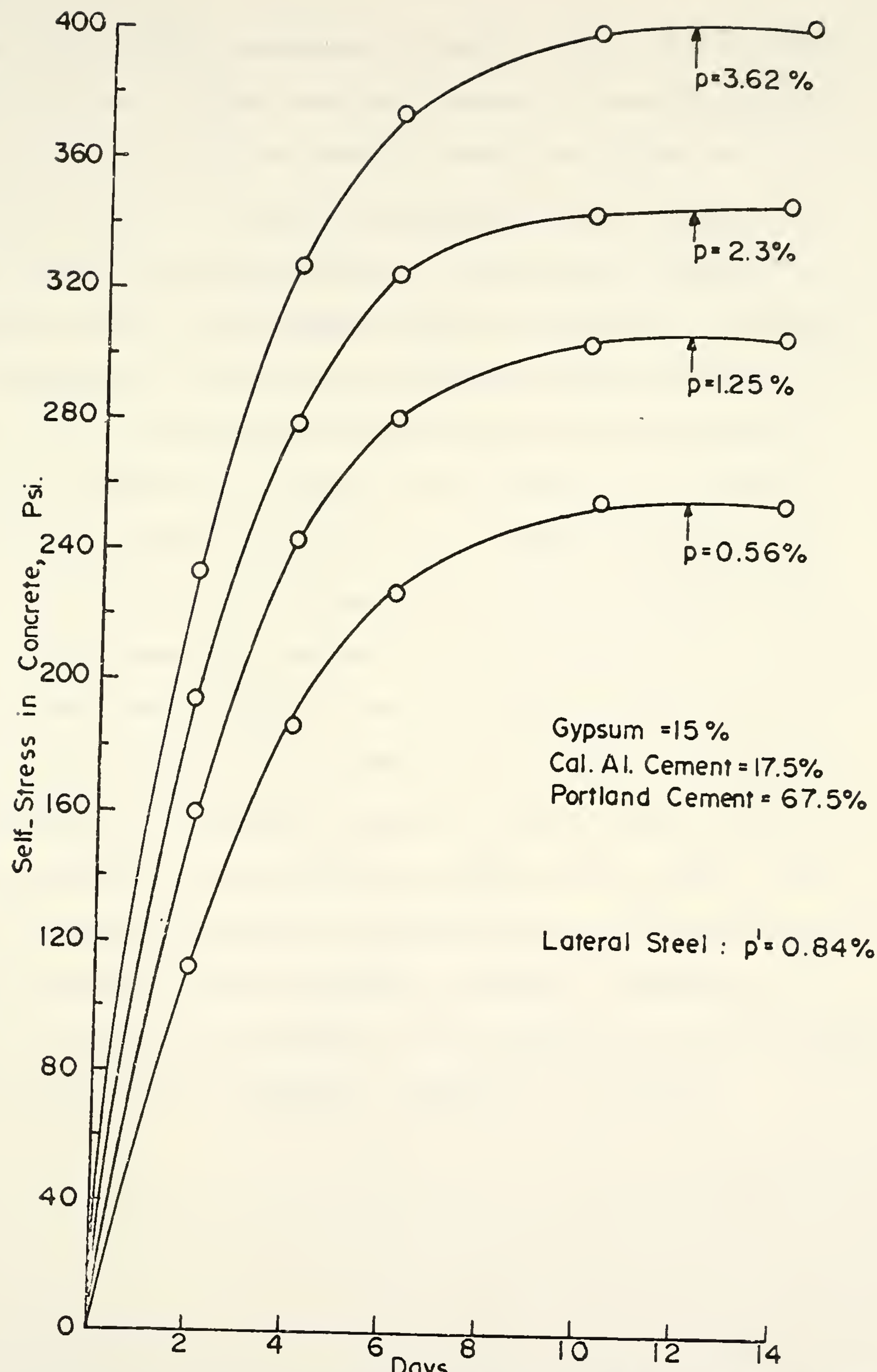


FIGURE 3-7 SELF-STRESS DEVELOPMENT IN TRIAXIALLY RESTRAINED HIGH EXPANSIVE CONCRETE SPECIMENS WITH PI = 0.84%

increases. But at higher percentages of lateral steel, the self-stress development may stay constant even with increase of lateral steel due to the fact that no more steel is needed to control the lateral expansion. For a further increase of this lateral restraints might result in a decrease of longitudinal self-stress due to less concrete involved in the same cross section. Very limited results are available in the literature. The results of Iido and Monji indicated that the development of self-stress for the lateral steel ratios of 0.63 and 0.82 was almost constant and a decrease of 15 percent of self-stress when the lateral steel percentage increased from 0.82 to 1.01. Their longitudinal reinforcements had about the same range as that used in the present study. However, in the present investigation, an increase of self-stress development was measured when the lateral steel percentage increased from 0.56 to 0.84 (Figure 3-6 and 3-7). For lateral steel higher than 0.84%, it becomes unrealistic for practical reinforced concrete design. Thus, an optimum ratio appears to be not within the range of our interest. The study made by Iido and Monji used pipe structures and a different type of expansive cement. Direct comparison is difficult. However, the question of an optimum ratio remains to be an important topic for further study when other types of triaxially restraints are employed.

CHAPTER 4 EFFECT OF ECCENTRIC RESTRAINTS ON SELF-STRESS DEVELOPMENT

Reinforcements in a concrete structural member are generally not axially-symetrical. Thus, the triaxially restraints introduced by the reinforcements for the development of self-stress are generally eccentrical. Fig. 4-1 shows the schematic diagram of a typical eccentrically restrained beam used in the present study. The details concerning the preparation and testing of the samples are referred to the Interim Report. Figure 4- 2 shows the calculated average longitudinal self-stress developed as a function of time. Since the electrical resistance gages were placed at the neutral axes, presumably no bending strains were recorded. Also, as a precautionary measure against bending, the holes of the end plates were 1/8 in. greater than the diameter of the rods. As expected, the self-stress development increases with increase of longitudinal steel percentage. Comparison of curves for eccentrically restrained specimens given in Figure 4-2 with those of triaxially restrained specimens shown in Figure 3-6 (both having 0.56 percent of lateral steel) shows a slight difference in self-stress development. The decrease in self-stress development in the case of eccentrically restrained specimens is probably due to the readjustment of the end plates in the early expansion stage.

Table 4-1 shows the significant difference between the top and bottom fiber expansion of eccentrically restrained specimens used in our investigation. Also, during this investigation curvature due to eccentricity was observed in the four different percentages of longitudinal steel. From the results it is concluded that different levels of self-stress can be achieved in a self-stressed concrete structure by reinforcing it

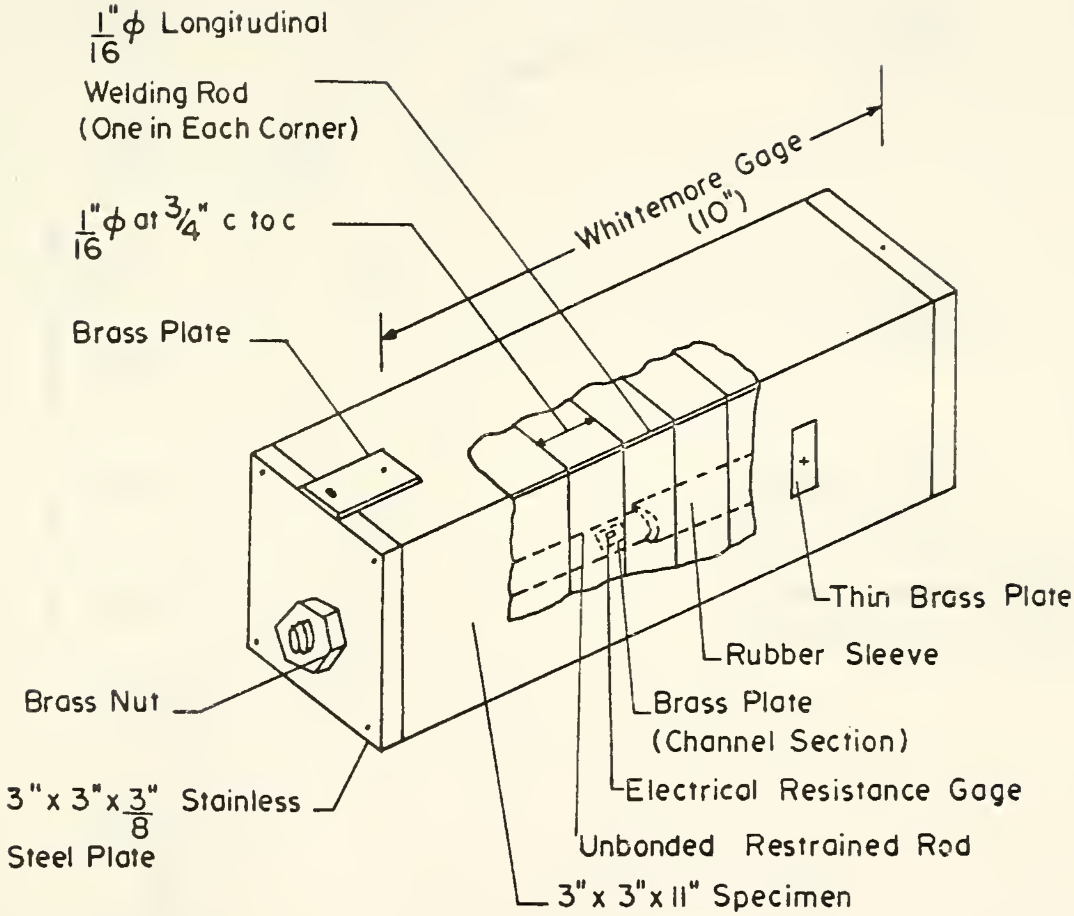


FIGURE 4-1 SCHEMATIC DIAGRAM OF ECCENTRICALLY RESTRAINED BEAM.

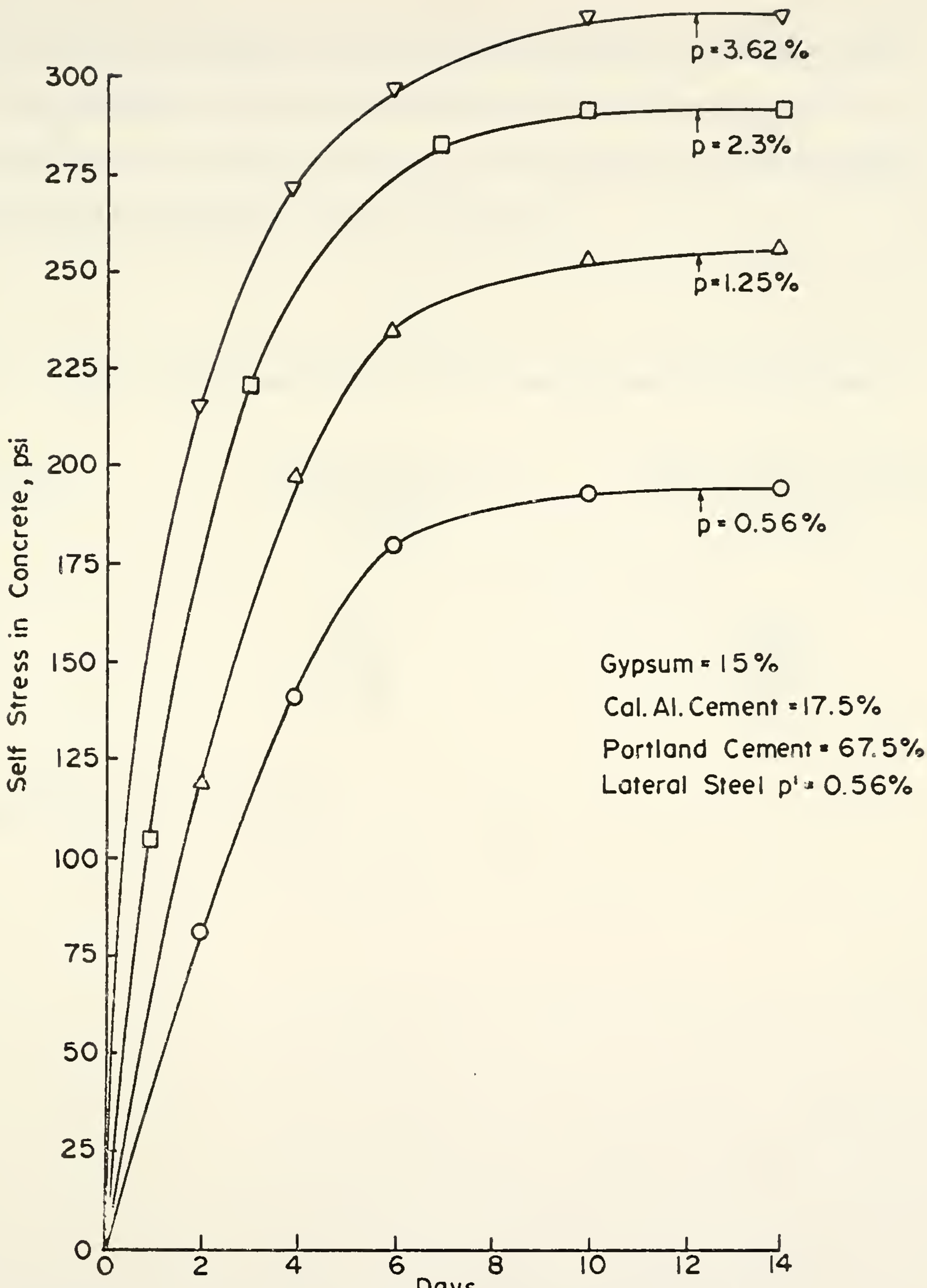


FIGURE 4-2 SELF-STRESS DEVELOPMENT IN ECCENTRICALLY RESTRAINED HIGH EXPANSIVE CONCRETE SPECIMENS.

eccentrically and the state of stress follows closely to the simple beam theory used commonly in the structural design. The different levels of self-stress causes the upward deflection at the center of the beam similar to eccentrically prestressed concrete structures.

TABLE 4-1. Longitudinal Expansion of Eccentrically Restrained Specimens

Long. steel (percent)	Top Fibre Expansion (percent)	Bottom Fibre Expansion (percent)
3.63	+0.610	-0.260*
2.30	+0.655	-0.258
1.25	+0.710	-0.254
0.56	+0.786	-0.244

* Negative means contraction.

CHAPTER 5 MECHANICAL PROPERTIES OF SELF-STRESSED CONCRETE AND THE STRUCTURAL BEHAVIOR OF CONCRETE BEAMS

The unrestrained expansion of concrete produces a large volume change and has generally poor mechanical properties (compressive strength and modulus of elasticity). In an uniaxially restrained concrete, due to the expansion perpendicular to the direction of restraint, the mechanical properties also failed to show any improvement. A promising application of self-stressing cement concrete thus lies in the utilization of its three dimensional expansion to provide three dimensional prestressing of concrete and to achieve better mechanical properties. In this investigation the effect of triaxial restraints on mechanical properties was studied. Also, the method of testing self-stressed concrete for its mechanical properties was evaluated.

The samples used in the study were triaxially restrained cylinders. A schematic diagram of the sample is shown in Fig. 5-1. All cylinders were tested in compression in accordance with ASTM C-39 to obtain stress-strain relationships.

Since the cylinders were tested without removing the lateral restraints (helix), the strength contributed from the helix to the compressive strength of the concrete was first evaluated. This information was obtained by casting conventional portland cement concrete cylinders with and without the helix. The stress-strain relationships obtained from the compression tests of these cylinders are shown in Figure 5-2. The stress difference between these two curves is taken as the strength contributed by the helix. All modified stress-strain curves of which the helix effect has been removed, are shown in Figure 5-3. In the same figure, the stress-strain

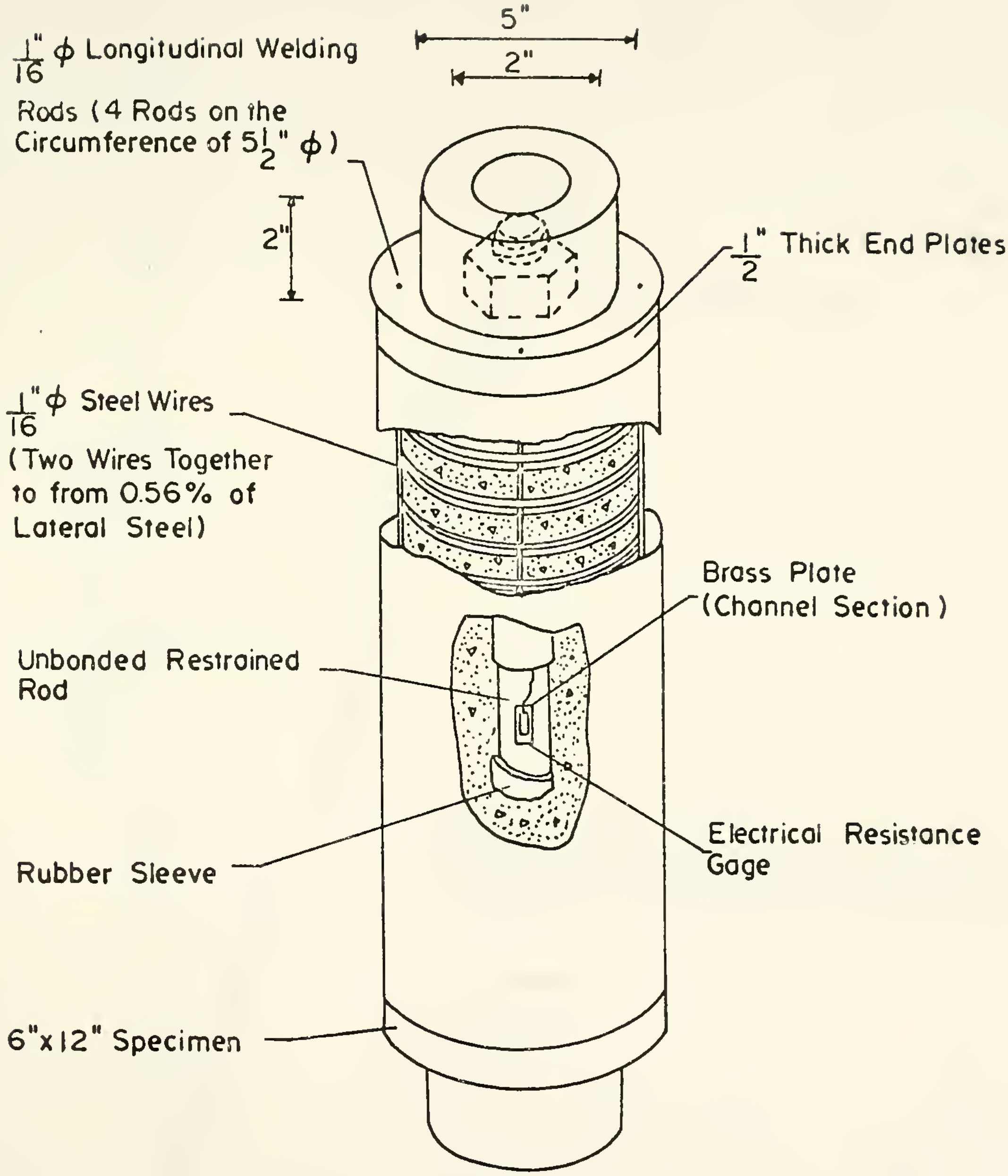


FIGURE 5-1 SCHEMATIC DIAGRAM OF TRIAXIALLY RESTRAINED CYLINDER.

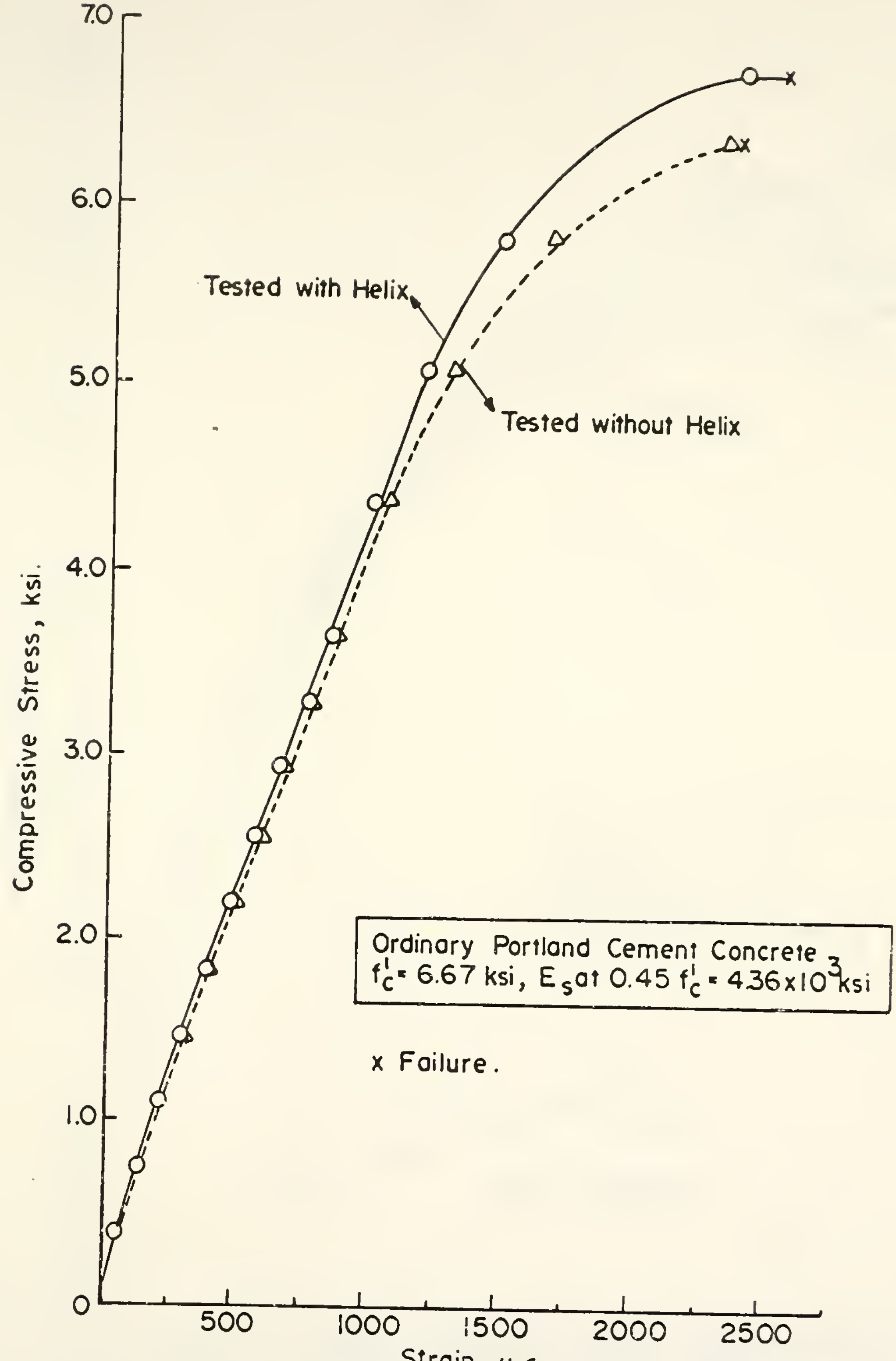


FIGURE 5-2 CONCRETE STRESS-STRAIN DIAGRAM FOR CYLINDERS WITH AND WITHOUT HELIX.

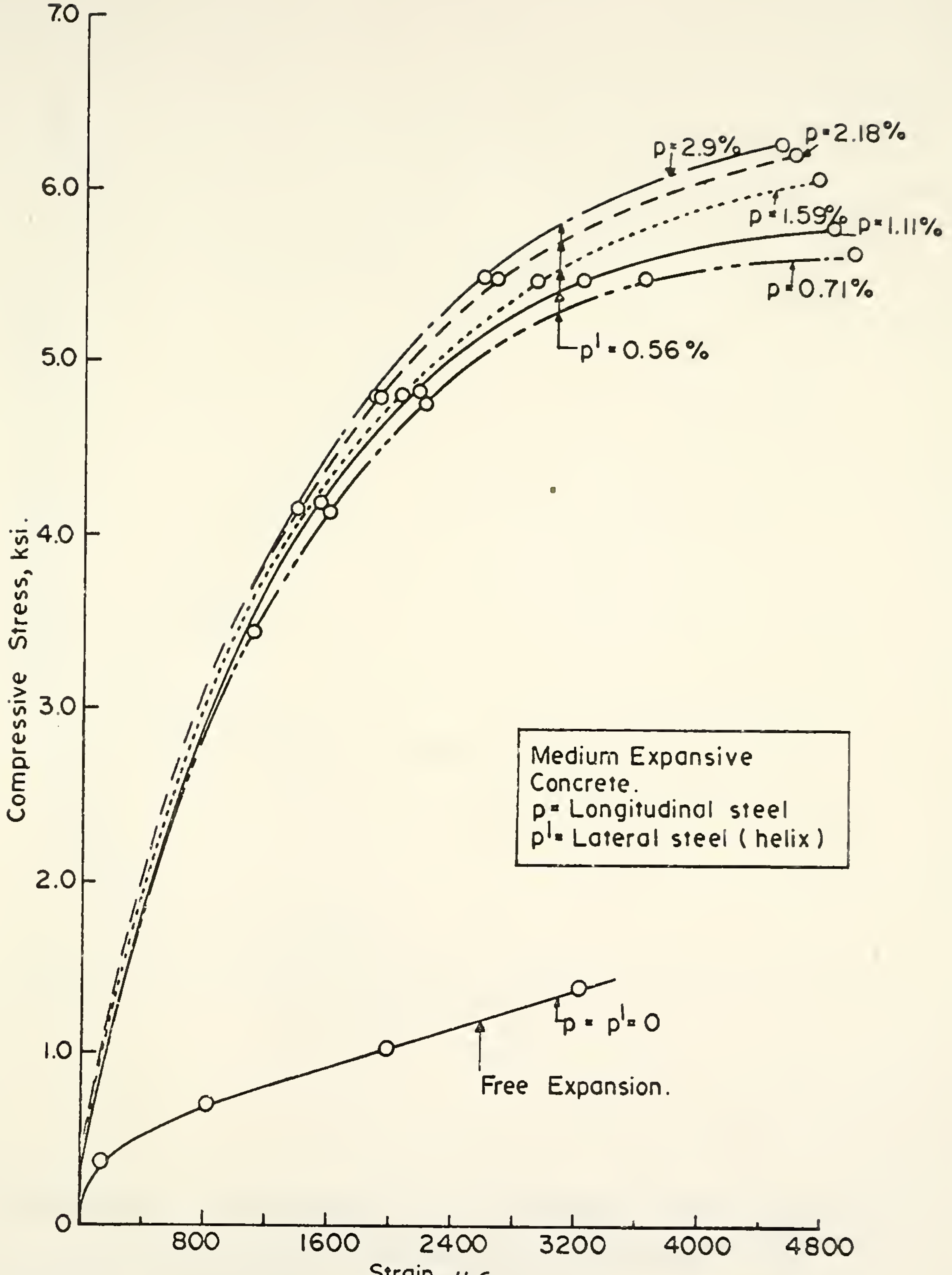


FIGURE 5-3 MODIFIED STRESS–STRAIN DIAGRAM OF COMPRESSION TEST CYLINDERS.

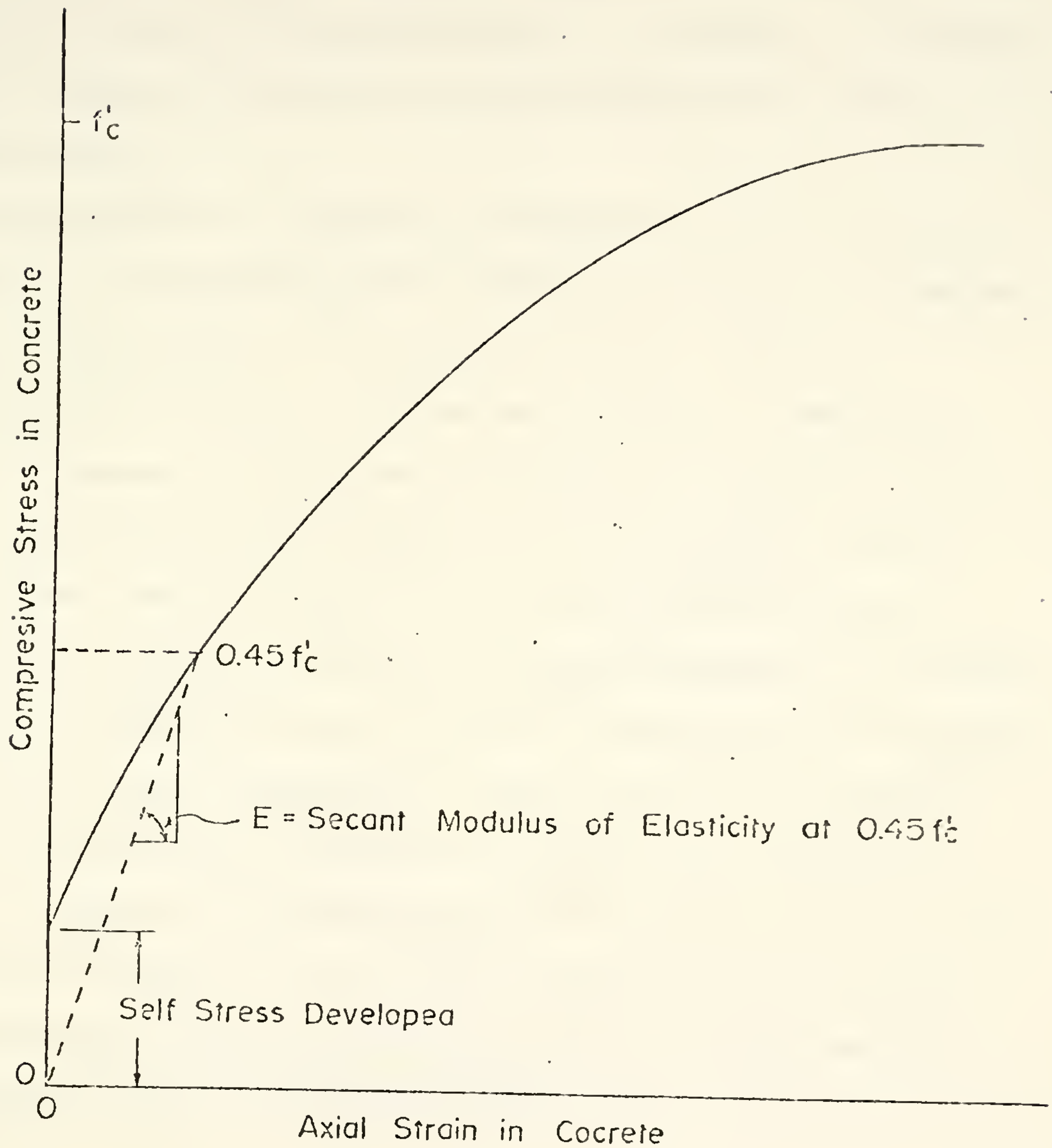


FIGURE 5-4 CONTEMPLATED STRESS-STRAIN CURVE FOR COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY.

relationship of a free expanded concrete cylinder is also shown. Each curve in Figure 5-3 represents the average values of two specimens. The variation of results obtained in these two tests was generally within 5 percent of the average values.

The secant modulus of elasticity is calculated at 0.45 f'_c and shown in Figure 5-4. The final magnitude of self-stress developed, the compressive strength and secant modulus of elasticity are shown in Table 5-1.

From Figure 5-3 and Table 5-1, the results indicate a significant increase in compressive strength and secant modulus of elasticity in triaxially restrained specimens as compared to the free expanded concrete. Thus, even with a small percentage of lateral and longitudinal steels, properties were improved. In triaxially restrained specimens, for different percentages of longitudinal steel the modulus of elasticity and compressive strength did not change significantly. However, due to the expansion of the self-stressed concrete, the modulus of elasticity and compressive strength of triaxially restrained specimens were slightly lower than those of the ordinary concrete. This is a major limitation to the utilization of highly expansive concrete for self-stressing, i.e. high strength and large expansions are not mutually obtainable.

Table 5-1 Analysis of Compression Test Cylinders

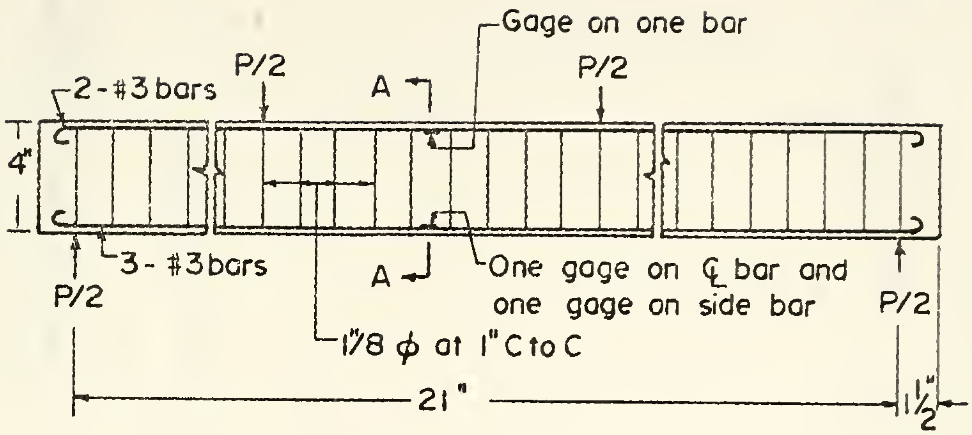
Percentage of Longitudinal Restraint, P, Percent	Self-Stress Developed, psi	Compressive Strength f'_c at 28 days, psi	Secant Modulus of Elasticity, E, at 0.45 f'_c, million psi
0	0	1460	0.91
0.71	183	5660	3.75
1.11	200	5770	3.82
1.59	219	6130	3.94
2.18	228	6200	4.10
2.9	246	6310	4.18

The advantages of using self-stressing cement in reinforced concrete structures are, in general, similar to the conventional prestressed concrete. From the theory of prestressed concrete it is well-known that the development of compressive stress reduces deflection and improves the mechanical behavior of structural members. In this investigation, the structural behavior of self-stressed concrete beams was studied. The reinforcement details and the placement of gages used in the beam are shown in Figure 5-5. The beams were 3" x 4" in cross section and 24" in length and cast in rigid steel molds.

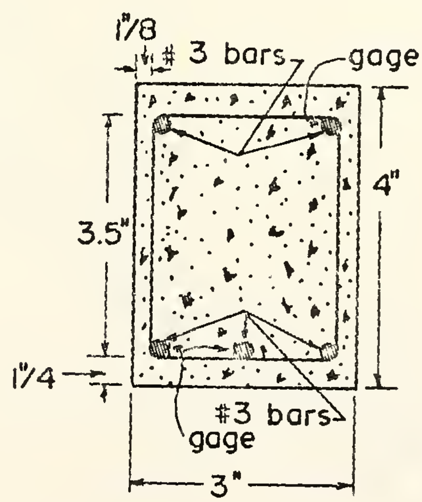
A total of six beams, two for the ordinary concrete, two for the medium expansive concrete, and two for the high expansive concrete were prepared on the same day. Water sufficient to produce 2 to 3 inch slump was used in each mix. Beams were cured up to 21 days and tested in flexure.

Figure 5-6 shows the load versus the calculated deflection for the ordinary portland cement concrete, medium expansive and the high expansive concrete beams. The deflections were calculated in the following order: First, the curvature of the beam was calculated by dividing the sum of the measured strains of concrete and steel by the structural depth. Then, by using the conjugate beam method, the central deflections were calculated.

Figure 5-7 shows the load versus measured deflection for ordinary concrete and self-stressed concrete beams. The deflection were measured at the center of the beam with a dial gage having a least count of 0.001 in. The data given are the average values of two tested beams and the variation in two tests was within 10 percent of the average value.



Side View



Section A-A

FIGURE 5-5. REINFORCEMENT DETAILS.

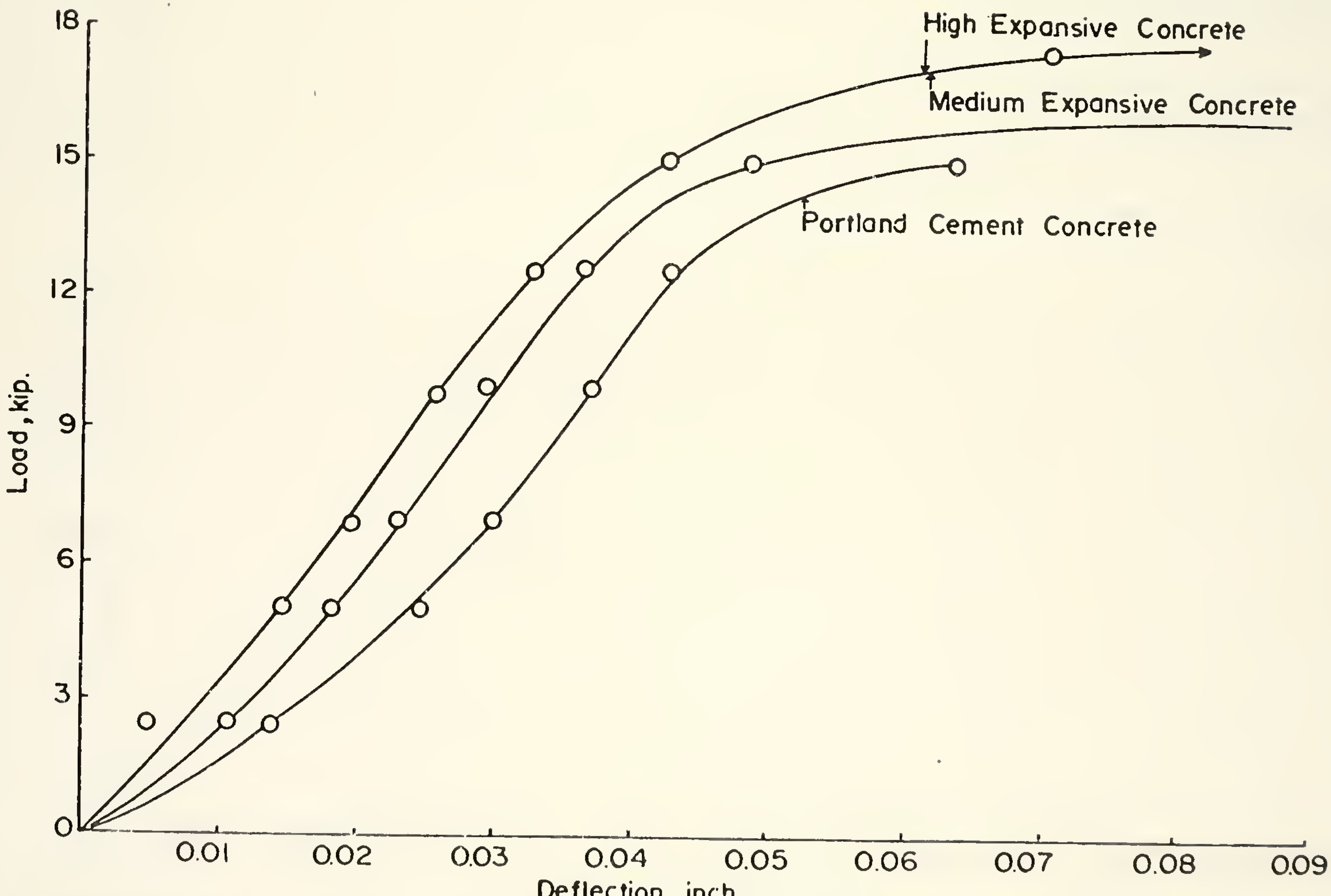


FIGURE 5-6 LOAD VS CALCULATED DEFLECTION CURVES.

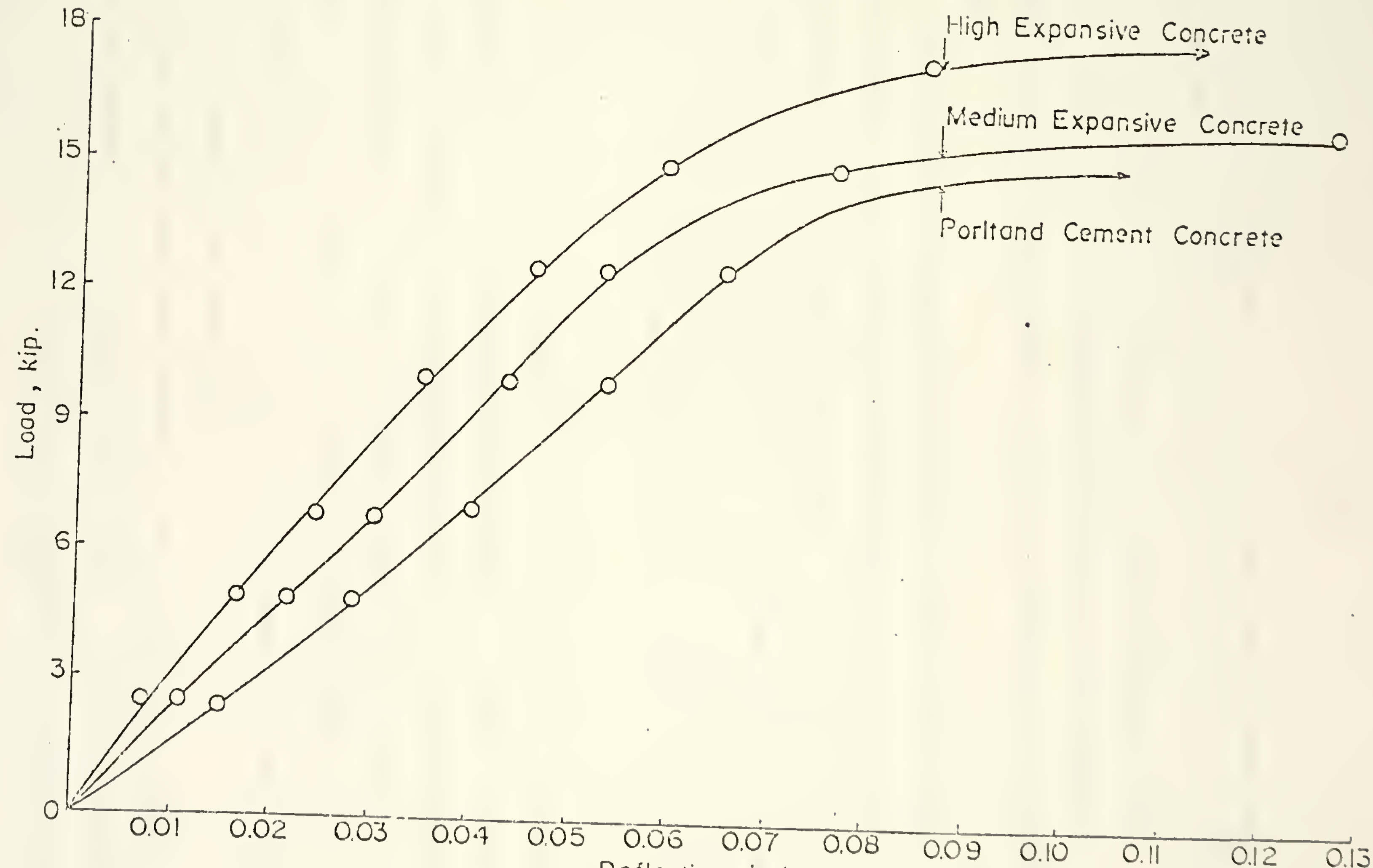


FIGURE 5-7 LOAD VS MEASURED DEFLECTION CURVES.

In view of the behavior of the tested beams, two different stages were observed. The first stage represents the part of the load deflection curve up to the yielding of the steel. It can be considered as the elastic range. The second stage was characterized by an increase of deflection without load increment. Thus, the steel was in the plastic range.

A comparison of the measured and calculated deflections shows a close agreement in the early stage of the loading. However, the difference is significant when the steel becomes plastic. At the failure of the beam, the strains in the steel were calculated by using the concrete strains at the failure load and the neutral axes at yield. This procedure was followed because the gages had failed before the ultimate load were reached.

Comparison of curves for different concretes shows that the measured deflections as well as the calculated deflections for ordinary concrete were larger than those for self-stressed concrete as contemplated before the investigation. Furthermore, the appearance of crack patterns of ordinary and self-stressed concrete beams are significantly different. The self-stress development in medium and highly expansive concrete beam were 520 psi and 644 psi, respectively.

Higher failure loads were recorded in the self-stressed concrete beams as compared to the ordinary concrete beams. This higher strength was due to the existence of prestress and the higher steel stress at failure strain.

The elastic limits of the load-deflection curves are approximately 12.5 kips and 15 kips (Figures 5-6 and 5-7). From the Figure 5-6, the corresponding deflection for the load 12.5 kips in ordinary concrete was 0.043 inches. The corresponding load for the same deflection in high expansive concrete beam is about 15 kips. Therefore, for the same deflection, the increase in the ultimate load is about 20 percent. The ultimate increase in the ultimate load shown in Figure 5-7 for the measured values is about 24 percent. Hence, it may be expected that a possible saving in steel is about 20 percent in the case of self-stressed concrete for the same limiting deflection or for the same ultimate load.

PART II:

STRUCTURAL BEHAVIOR OF SELF-STRESSED REINFORCED CONCRETE COLUMNS

CHAPTER 6

INTRODUCTION

Statement of Problem

When properly reinforced a concrete member made with expansive cement concrete can develop a beneficial triaxial self-stress. Proper reinforcement is achieved when the volume increase of the expanding concrete is restrained in all directions. Conventional deformed steel reinforcing bars in the longitudinal direction with ties or a spiral helix for lateral restraint are ways of obtaining three dimensional confinement of the expansion. When the concrete expands the reinforcing steel restrains the expansion, thus causing tensile stresses in the longitudinal and lateral steel and this, in turn, creates triaxial compressive stresses in the concrete. The triaxial condition of prestress is what makes this method of prestressing potentially more valuable than conventional prestressing and post-tensioning methods which develop only uniaxial prestress.

The amount of self-stress that is developed in the concrete depends on several factors. The important factors are:

1. Amount of components in the cement which cause the concrete to expand.
2. Amount of longitudinal reinforcement.
3. Amount of lateral reinforcement.
4. Air entrainment in concrete.
5. Curing conditions such as humidity and temperature.

Objectives and Purpose of the Investigation

The purpose of this investigation was to obtain a better understanding of the structural and material behavior of reinforced Type M expansive cement concrete.

The investigation was comprised of the following studies:

1. A comparison of the structural behavior of expansive cement concrete to that of regular portland cement concrete.
2. A comparison of tied and spiral reinforcement arrangements as lateral restraint for expansive cement concrete.
3. A study of the effects of air entrainment on the expansion, strength and durability properties of expansive cement concrete.
4. A study of the time of set properties of expansive cement.

Preview of Experiments

The experiments in this investigation included column tests, beam tests, freeze-thaw tests and time of set tests.

In the column tests loads were applied concentrically to 5 x 5 x 21 inch specimens. The variables were type of cement, type of lateral reinforcement and air entrainment.

In the beam tests loads were applied at the midpoint of a 19 inch span on 5 x 5 x 21 inch specimens. The variables were type of cement and type of lateral restraint.

Seventy-one freeze-thaw cycles were performed on three air entrained, expansive cement concrete specimens and three air entrained, portland cement concrete specimens. The size of the specimens was 3 x 4 x 16 inch.

Time of set tests were performed on expansive cement mortars and portland cement mortars. The variables were water to cement ratio, mix water temperature and type of gypsum.

Table 6-1 shows all the experiments performed in this investigation excluding the time of set tests which appear in Chapter 8.

Throughout this investigation Type M self-stressing cement and Type I portland cement were used. Tables 6-2 and 6-3 show the mix designs of the concretes used in this investigation.

Table 6-1 Summary of Mixes.

Mix Code	Number and Type of Specimen	Type of Cement	Tests Performed
1	2 Tied	Portland Type I	Air content, slump, column tests
2	2 Tied	Expansive	Air content, slump, expansion, column tests
3	2 Spiral	Portland Type I	Air content, slump, column tests
4	2 Spiral	Expansive	Air content, slump, expansion, column tests
5	2 Tied	Expansive (air entrained)	Air content, slump, expansion, column tests
6	2 Spiral	Expansive (air entrained)	Air content, slump, expansion, column tests
7	2 Tied	Portland Type I	Air content, slump, beam tests
8	2 Tied	Expansive	Air content, slump, expansion, beam tests
9	2 Spiral	Portland Type I	Air content, slump, beam tests
10	2 Spiral	Exapnsive	Air content, slump, expansion, beam tests
11A1	2 Freeze-Thaw	Portland Type I (air entrained)	Air content, slump
11B1	2 Freeze-Thaw	Expansive (air entrained)	Air content, slump
11A2	3 Freeze-Thaw	Portland Type I (air entrained)	Air content, slump, freeze thaw
11B2	3 Freeze-Thaw	Expansive (air entrained)	Air content, slump, freeze thaw
12	1 Spiral, 1 Tied	Portland Type I	Air content, slump, column test, beam test

Table 6-2 Proportions for One Cubic Yard Mixes.

Mix Numbers / Materials	1, 3, 7, 9, 12	11A1, 11B1	2, 4, 8, 10	5, 6, 11A2, 11B2
Cement	855 lb	855 lb	855 lb	855 lb
Fine Aggregate (oven dry)	1387 lb	1183 lb	1397 lb	1183 lb
Coarse Aggregate (oven dry)	1119 lb	1119 lb	1119 lb	1119 lb
Water	388 lb	388 lb	400 lb	400 lb
Vinsol Resin Solution	---	2.4 lb	---	2.4 lb

Table 6-3 Cement Proportions.

Mix Numbers	1, 3, 7, 9, 11A1, 11A2, 12	2, 4, 5, 6, 10 11B1, 11B2
Percent of Portland Type I	100	65
Percent of Calcium Aluminate	0	20
Percent of Gypsum	0	15

CHAPTER 7

MATERIALS AND PREPARATION

Materials

Cements and Gypsum

The Type I portland cement used in all the experiments was from a single clinker batch. The designation for the portland cement was No. 323 in the Joint Highway Research Project Concrete Laboratory at Purdue University.

The calcium aluminate cement was obtained in one shipment and used throughout all experiments.

The gypsum was a fine ground food and pharmaceutical grade hydrous calcium sulfate. This differs from previous expansive concrete experiments performed at the Joint Highway Research Project at Purdue in that reagent quality gypsum had been used in the past.

Aggregates

The coarse aggregate used throughout the experiments was a limestone of size No. 11 (designation by the Indiana State Highway Standard Specifications). The fine aggregate was a local glacial-alluvial sand. The gradation and absorption were found according to ASTM Standard Tests and were assumed to remain constant throughout all the mixes.

Water

Ordinary tap water was used in all the mixes.

Admixtures

A vinsol resin solution of 13.46% by weight was used for all mixes where air entrainment was desired.

Reinforcing Steel

The main longitudinal reinforcement in all of the specimens was No. 3 deformed reinforcing bars with a nominal yield strength of 50,000 psi. The actual properties appear in Figure A1 in Appendix I.

The lateral reinforcement in the tied specimens was a cold rolled 1/8 inch diameter wire with a yield strength of 61,100 psi and an ultimate strength of 69,300 psi.

The lateral reinforcement in the spiral specimens was a cold rolled 1/16 inch diameter wire with a yield strength of 70,600 psi and an ultimate strength of 80,200 psi.

Common hot rolled steel was used for all the end plates.

Fabrication

5 x 5 x 21 Inch Tied Specimens

The reinforcement cages for the tied column and beam specimens were each made up of eight #3 deformed bars arranged as shown in Figures 7-1 and 7-2. The bars were

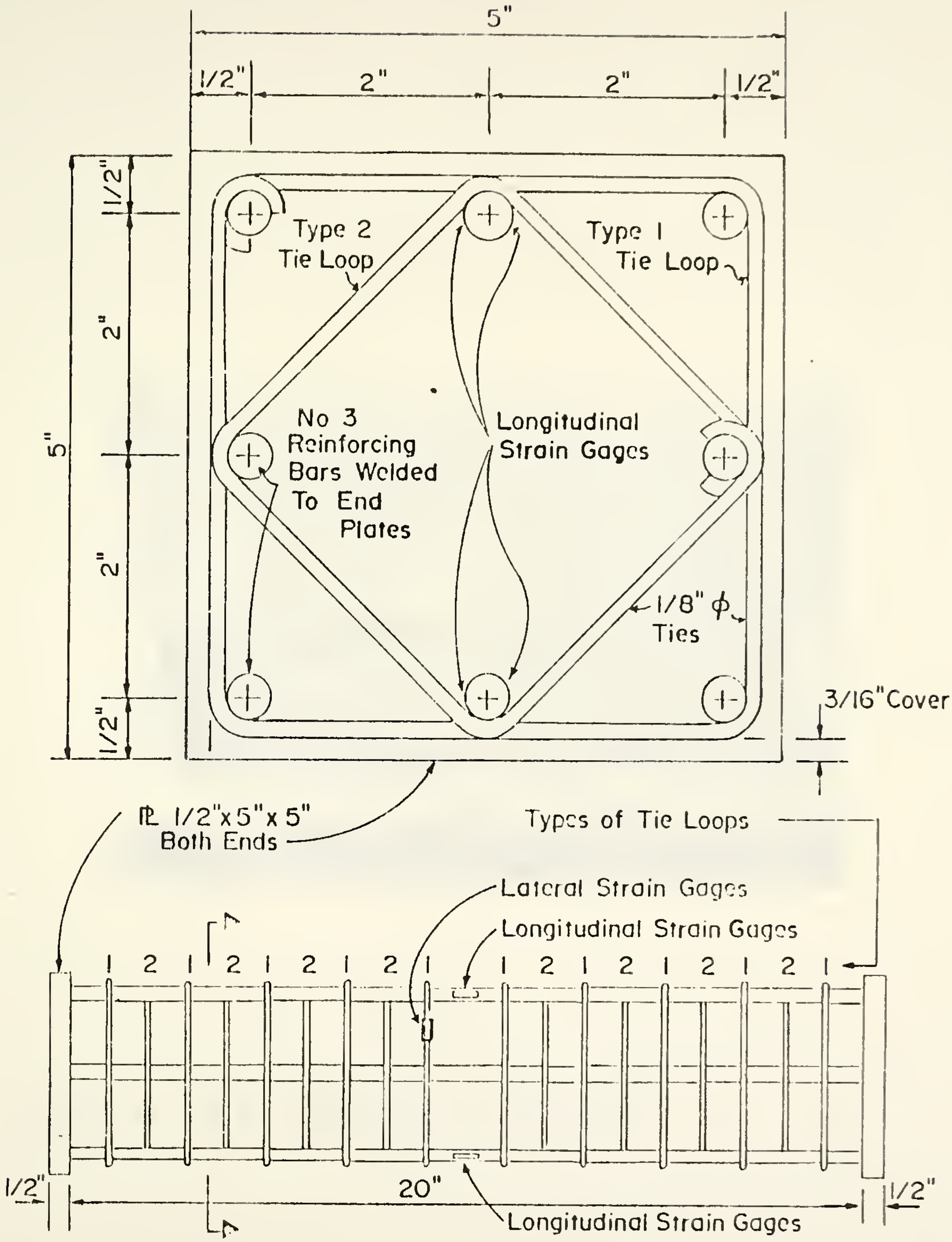


FIGURE 7-1 TIED SPECIMEN REINFORCEMENT ARRANGEMENT

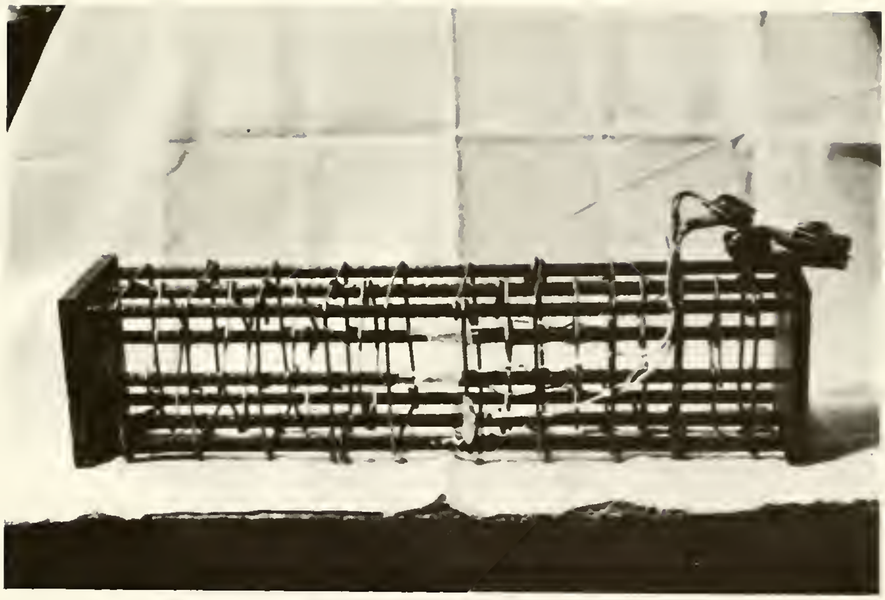


Figure 7-2. Reinforcing Cage for Tied Specimen.

welded to 5 x 5 x 1/2 inch end plates at either end of each specimen. The welds were plug type welds opposite the specimen side of each plate as shown in Figure 7-3. The tie arrangement for each cage was made up of ten 1/8 inch diameter wires connecting the four corner bars and nine 1/8 inch diameter wires connecting the middle bars of the faces. The ties were spaced 1 inch apart alternating the two types of ties at every inch.

Four electrical resistance strain gages were used to obtain an average of the longitudinal strain on each specimen. They were mounted on opposing sides of two opposing bars at the midpoint of each specimen as shown in Figure 7-1. The deformations on the bars where the gages were to be mounted were lathed off to provide smooth mounting surfaces. For all of the specimens this reduction in steel cross-sectional area was less than 5%. The reductions were taken into account for the self-stress calculations and concrete stress calculations. Two smaller strain gages were placed on opposing sides of a tie near the midpoint of the specimen to obtain averages for the lateral strains. The strain gages were waterproofed and protected against vibration damage.

5 x 5 x 21 Inch Spiral Specimens

The reinforcement cages for the spiral column and beam specimens were each made up of eight #3 deformed bars arranged as shown in Figures 7-4 and 7-5. The bars were welded to end plates in the same manner as previously mentioned. The spirals were double spirals of 1/16 inch diameter wire with a pitch of 5/8 inch. The total percentage of lateral reinforcement was 0.873% in each specimen.

Four electrical resistance strain gages were used to obtain an average of the longitudinal strain on each specimen. They were mounted on opposing sides of two opposing bars at the midpoint of each specimen as shown in Figure 7-4. One strain gage was placed on each of the two spirals of each specimen near the midpoint of each specimen to obtain average lateral strains. The actual sizes of the different gages are illustrated in Figure 7-6.

3 x 4 x 16 Inch Freeze-Thaw Specimens

To obtain reinforcement for the restraint of the expansive concrete of the freeze-thaw specimens it was decided, at first, to use a completely internal cage for each specimen. When this was tried the unrestrained exterior expansive concrete at the ends of the specimens severely cracked and spalled off during normal curing in a fog room. It was then decided to utilize the end plate type arrangement that was used for the column specimens. The reinforcing cage was made with four corner #3 deformed bars welded to end plates. The ties were 1/8 inch diameter wires spaced at 1 inch as shown in Figures 7-7 and 7-8. No strain gages were mounted on the steel of the freeze-thaw

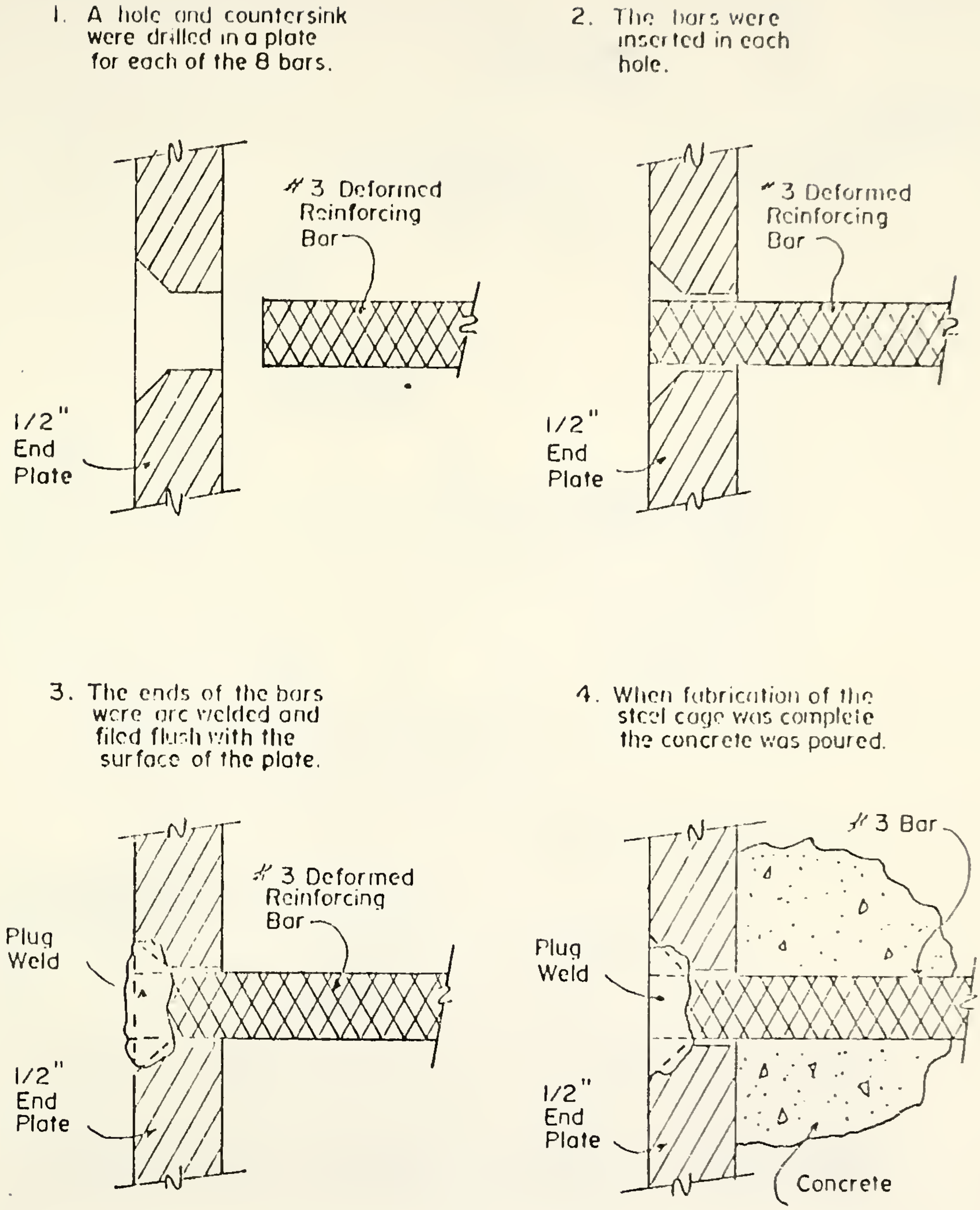


FIGURE 7-3 ILLUSTRATION FOR CONNECTION BETWEEN END PLATE AND REINFORCING BAR

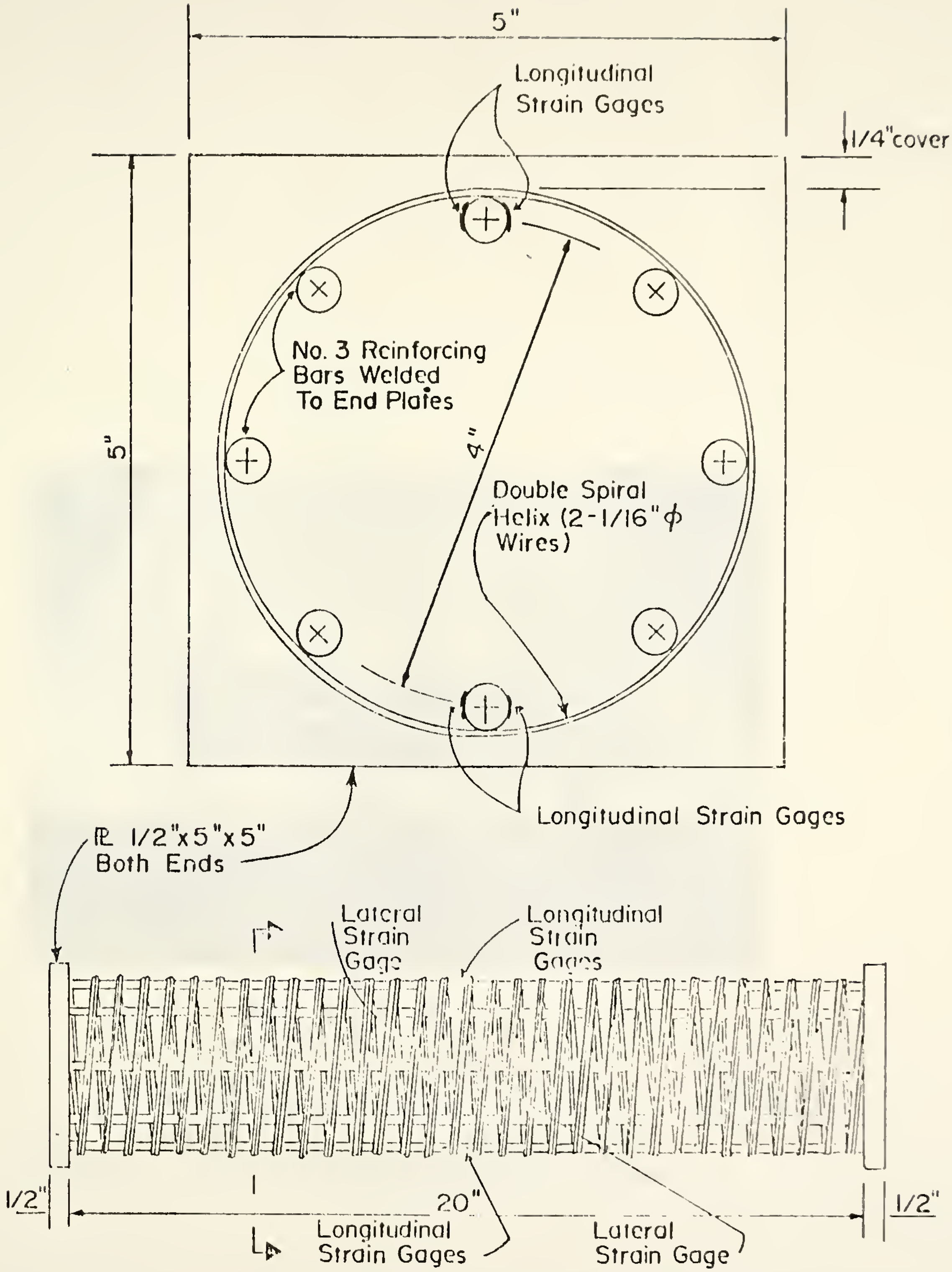


FIGURE 7-4 SPIRAL SPECIMEN REINFORCEMENT ARRANGEMENT

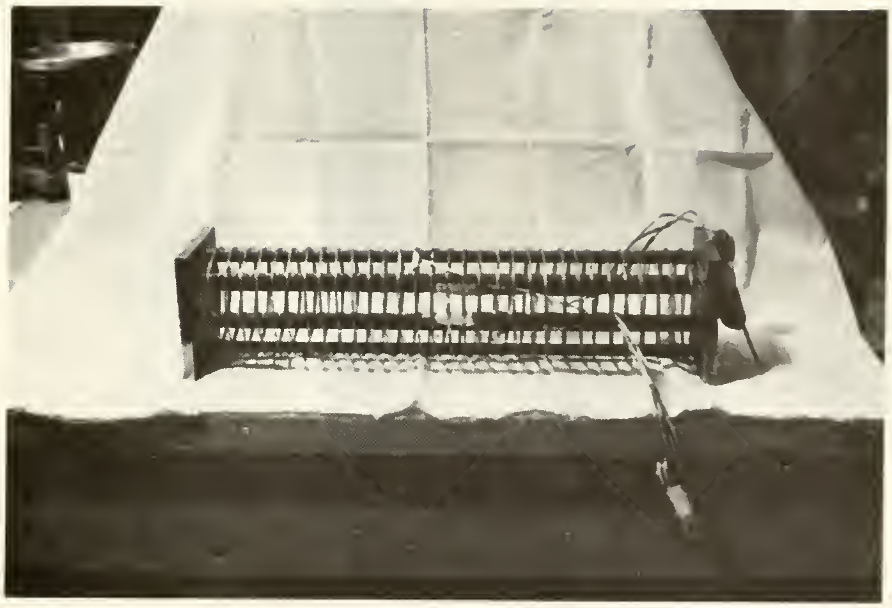


Figure 7-5. Reinforcing Cage for Spiral Specimen.

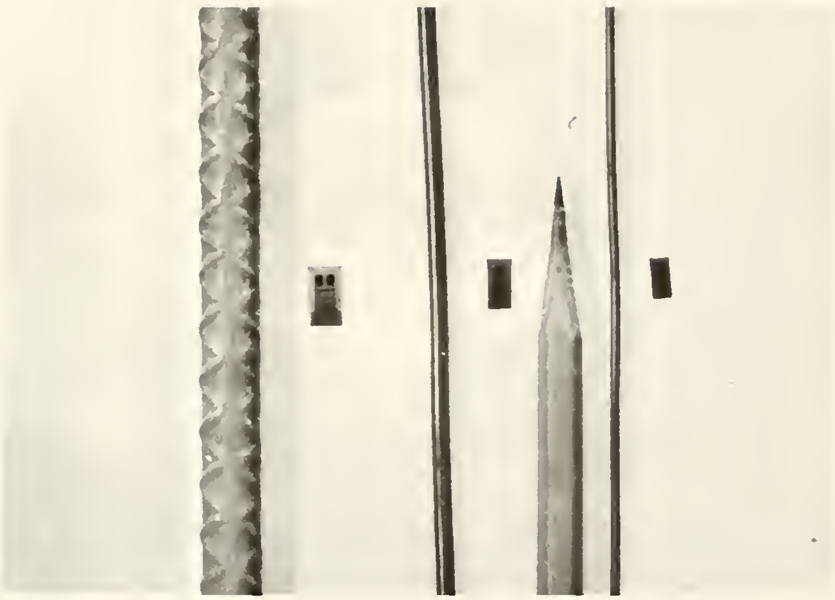


Figure 7-6. Typical Reinforcing Steel and Strain Gages Used. Left to Right: #3 Reinforcing Bar, EA-06-125AD-120 Strain Gage, 1/8 Inch Wire, EA-06-062DF-120 Strain Gage, Pencil, 1/16 Inch Wire, EA-06-031DE-120 Strain Gage. (All Gages Manufactured by Micro-Measurements, Div. of Vishay Intertechnology, Inc.)

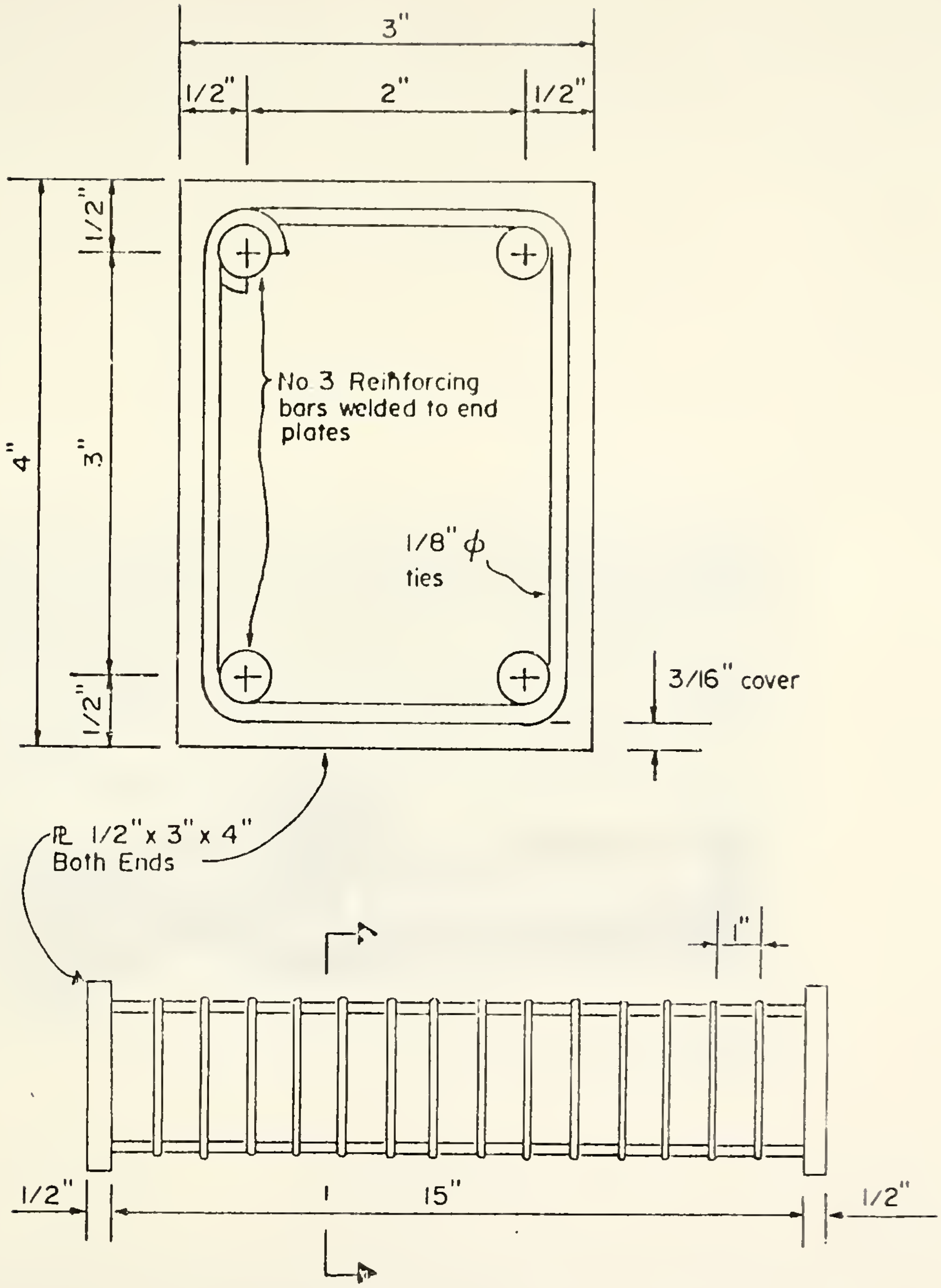


FIGURE 7-7 FREEZE-THAW SPECIMEN REINFORCEMENT ARRANGEMENT

Figure 7-8. Reinforcing Cage for Freeze-Thaw Specimens.

specimens. It was assumed that the expansive properties would be similar to those exhibited by the air entrained tied column specimens which had nearly the same percentage of longitudinal steel.

Mixing and Casting

The mixes for the portland cement concrete control specimens were made following the laboratory procedures of ASTM Standard Specification C 192. In summary, the mixer was "buttered" and the coarse aggregate along with some water for absorption was put in the mixer. The mixer was then started and the sand, cement and the remainder of the water were added. The water was at room temperature. The mixer was allowed to run for 3 minutes, and after a 3 minute rest, 2 more minutes. The specimens were cast around the steel cages in steel forms and then vibrated until well consolidated. A slump test and a volumetric air content test were performed for each mix. Figures 7-10 and 7-11 show the apparatus used in mixing and casting.

When the first mix with expansive concrete (Mix 2) was made the procedure described above was used. Due to the high room temperature on that day and the tendency for the expansive concrete to set much faster than regular concrete, a flash set occurred and there was barely enough time to cast the column specimens. The slump test and the air content test could not be performed.

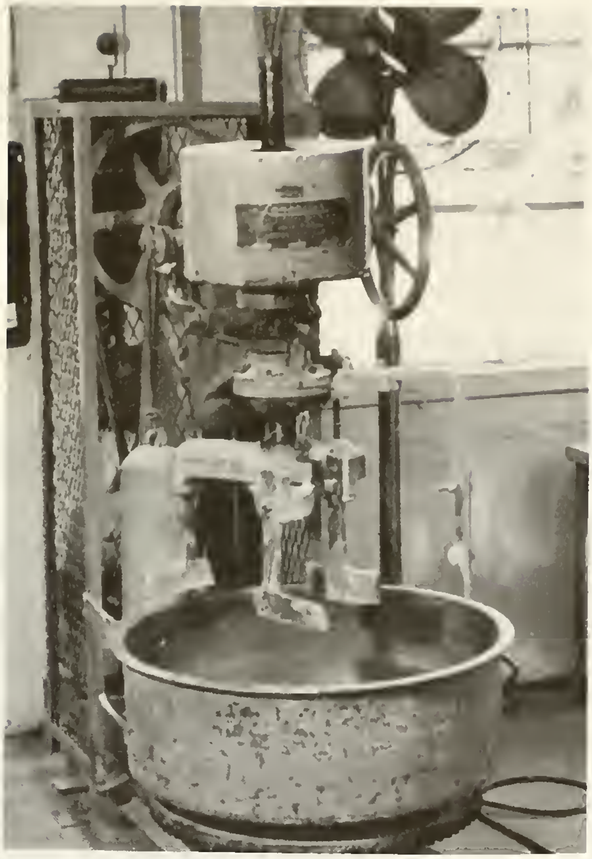


Figure 7-9. Mixing Machine.

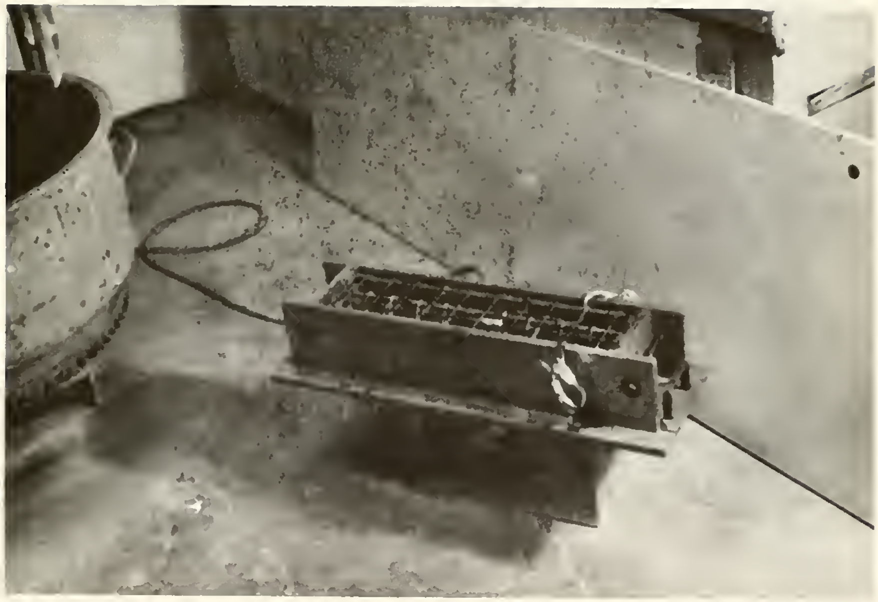


Figure 7-10. Reinforcing Cage in Form on Vibrating Table Ready for Casting.

Figure 7-11. Volumetric Air Meter.

This prompted the decision to adopt a different mixing procedure for the remainder of the expansive mixes. Also, some time of set tests were carried out to verify the tendency of the expansive concrete toward flash set.

The new procedure for the expansive mixes differed from the old one in that the mixing time was reduced to four minutes with no rest period and the mix water was cooled to 8°C just prior to mixing. The vinsol resin admixture used in the mixes requiring air entrainment was added to the dry ingredients before the water was added.

Some preliminary trial mixes revealed that the consistency of a given expansive concrete mix would typically be much more harsh than that of normal concrete with the same water to cement ratio. Thus, the water to cement ratio of the expansive concrete mixes was increased above that of the regular concrete mixes so that the expansive concrete could be consolidated around the reinforcing cages which were heavily congested with closely spaced lateral reinforcement. This increase of water in the expansive mixes improved their consistency and workability but the consistencies of the expansive mixes were still much more harsh than those of normal concrete mixes. See Table 7-4 for a summary of the characteristics of the fresh concrete.

Table 7-4 Characteristics of Fresh Concrete.

Mix Number	Type of Cement in Concrete	Water to Cement Ratio	Temperature of Mix Water	Air Content	Slump	Consistency
1	portland	0.40	room temp.	1 1/2%	7 in.	wet
2	expansive	0.42	room temp.	FLASH SET		very harsh
3	portland	0.40	room temp.	1 1/4%	8 in.	wet
4	expansive	0.42	8°C	2%	3 in.	good
5	expansive	0.42	8°C	6 1/2%	4 in.	good
6	expansive	0.42	8°C	7 1/2%	4 1/4 in.	good
7	portland	0.40	room temp.	1 1/4%	7 1/2 in.	wet
8	expansive	0.42	8°C	2%	4 in.	good
9	portland	0.40	room temp.	1 1/4%	8 in.	wet
10	expansive	0.42	8°C	2%	2 in.	harsh
11A1	portland	0.40	room temp.	7%	8 in.	wet
11B1	expansive	0.40	room temp.	7%	1 in.	very harsh
11A2	portland	0.42	8°C	7%	10 in.	very wet
11B2	expansive	0.42	8°C	8%	4 in.	good
12	portland	0.40	room temp.	1 3/4%	9 in.	wet

Instrumentation

Accurate measurement of the expansions of the confined expansive concretes was a major concern. The size of the expansion from the day of mixing and casting through the day of testing the columns was so small that mechanical gaging could not provide precise data. Through the use of electrical resistance strain gages the expansions at arbitrary localized points in each specimen could be measured with a resolution of one micro-strain (10^{-6} in/in or $\mu\varepsilon$).

Stability of the Ten Channel Strain Indicator

In order to monitor the change in strain of the gages mounted on each expansive specimen it was necessary that the instrumentation used to measure strain gage output be free from drift. The instrument used for this was a Vishay/Ellis switch and balance unit with its mated counterpart, a digital readout as shown in Figure 7-12. This system has a ten channel capacity for strain gages.

The performance of the strain indicator was checked for drift over a period of several weeks. Four electrical strain gages were mounted to a steel specimen and placed in a temperature controlled room. The Vishay/Ellis system was placed next to the specimens, turned on for several hours, wired to the gages, and then balanced to indicate strains of zero micro-strains. During the

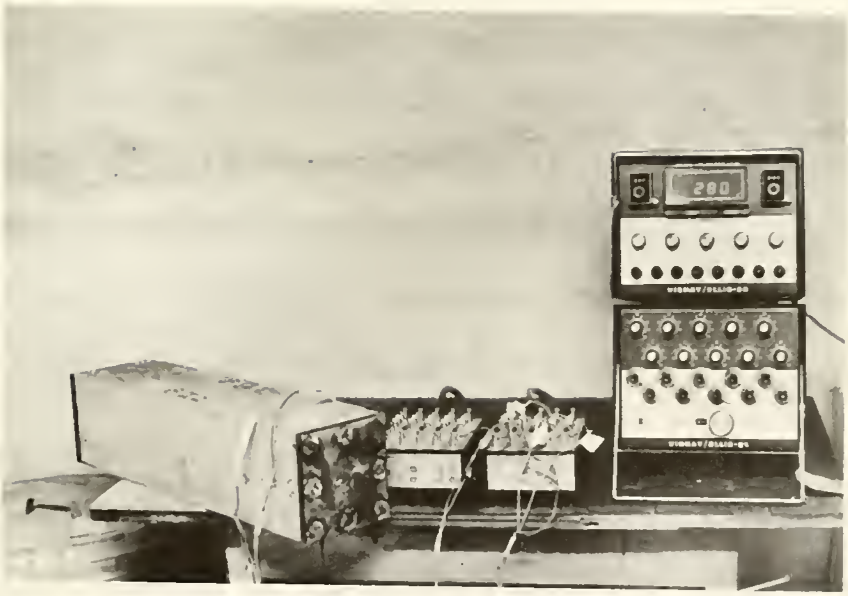


Figure 7-12. Specimen Connected to Ten Channel Strain Indicator, Vishay/Ellis-20 and 21, for Expansion Readings.

following two weeks the system went untouched except to turn the digital readout unit on and off at the beginning and end of each day. At intervals of two to four hours the room temperature and indicated strain readings were monitored. The mean room temperature was 71°F with a maximum variation of ± 1°F. The maximum deviation from the zero setting was four micro-strains. This was found only once on one gage. Meanwhile, the other three gages were still near zero. This deviation was encountered early in the day just after the unit had been turned on. Shortly thereafter the readings reapproached zero. From the results of this investigation it was determined that the stability of the Vishay/Ellis ten channel strain indicator was sufficiently reliable to make strain readings over the period of many days.

Modification of the Ten Channel Strain Indicator

A simple modification was made to the Vishay/Ellis ten channel strain indicator to enable it to interpret more than ten gages concurrently. On the switch and balance unit, the knobs used to balance out the strain readings to zero were replaced with ten-turn calibrated knobs. Thus, at the beginning of the expansion period for each specimen, for each gage the settings of the knobs indicating zero strain could be recorded and then reset at any later date.

The procedure for monitoring the expansive strains for a typical specimen with six gages is outlined below:

1. These substeps were followed for the first readings.
   a. After casting the expansive concrete, connect the six gages to six of the ten channels on the ten channel strain indicator.
   b. Zero the instrument.
   c. For each of the six channels set the initial reading at zero using the balance control and set the span control at the proper level to permit direct readout of the strain in micro-strain. Span settings are determined from the manufacturer's specified gage factor.
   d. Recheck step c. until both readings are satisfactory.
   e. For each gage, record the channel assignment and the zero setting calibrations on the coarse adjustment dial and on the ten-turn calibrated knobs.
2. Note that during the interval between the first reading and the second, another specimen may have been connected to the same strain indicator channels as shown in step 1. Thus, all of the knobs will probably have to be returned to their initial settings.

a. After an interval of time has passed from the initial reading, reconnect the specimen's six strain gages to the same six channels as before.

b. Reset the coarse adjustment and ten-turn calibrated knobs to the settings recorded initially in step 1. d.

c. Adjust the span control for each channel so that the reading is equal to the indicated strain plus the initial span setting.

d. Read the strain directly.

3. All subsequent readings simply repeat the procedure of step 2.

CHAPTER 8

EXPERIMENTAL INVESTIGATION

Expansion Tests

The rates of expansion for all the specimens for the first 2 days were similar. After that time the expansion rates and magnitudes varied considerably. The general shapes of all the expansion versus time curves were the same. The curves began with a high rate of expansion then came to a maximum which was followed by contraction presumably due to creep. The curve then leveled off and a constant prestress was held. All this occurred within a 14 day curing period for all the specimens in a fog room with temperatures between 74°F and 80°F. Each point on the curves was obtained by averaging four longitudinal strain readings for each of the two identical specimens in one mix. Then those two strain values were averaged. Thus, one curve per mix was obtained.

Tied Specimens

The specimens with tied lateral reinforcement exhibited longitudinal strains as high as 535 $\mu\varepsilon$. For Mix 2 (non-air entrained) the average longitudinal strain leveled off after contracting due to creep at 460 $\mu\varepsilon$ or

552 psi self-stress in the concrete as shown in Figure 8-2. (In determining the values for self-stress of the concrete in all the specimens, the areas of the badly cracked cover concrete were excluded in the calculations since the poor quality cover concrete was assumed to contribute very little to the longitudinal expansion of the specimens). For Mix 8 (non-air entrained) the leveling off strain was about 320 $\mu\varepsilon$ or 388 psi self-stress in the concrete. For Mix 5 (air entrained) the expansion strains were considerably lower with a maximum average of only 250 $\mu\varepsilon$ after creep, and the corresponding self-stress being only 231 psi.

Spiral Specimens

The specimens with spiral lateral reinforcement expanded more and attained a higher self-stress in the concrete than the tied specimens. The spiral specimens exhibited more cracking on the cover concrete than the tied specimens, especially along the edges. This was due to the fact that a considerable portion of the cross section at the corners was unrestrained laterally.

For Mix 4 (non-air entrained) a self-stress in the concrete of 932 psi was the highest attained of all the mixes. The self-stress after creep was about 790 psi for both non-air entrained spiral mixes. The expansion and self-stress obtained in Mix 6 (air entrained) were

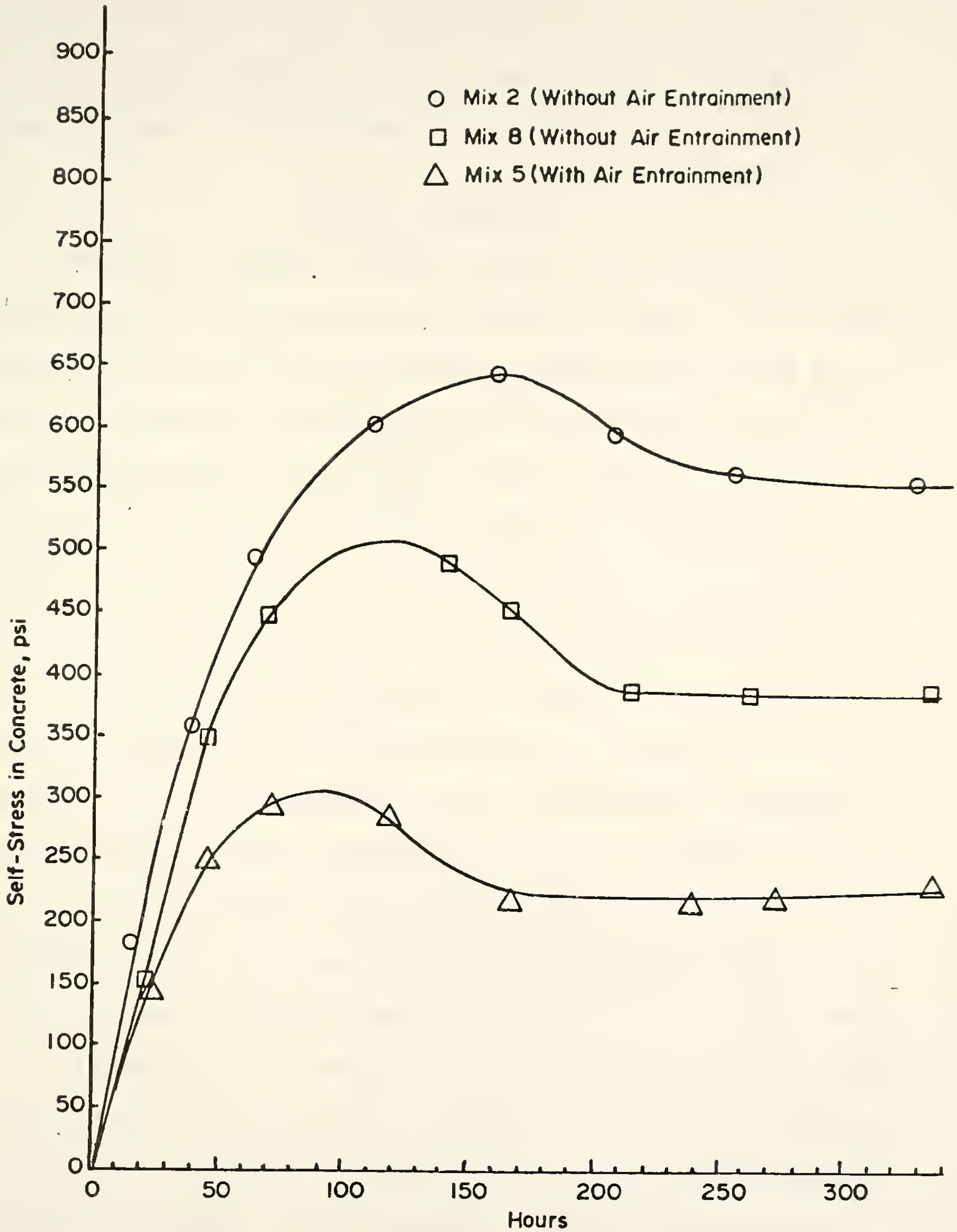


FIGURE 8-2 LONGITUDINAL SELF-STRESS IN CONCRETE OF TIED SPECIMENS

considerably lower than those in the non-air entrained mixes. After creep only 275 $\mu\varepsilon$ and 454 psi self-stress were attained. See Figures 8-3 and 8-4.

Lateral Expandions

Finding a way to correctly measure the lateral expansion in the specimens was very difficult. The lateral expansion could not be accurately measured by using any external mechanical gages because the unrestrained cover concrete expanded considerably more laterally than the internal restrained concrete. Finding the strains of the restrained concrete within the lateral reinforcement was all that was of significance since the poor quality cover concrete of the expanded specimens was of little value in regards to the self-stress attained and the strength of the specimens. So it was decided to attempt measuring the lateral expansion strains by means of small electrical reisitance strain gages mounted on the lateral reinforcement. This was a major undertaking in that the extremely small size of gages that would fit on 1/8" and 1/16" diameter wires made application and protection from moisture and vibration very difficult. Most of the gages on the 1/8" diameter ties gave consistent and reasonable readings but most of the readings from the gages on the 1/16" spiral wire provided very little useful data. The results of some of the more reliable tests are shown in Figure 8-5.

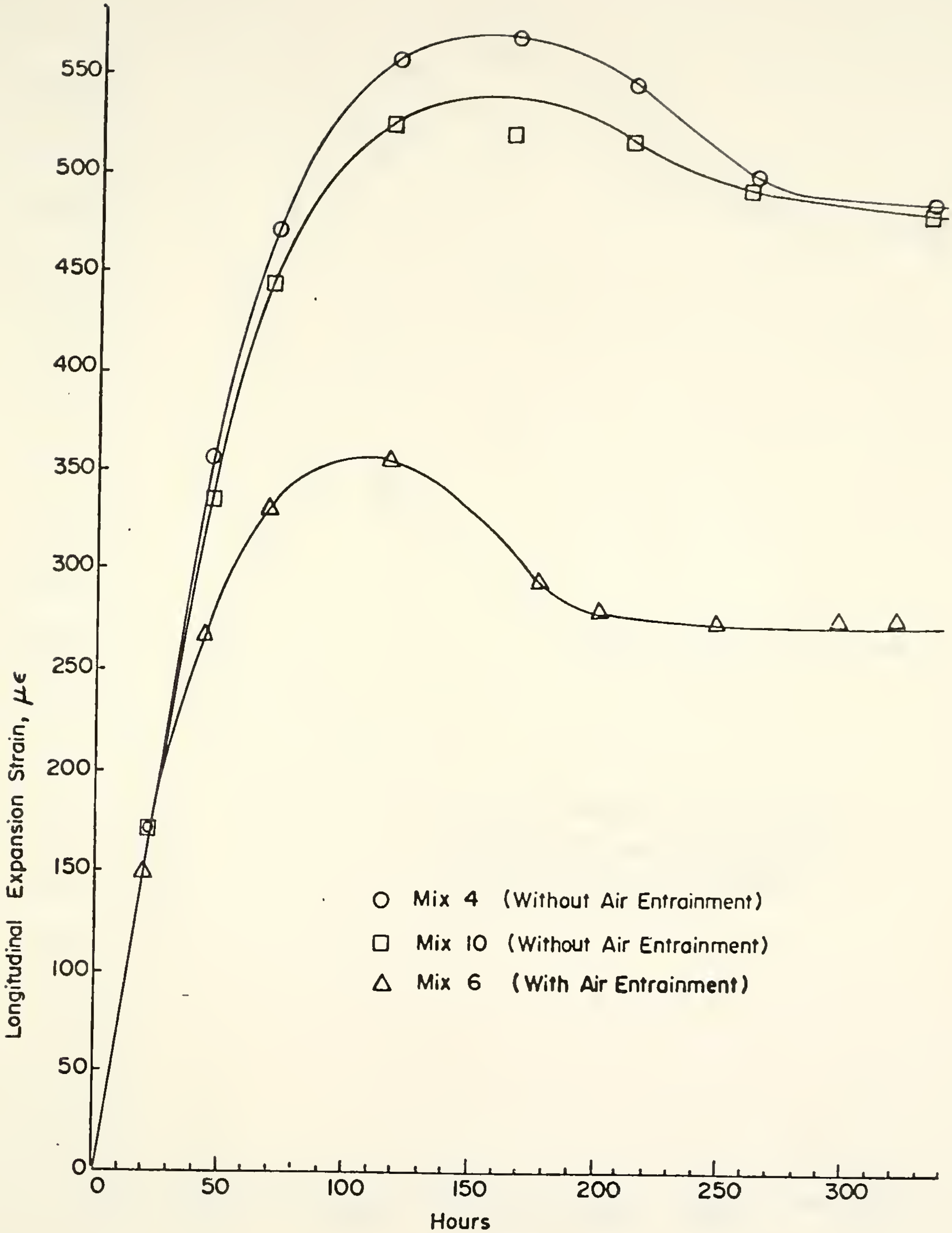


FIGURE 8-3 LONGITUDINAL EXPANSION STRAINS FOR SPIRAL SPECIMEN MIXES.

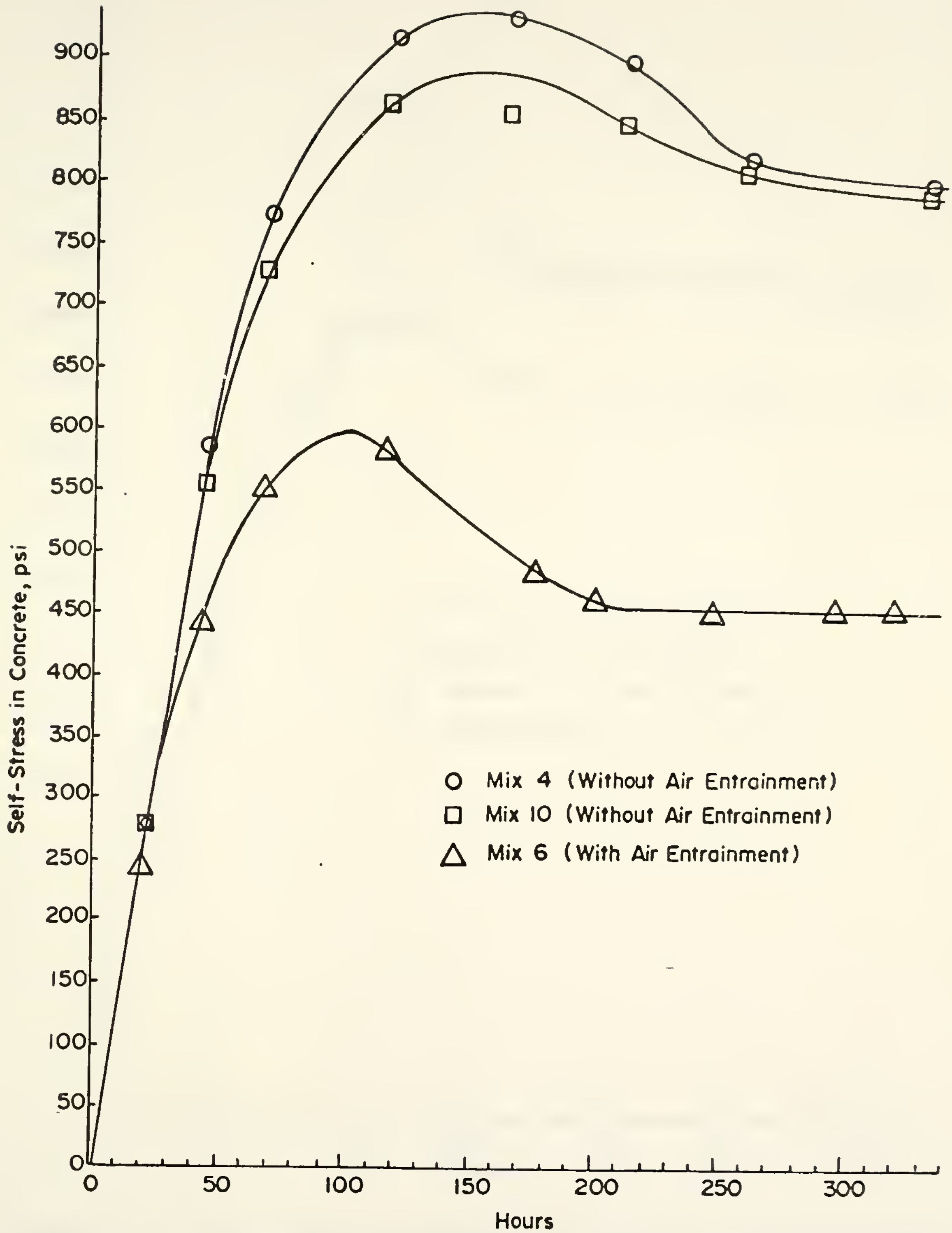


FIGURE 8-4 LONGITUDINAL SELF-STRESS IN CONCRETE OF SPIRAL SPECIMENS

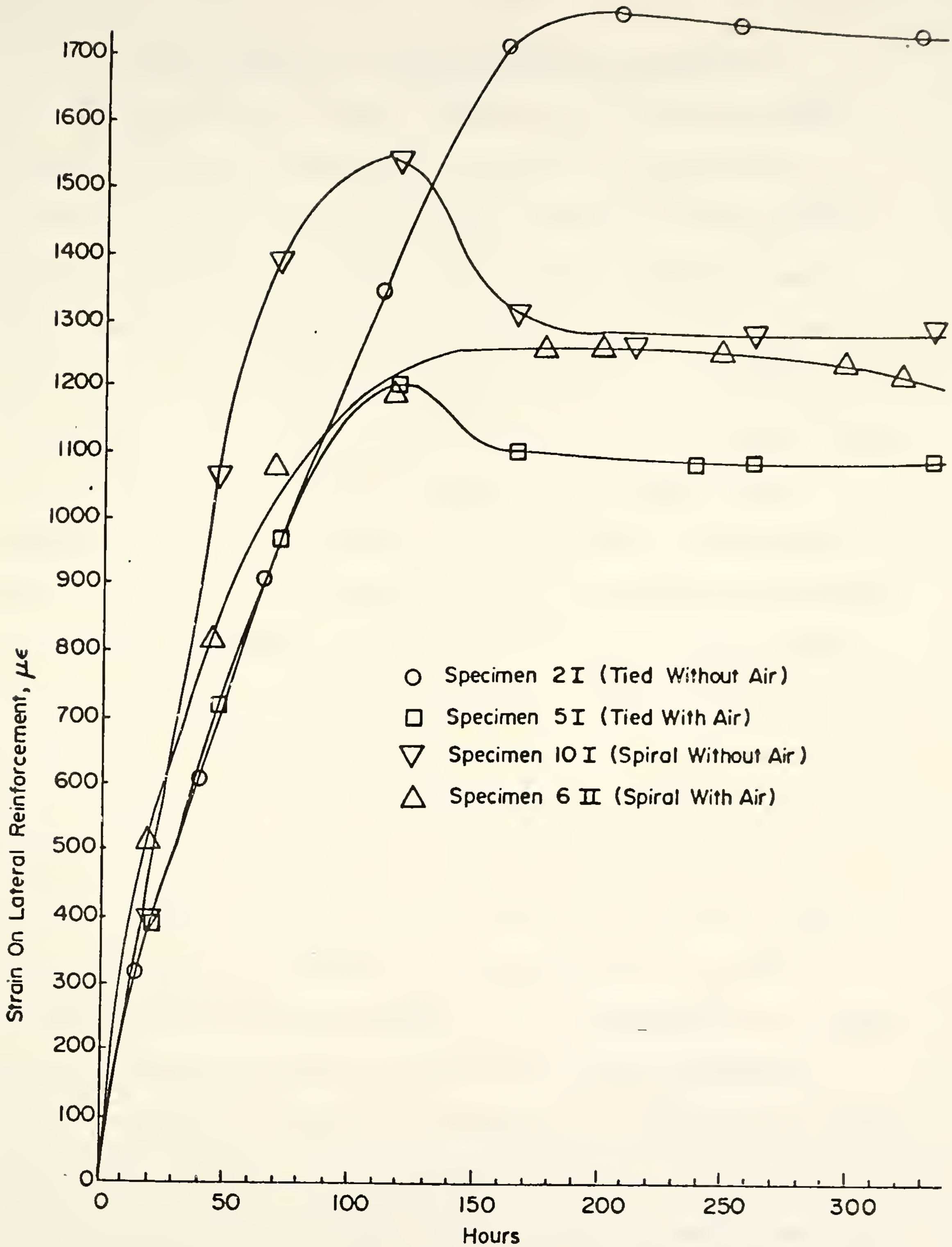


FIGURE 8-5 EXPANSION STRAINS ON LATERAL REINFORCEMENT OF FOUR SPECIMENS.

Column Tests on Tied and Spiral Specimens

Concentrically loaded column tests were performed on 13 specimens. Each strain point was obtained by taking an average of the readings from the four longitudinal strain gages mounted on the steel reinforcing of each specimen. Using the idealized steel stress-strain properties (see Appendix I), the measured prestrain in the steel, and the cross-sectional area of the steel the load carried by the steel at each strain point was obtained for each specimen. This yielded a load versus strain curve for each specimen that represented the load carried by the steel. Then using the total load versus strain relationship for a specimen, which was obtained in the column test, and subtracting the steel portion of the load carried, a load versus strain relationship was obtained for the concrete. The load carried by the concrete was divided by an "effective" cross-sectional area of concrete to obtain a concrete stress versus strain curve for each specimen. For the regular portland cement concrete specimens the entire cross-sectional concrete area was taken as effective in helping the steel resist the axial loads. For the expansive concrete specimens the unrestrained cover concrete, which was always of very poor quality, was not included as concrete area effective in resisting axial loads. The calculations for concrete stress at a given strain are outlined in Appendix IV.

Column Tests on Tied Specimens

Three mixes for tied column tests were made. The curves for concrete stress versus strain are plotted in Figures 8-6 and 8-7. The tested specimens are shown in Figure 8-8. The two specimens of Mix 1 were made with regular portland cement concrete. The two specimens in Mix 2 were made with expansive cement concrete without air entrainment, and the two specimens in Mix 5 were made with expansive cement concrete with air entrainment. The specimens within mixes were consistent in behavior except those in Mix 1. The variation of the two specimens in Mix 1 was unaccountable except the sharp drop and recovery in the concrete stress versus strain curve for specimen 1I was due to an accidental unloading of the specimen during testing.

There was little difference in behavior indicated between the specimens of Mixes 1 and 2. The stiffnesses and strengths were similar and no clear advantage could be determined for using one mix over the other by observation of the concrete stress versus strain curves. It should be pointed out that Mix 2 was the expansive cement concrete mix where flash set occurred during casting.

The air entrained specimens of Mix 5 showed distinct differences in behavior from the other tied specimens. The stiffnesses were less and the strengths were lower.

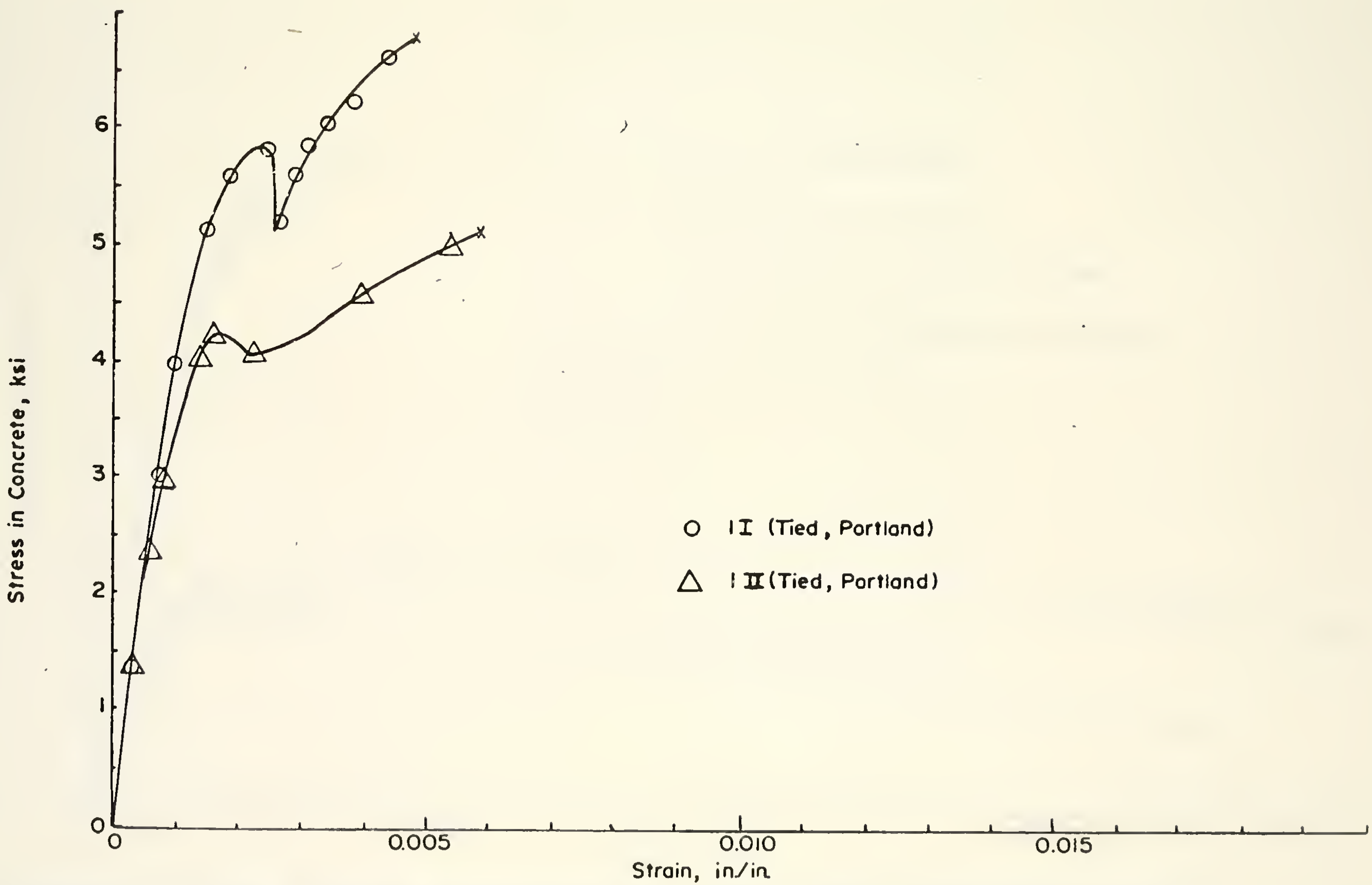


FIGURE 8-6 AXIAL STRESS IN CONCRETE VS. AXIAL STRAIN FOR MIX I.

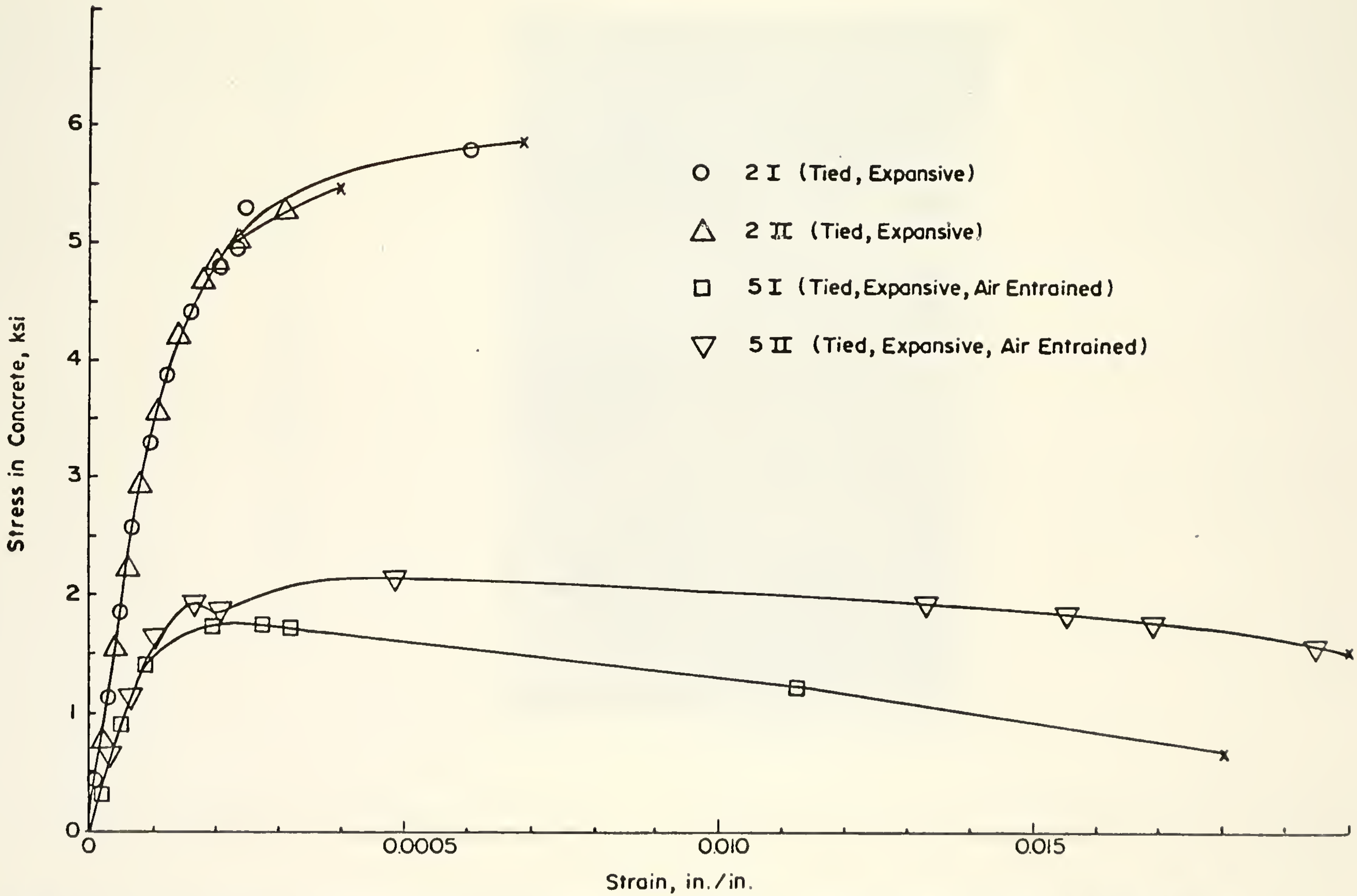


FIGURE 8-7 AXIAL STRESS IN CONCRETE VS. AXIAL STRAIN FOR MIXES 2 AND 5.

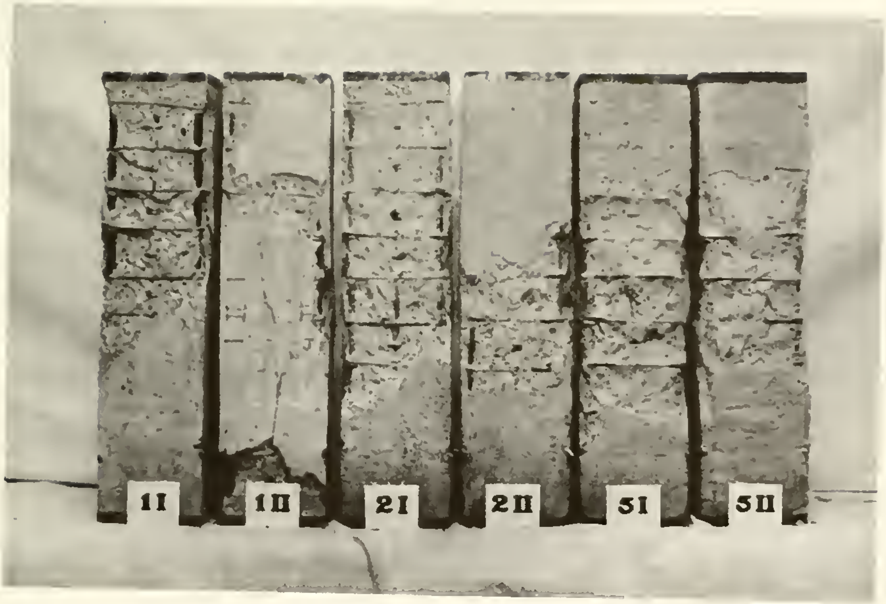


Figure 8-8. Tied Specimens After Column Tests.

The ductilities in the concrete of the specimens of Mix 5 were large as indicated by the definite and prolonged yield plateaus on the concrete stress versus strain curves in Figure 8-7.

Column Tests on Spiral Specimens

Seven specimens were made for the spiral column tests. The curves for concrete stress versus strain are plotted in Figures 8-9 and 8-10. The three specimens of Mix 3 were made with regular portland cement concrete. The two specimens of Mix 4 were made with expansive cement concrete without air entrainment, and the two specimens of Mix 6 were made with expansive cement concrete with air entrainment. The behavior of the specimens within each mix were very consistent for the spiral column tests.

Specimens 3I and 3II were made with a single spiral helix for the lateral reinforcement. The pitch was 1/2" which made the lateral reinforcement ratio, ρ', equal to 0.0055 by volume. Specimen 3III was made with a double spiral helix with a pitch of 5/8" which made ρ' equal to 0.00873. This extra lateral reinforcement slightly improved the ductility of the portland cement concrete. Although the specimens of Mix 3 carried more load than the specimens of Mix 4 made with expansive concrete the ultimate stresses in the concrete were similar. The expansive specimens of Mix 4 yielded sooner than the portland specimens of Mix 3, but the ductilities of the

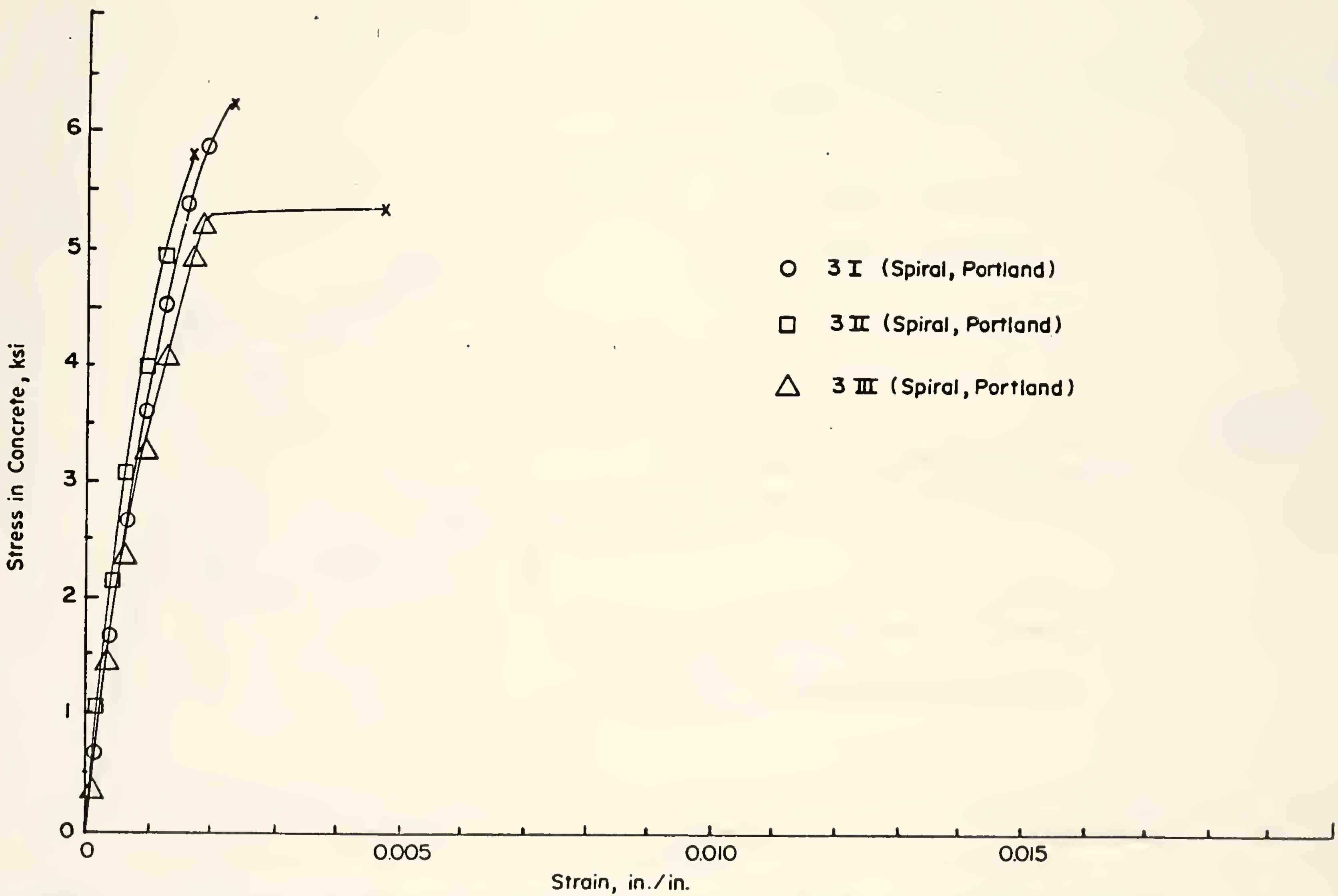


FIGURE 8-9 AXIAL STRESS IN CONCRETE VS. AXIAL STRAIN FOR MIX 3

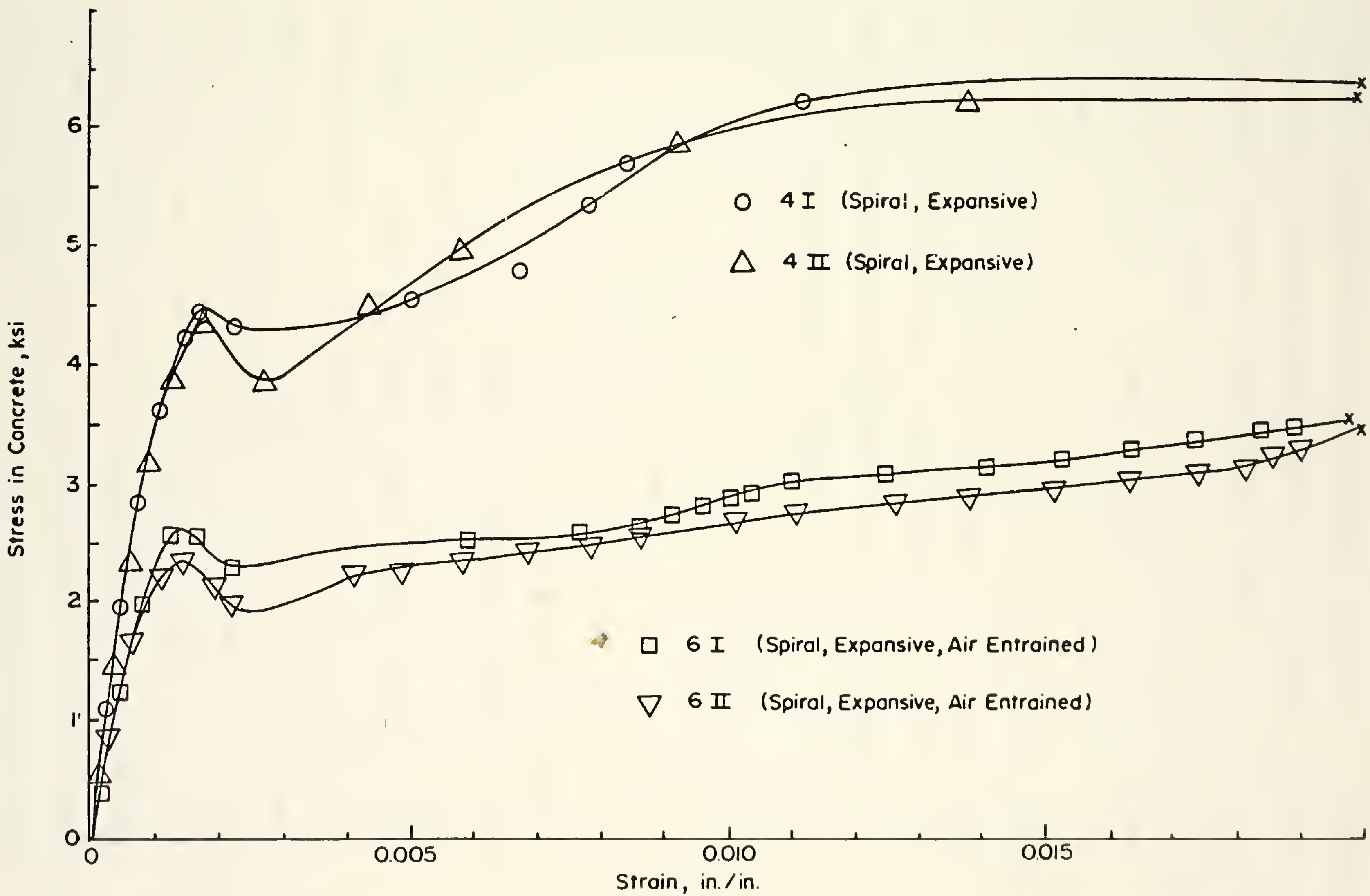


FIGURE 8-10 AXIAL STRESS IN CONCRETE VS. AXIAL STRAIN FOR MIXES 4 AND 6.

expansive specimens were superior to those of the portland specimens including 3III which had the same ρ' as the expansive specimens. The ductilities of the air entrained expansive specimens of Mix 6 were good, but the strengths were far inferior to the strengths of the other spiral specimens.

In Specimens 1II, 4I, 4II, 6I and 6II there appears a dip in the concrete stress versus strain curves soon after yielding occurs. An explanation of this can be seen by considering the load carrying allotments of the steel and concrete of a composite section. Figure 8-11 uses the load versus strain curve of Specimen 6I as an illustration. The load carried by the composite section is the load carried by the steel, S, plus the load carried by the concrete, C. To find the concrete load values for the concrete stress versus strain curves the load carried by the steel is subtracted from the total load at each strain value. The position of the yield "corner" in the steel load curve is determined by the steel properties and the prestrain in the steel due to expansion. Depending on the shape of the total load curve and the position of the yield "corner" in the steel load curve the value C_2 in Figure 8-11 could be less than C_1, hence creating a dip in the concrete stress versus strain curve. This is the obvious condition for Specimen 6I, and the other specimens that exhibit a dip in their

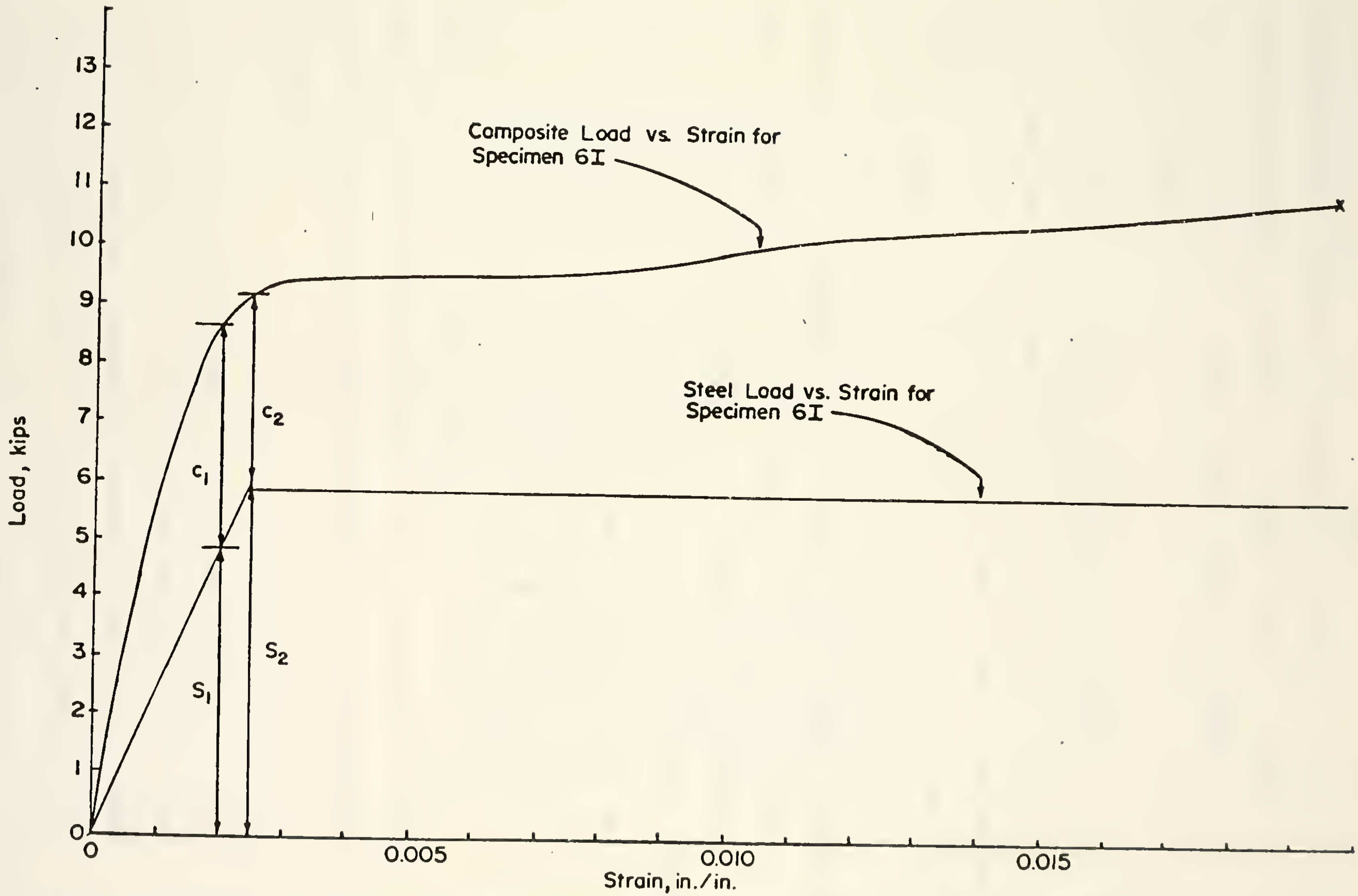


FIGURE 8-11 LOAD CARRYING DISTRIBUTION BETWEEN STEEL AND CONCRETE.

concrete stress curve follow similar patterns. The tested specimens are shown in Figure 8-12. The column test set up used for testing the column specimens is shown in Figure 8-13.

Beam Tests on Tied and Spiral Specimens

Beam tests were performed on 9 specimens from 4 mixes. The curvatures plotted are the midpoint curvature values of each specimen derived using the strain readings on the middle top and bottom reinforcing bars on each specimen. The average of the compression strain readings from the two gages on the middle top bar was added to the average of the tension strain readings from the two gages on the middle bottom bar. This sum was then divided by the distance between the top and bottom bars to get the curvature at the midsection of a specimen. It should be mentioned here that due to cracking and bond slip, the curvature at the middle portion of a beam may not be equal to the average curvature along the beam nor necessarily equal to the maximum curvature along the beam as in theory.

Beam Tests on Tied Specimens

The three 5 x 5 x 21 inch specimens of Mix 7 were made with portland cement concrete. Specimens 7I and 7II were tested using an 18 inch span and thrid points loading. The beams failed in shear within the end thirds.

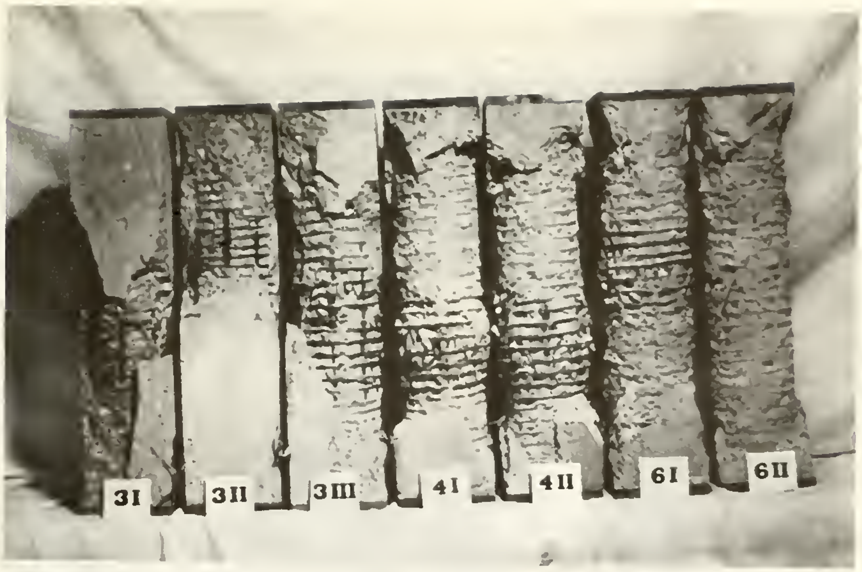


Figure 8-12. Spiral Specimens After Column Tests.

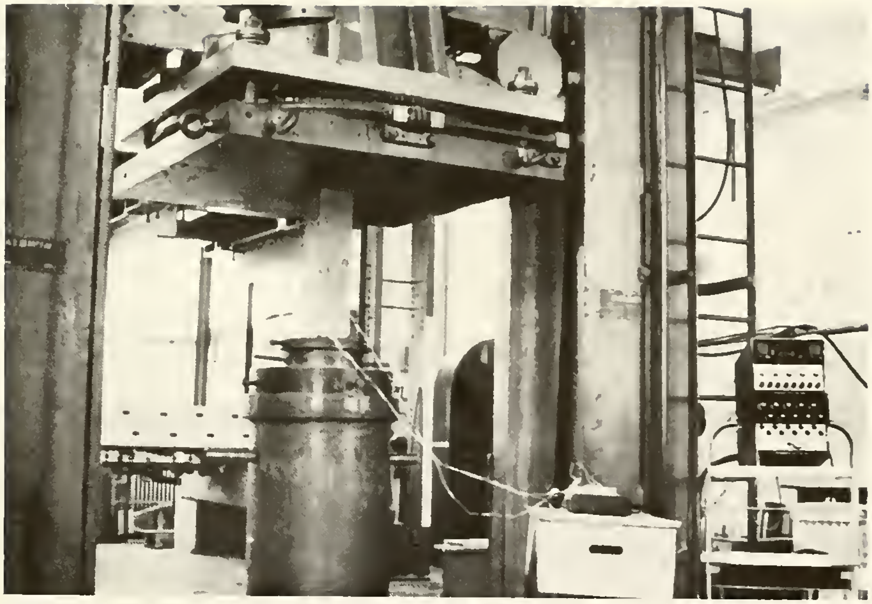


Figure 8-13. Column Test Setup.

Specimen 7III was tested using single point loading at the center of a 19 inch span. This allowed the specimen to reach its moment capacity before its shear capacity. This, however, created a combined shear and bending condition at the midpoint of the specimen instead of pure bending as in the case of third points loading. Still, since the moment and shear increased proportionally during loading, the tests proved to be valuable as a comparison between the behaviors of bending members made with portland and expansive concretes. Specimens 8I and 8II were made with expansive concrete and were tested using a single point loadings on 19 inch spans the same as Specimen 7III.

In looking at Figure 8-14 one can see that Specimen 7III, the portland concrete beam, was overreinforced. But the two specimens of Mix 8, which were made with expansive concrete and had the same steel arrangement as 7III, were underreinforced. This, at first, seems quite surprising since the preceeding columns tests showed little if any difference in the stress capacities of the portland concrete and the confined expansive concrete without air entrainment. The explanation for this lies in the prestrain in the tension steel of the expansive specimens. The determination of a beam being overreinforced or underreinforced is the crushing of the compression concrete before yielding of the tension steel.

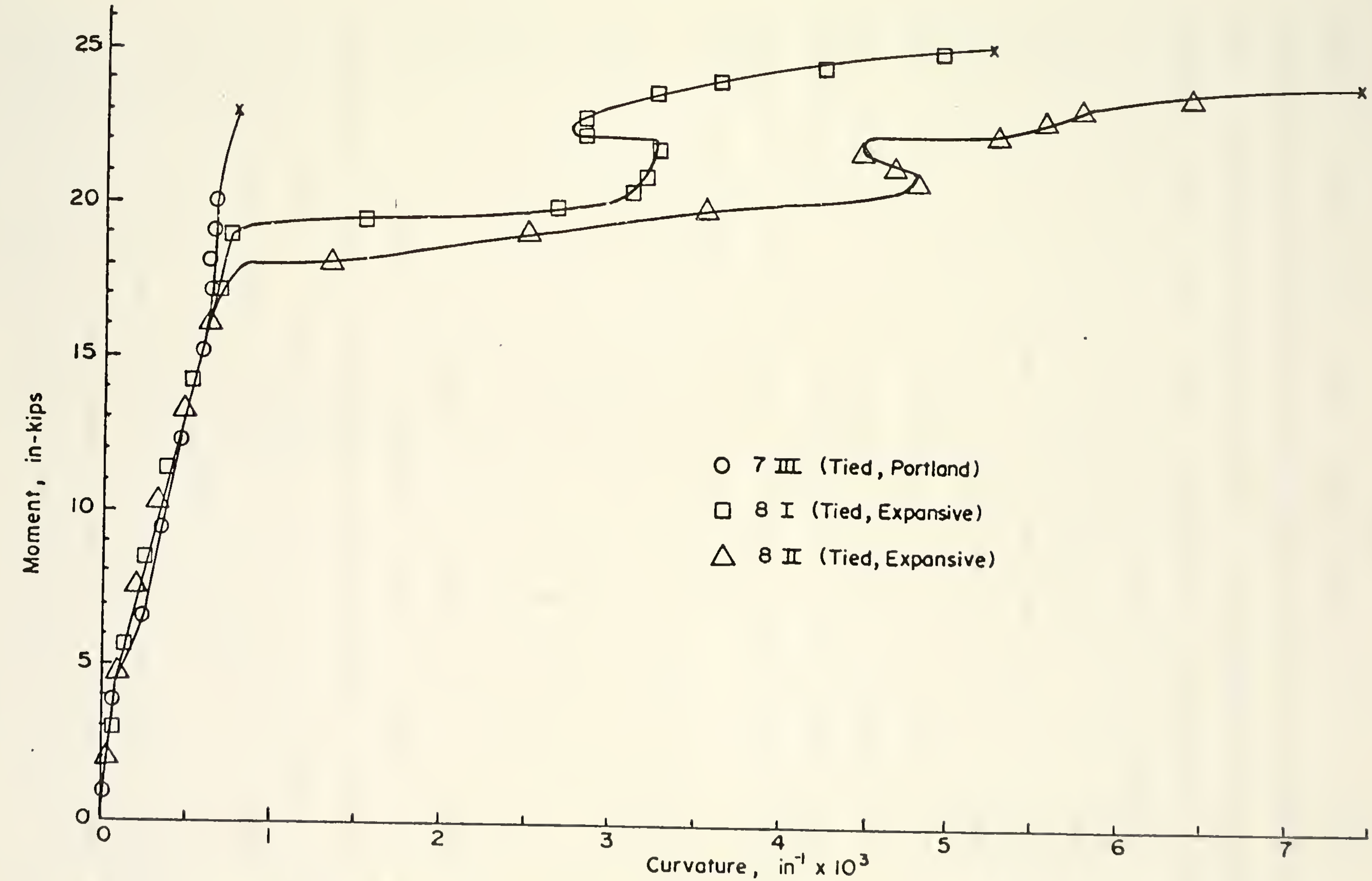


FIGURE 8-14 MOMENT VERSUS CURVATURE AT MIDSECTION OF TIED SPECIMENS.

Since all the steel including the tension steel in the expansive specimens were in tension before the beam tests were started, the tension steel in the expansive specimens reached yield sooner during loading than the tension steel in the non-expansive specimen which had no prestrain. Therefore, the amount of tensile prestrain in the tension steel determined whether or not the specimen was overreinforced for bending with the particular steel arrangement used. With little or no prestrain in the tension steel, as in the case of the portland specimen, the beam was overreinforced. But with prestrain in the steel due to expansion of the expansive concrete, beams with the same steel arrangement were underreinforced.

It is interesting to note that the tension steel in Specimen 8II had slightly higher measured prestrain due to concrete expansion than the tension steel in 8I. This would account for why 8II appears to be more underreinforced than 8I when observing the moment versus curvature plots on Figure 8-14. This implies that if a beam with a higher prestrain in its tension steel than 8II were tested, its plot would fall under that of 8II. And similarly, a beam with a lower prestrain in its tension steel than 8I would produce a plot over that of 8I.

The "S" shaped portion in each of the moment versus curvature plots of Specimens 8I and 8II is due to crack widening on each specimen at sections other than the midsection. By following the behavior of one of these beams along its moment versus curvature plot one can obtain a better understanding of what happened during testing. At the beginning of loading the beam acts as a composite, elastic member. Initially, when the tension steel yields the beam is still acting as a composite member. The curvature along the beam is still continuous. When the concrete on the tension side of the beam has reached its tensile strength cracks begin to open. This means that at the cracks the curvature continues to increase as the moment increases, but at sections where cracks are not opening the tensile strain is momentarily releaved due to a redistribution of strain to the cracks, and the curvature at the uncracked sections are reduced somewhat. Here the curvature along the beam becomes discontinuous with large variations due to very high curvatures at the cracked sections and lower curvatures at the uncracked sections. For Specimens 8I and 8II large cracks formed on either side of their midsections but not at their midsections where the curvatures were being monitored by the strain gages. Therefore, when the cracks opened the curvatures at the midsections of these specimens reduced causing the "S" shapes in the moment versus curvature plots, however, curvatures elsewhere along the beam were still increasing with the applied moments.

The most important observation from the beam tests of the rectangular tied specimens is that the expansive specimens were much more ductile and reached a higher ultimate moment. The tested specimens are shown in Figure 8-15.

Beam Tests on the Spiral Specimens

Beam tests were performed on four spiral specimens. Two were made with portland concrete and two were made with expansive concrete. With the realization of the inefficiency of a circular arrangement of steel for bending, these tests were run only as a further comparison of the two types of concrete.

All the spiral specimens exhibited underreinforced behavior. See Figure 8-16. The tension steel in the expansive specimens, 10I and 10II yielded before the tension steel in the portland specimens, 9I and 9II. This was expected since the tension steel in the expansive beams had an initial prestrain as explained previously.

Specimens 9I and 10II did not produce an "S" shape in the moment versus curvature plot. This indicates that in each of these specimens a crack formed at the midsection of the beam allowing the curvature at the midsection to continue increasing with increasing applied moments. In Specimens 9II and 10I the cracks widened away from the midsections causing the curvature at the midsection to decrease due to a redistribution of tinsile strain to the cracks.

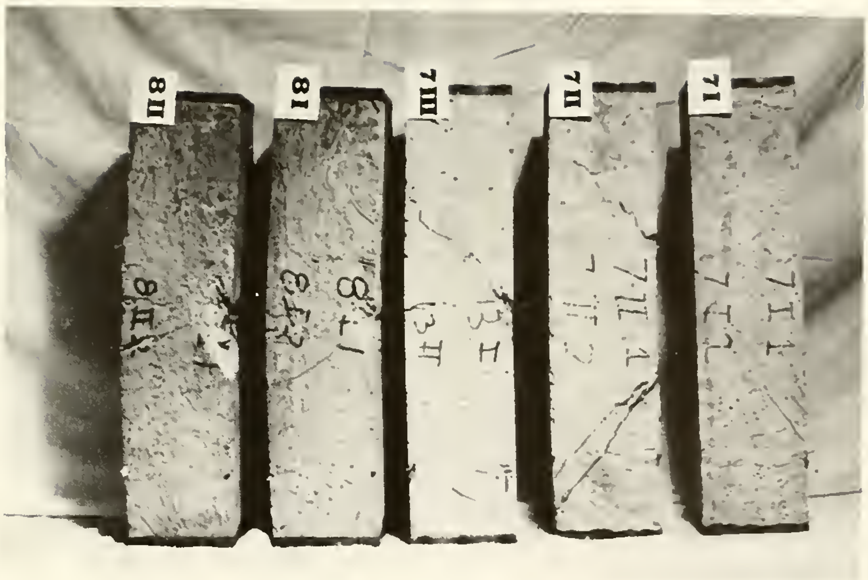


Figure 8-15. Tied Specimens After Beam Tests.

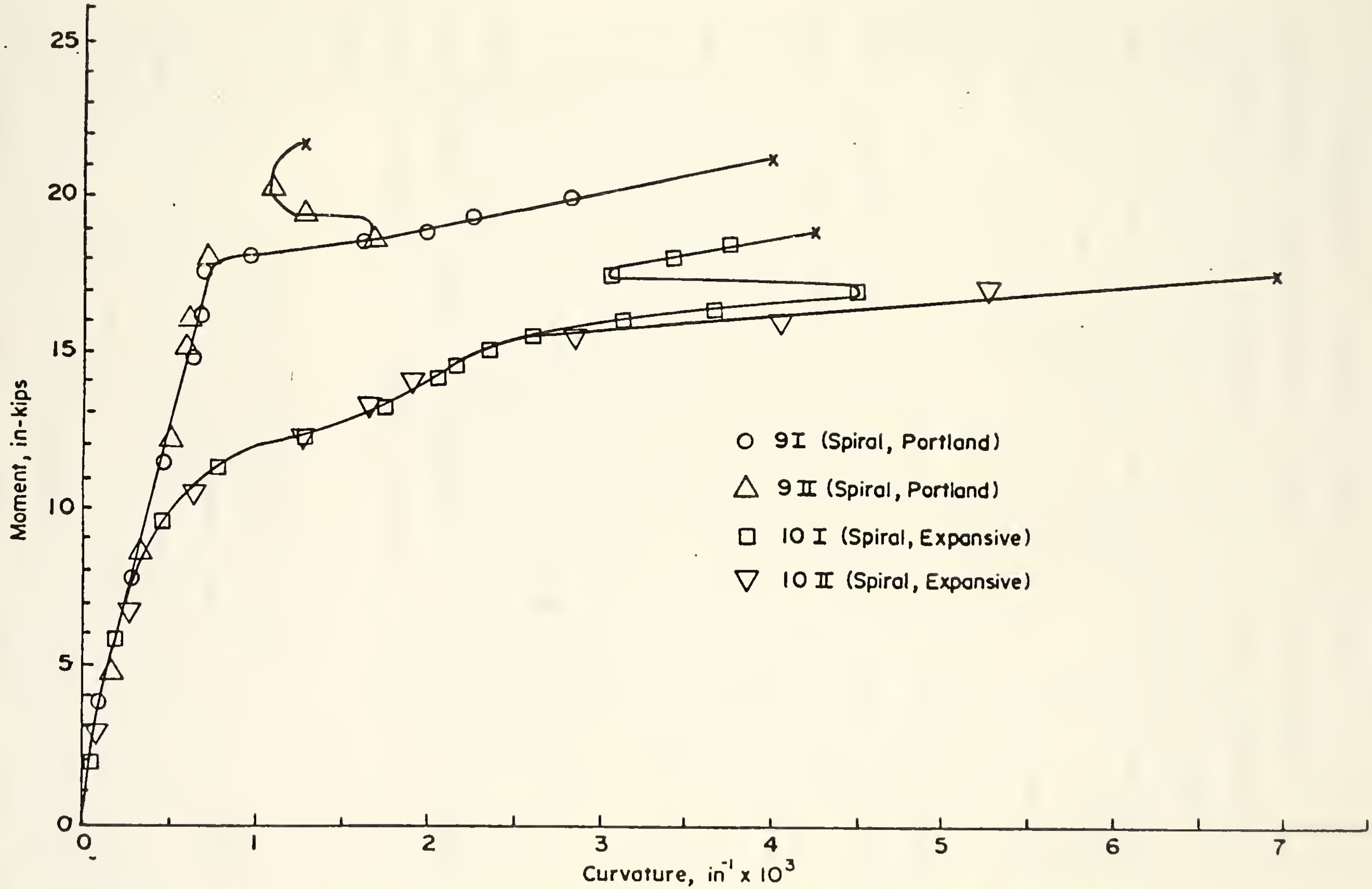


FIGURE 8-16 MOMENT VERSUS CURVATURE AT MIDSECTION OF SPIRAL SPECIMENS.

The expansive specimens were more ductile but the portland specimens showed higher ultimate moments capacities. The portland concrete spiral beams reached higher ultimate moments that the expansive concrete spiral specimens and the reverse was true for the tied specimens. The explanation for this is, there was more unrestrained expansive concrete in the compression zone of the expansive spiral bending specimens than the expansive tied bending specimens. The compression zone of the bending section of a rectangular spiral reinforced expansive beam being made up of mostly poor quality, unrestrained expansive concrete and this makes type of specimen very poor for carrying moments. Also, in these specimens the unrestrained expansive cover concrete at the point of load application showed considerable distress in bearing. This indicates that concentrated loads should be avoided with this type of member. The tested specimens are shown in Figure 8-17. The beam test setup used for testing the beam specimens is shown in Figure 8-18.

Freeze-Thaw Durability Tests

Six 3 x 4 x 16 inch specimens were cast for freeze-thaw durability tests. All the specimens were air entrained. Three were made with regular portland cement in the concrete and were used as controls for the other three which were made with expansive cement in the concrete.

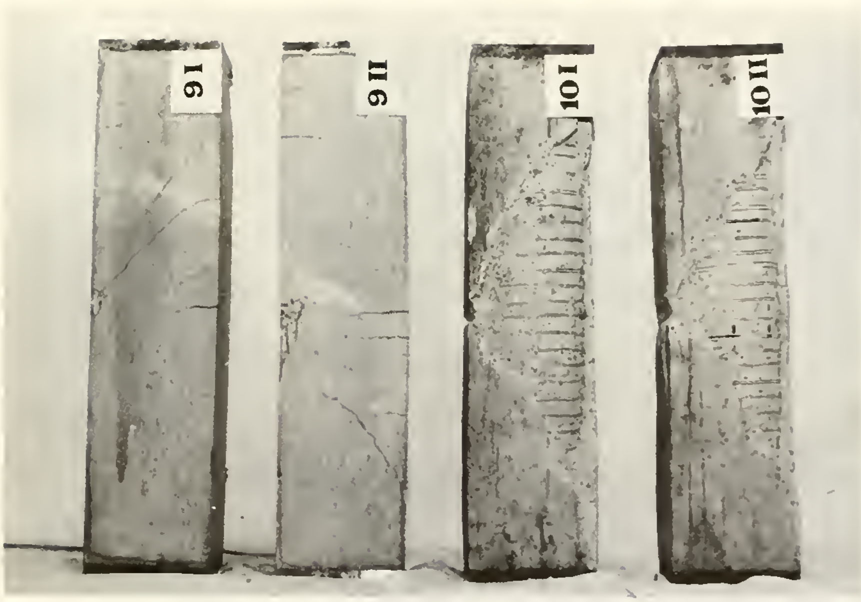


Figure 8-17. Spiral Specimens After Beam Tests.

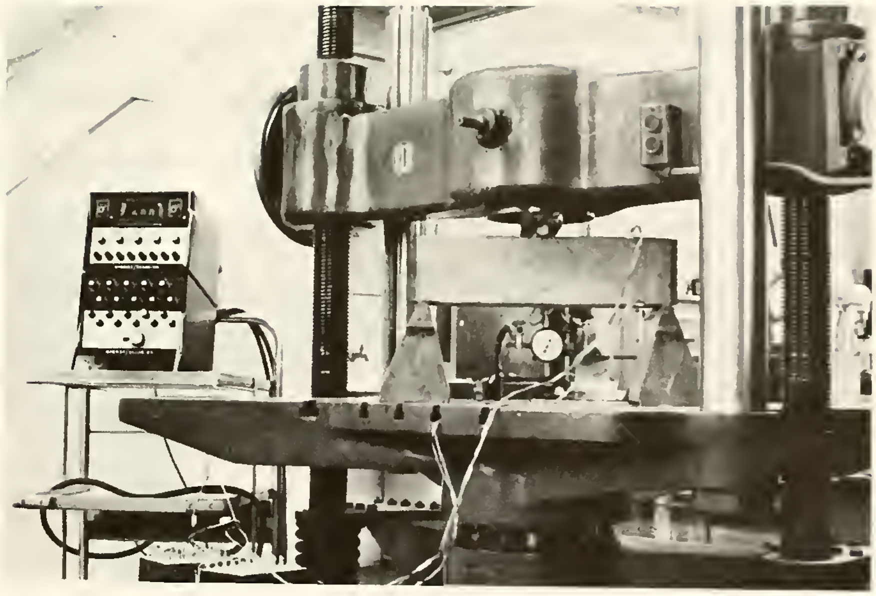


Figure 8-18. Beam Test Setup.

The water to cement ratios were 0.42 and the mix proportions were the same as used for the 5 x 5 x 21 inch specimens. All the specimens were cast on the same day, under the same conditions, and using the same type reinforcing cage in each. The percentage of steel used in the specimens was nearly the same as was used in the 5 x 5 x 21 inch tied specimens; therefore, the expansion and self-stress characteristics were assumed to be about the same.

After 15 days of curing in a fog room all the specimens were simultaneously put through 71 cycles of freezing and thawing over a 14 day period. Sonic modulus tests and weight loss tests were perfomred on the specimens at the start of the testing and at the completion of cycles 5, 11, 18, 30, 36, 58 and 71 using the following procedure:

1. Remove specimens from freeze-thaw chamber after approximately 40 minutes into a thaw cycle (temperature = 45°F).
2. Perform sonic modulus test and record the frequency.
3. Place specimens on a drip rack.
4. Towel dry the specimens to remove all the free water from the surfaces and then weigh each specimen and record the weight.

5. Using a wire brush, brush all surfaces excluding the plate surfaces. The brushing direction should be parallel to the longitudinal axis of each specimen. Brush the 3 x 16 inch sides 4 times (2 paths of 2 strokes each) and the 4 x 16 inch sides 6 times (3 paths of 2 strokes each).
6. Weigh the specimens again to determine the weight loss.
7. Return the specimens to the freeze-thaw chamber.

It should be noted the specimens were not standard freeze-thaw specimens in that they contained steel reinforcing for restraining the expansive concrete. The portland cement control specimens were also reinforced the same in order to obtain meaningful comparisons. Although the specimens were not standard and did contain steel, the frequency readings obtained showed definite differences between the two types of concrete in quality and durability. The values of the frequency readings and weight losses appear in Tables 8-1 and 8-2, Since the frequency values within each of the two sets of specimens were reasonably consistent, the average was taken in each set for each cycle. These values are shown in Figure 8-19. In general, as the sonic frequency measured through a specimen decreases the strength and modulus of elasticity decreases. Tests have shown this correlation to be good (9).

Table 8-1 Frequencies Attained in Sonic Modulus Tests.

	Frequencies (hertz)					
	Portland Specimens			Expansive Specimens		
Cycle	11A2I	11A2II	11A2III	11B2I	11B2II	11B2III
0	1885	1890	1890	1520	1485	1500
5	1865	1850	1860	1500	1470	1495
11	1855	1850	1855	1500	1470	1475
18	1855	1850	1860	1500	1465	1480
30	1850	1845	1855	1475	1460	1455
36	1845	1830	1855	1455	1120	1455
58	1860	1850	1850	1075	1110	1100
71	1955	1850	1840	--	--	--

Table 8-2 Accumulative Weight Loss of Freeze-Thaw Specimens.

	Weight Loss (grams)					
	Portland Specimens			Expansive Specimens		
Cycle	11A2I	11A2II	11A2III	11B2I	11B2II	11B2III
5	0	13	0	10	10	1
11	0	16	0	24	14	5
18	0	18	0	63	24	14
24	0	21	0	88	39	25
30	1	23	0	113	56	76
36	1	29	0	169	72	100
58	3	32	6	268	121	153
71	3	35	7	353	180	219

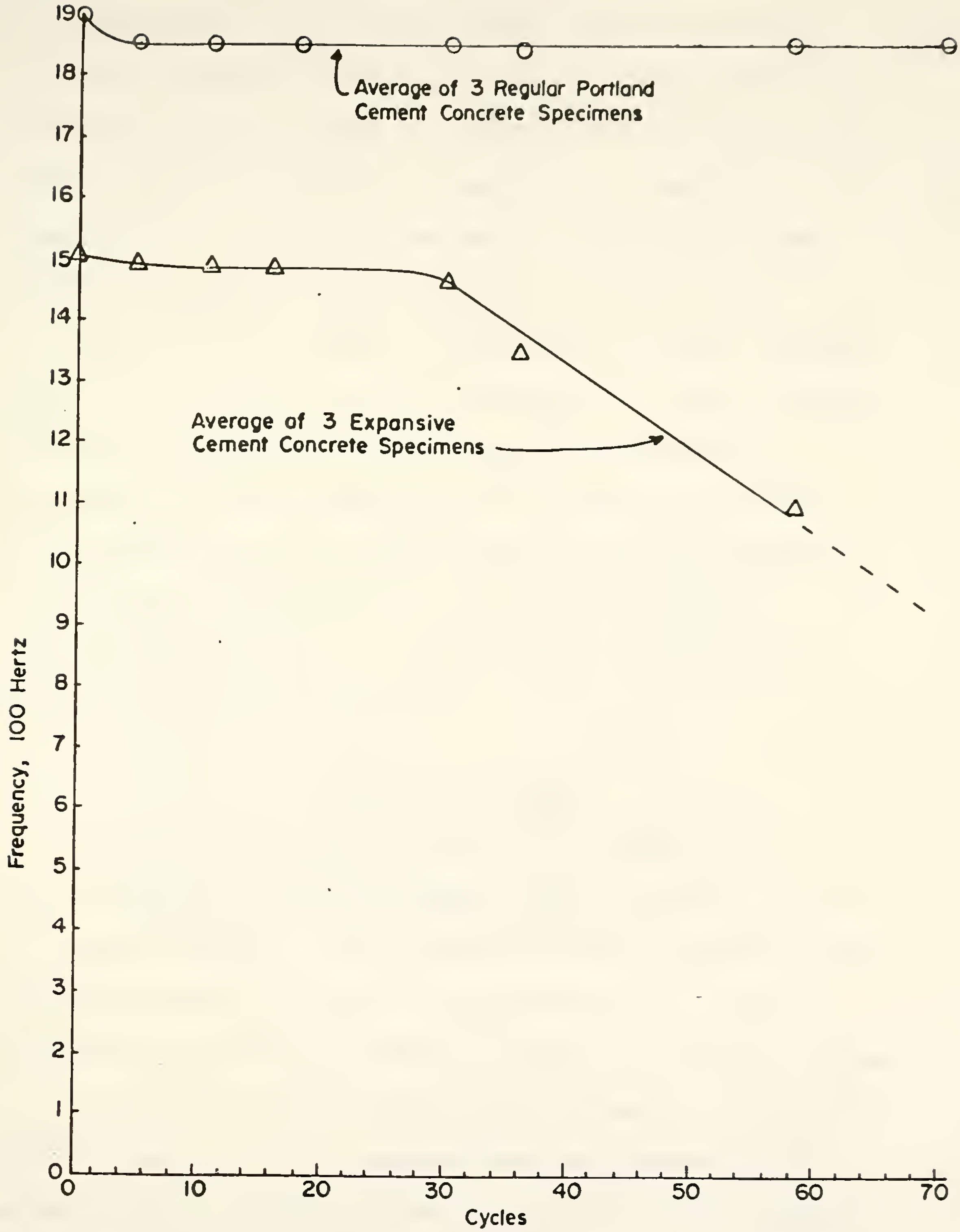


FIGURE 8-19 SONIC FREQUENCY VS. NUMBER OF CYCLES FOR FREEZE-THAW SPECIMENS.

The original qualities of the expansive concrete specimens were much less than those of the portland specimens as can be seen in Table 8-2 and Figure 8-19. The durability of the expansive specimens was very poor compared to the portland specimens as depicted by the drop in frequency readings relative to the initial.

After 58 cycles some reinforcing steel was exposed on all the expansive concrete specimens. Also, after 58 cycles the frequency readings on all the expansive specimens were less than 60% of the initial readings. Spalling off of the exterior concrete on the expansive concrete was considerable all through the testing. Figures 8-20 and 21 show all the freeze-thaw specimens before and after 71 cycles.

Time of Set Tests

The purpose of these tests was to compare the setting rate of expansive cement mortar to that of portland cement mortar. The variables in the tests were mix water temperature and water to cement ratio. The mix proportions and order of mixing appear in Tables 8-3 and 8-4.

The gypsum used in the expansive cements of the first twelve mixes was a fine ground food and pharmaceutical grade hydrous clacium sulfate. The reagent grade gypsum that was used in the final two mixes was not used anywhere else in this author's investigation. The purpose for inclusion here was to compare the effectiveness of the

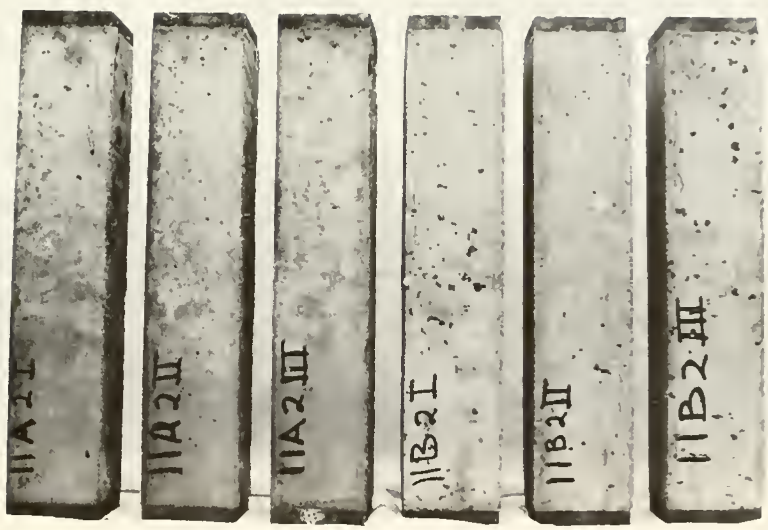


Figure 8-20. Freeze-Thaw Specimens

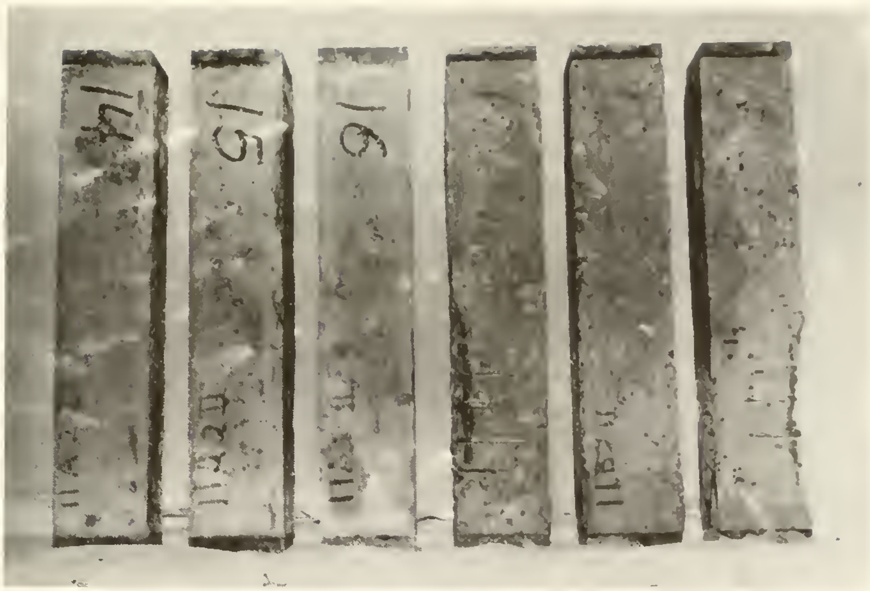


Figure 8-21. Freeze-Thaw Specimens After 71 Cycles.

Table 8-3 Mixes for Time of Set Tests.

Mix	Mix Water Temperature	Type of Cement	Mix Design
1	10°C	Portland	
2	10°C	Medium Expansive	Water 2540 ml
3	10°C	High Expansive	Cement 5770 g
4	20°C	Portland	Sand 9430 g
5	20°C	Medium Expansive	Water to Cement - 0.40
6	20°C	High Expansive	(Pharmaceutical grade gypsum in expansive cements)
7	10°C	Portland	
8	10°C	Medium Expansive	Water 2655 ml
9	10°C	High Expansive	Cement 5770 g
10	20°C	Portland	Sand 9430 g
11	20°C	Medium Expansive	Water to Cement - 0.42
12	20°C	High Expansive	(Pharmaceutical grade gypsum in expansive cements)
13	20°C	Medium Expansive	Water 2540 ml
14	20°C	High Expansive	Cement 5770 g
			Sand 9430 g
			(Reagent grade gypsum in expansive cements)

Table 8-4 Medium and High Expansive Cement Compositions.

	Portland Cement Type I	Calcium Aluminate Cement	Gypsum
Medium Expansive	65%	20%	15%
High Expansive	67.5%	17.5%	15%

pharmaceutical grade gypsum to the higher grade reagent gypsum which had been used in previous investigations of Type M expansive cements by Gowda (4).

Procedure for Time of Set Tests

The procedure followed for the time of set tests was basically according to ASTM C402-70 Standard Method of Test for Time of Setting of Concrete Mixture by Penetration Resistance. Two exceptions were made from the standard. First, the mortar used was obtained simply by mixing the concrete materials without the coarse aggregate. Second, the number of batches per mix condition was one rather than three. Two specimens per batch were tested.

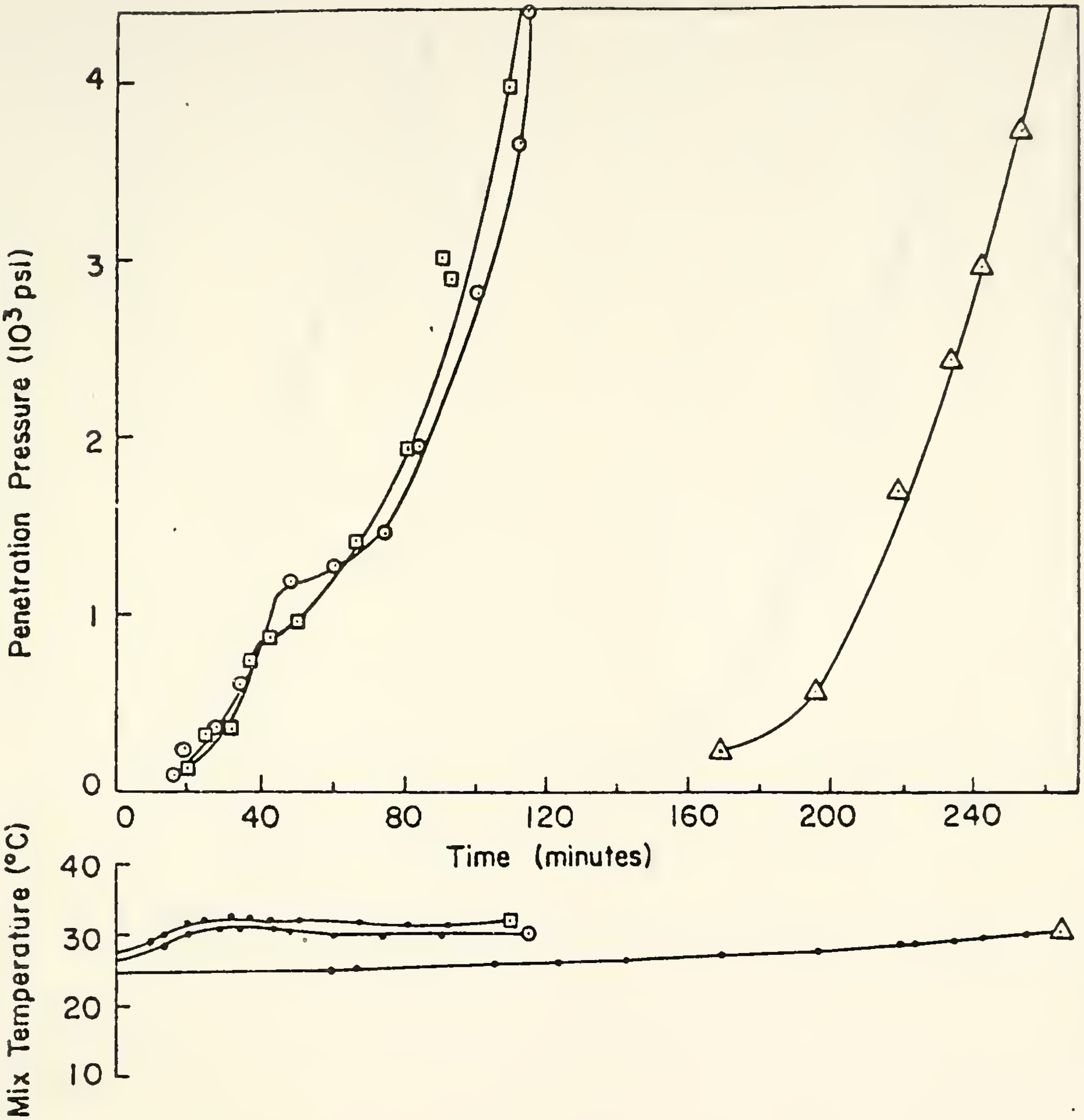
Results of Time of Set Tests

The results of the time of set tests indicated several general trends for the set of expansive mortars. The most obvious and important trend was consistent rapid set of the expansive cement mortars as compared to the portland cement mortars.

The expansive cement mortars set up with final sets ranging from 86 minutes to 186 minutes depending on the conditions of the mixes as shown in Table 8-5. The portland cement mortars on the other hand, took nearly 180 minutes before they even began to set. Final sets occurred between 236 minutes and 270 minutes. Figures 8-22 through 26

Table 8-5 Times of Set (Minutes).

	Water to cement ratio	0.40	0.40	0.42	0.42
	Water temperature	10°C	20°C	10°C	20°C
Mortar Type	Grade of Gypsum				
Portland	--	259	236	270	257
Meduim Expansive	Pharmaceutical	116	86	154	104
Medium Expansive	Reagent	--	67	--	--
High Expansive	Pharmaceutical	111	116	186	140
High Expansive	Reagent	--	129	--	--



Water-Cement Ratio = 0.40

Water Temperature = 10°C

Medium Expansive ○

High Expansive ⊡

Portland △

Figure 8-22 Time of Set for Medium Expansive, High Expansive, and Portland Cement Mortars; W/C = 0.40 Water Temp. = 10°C

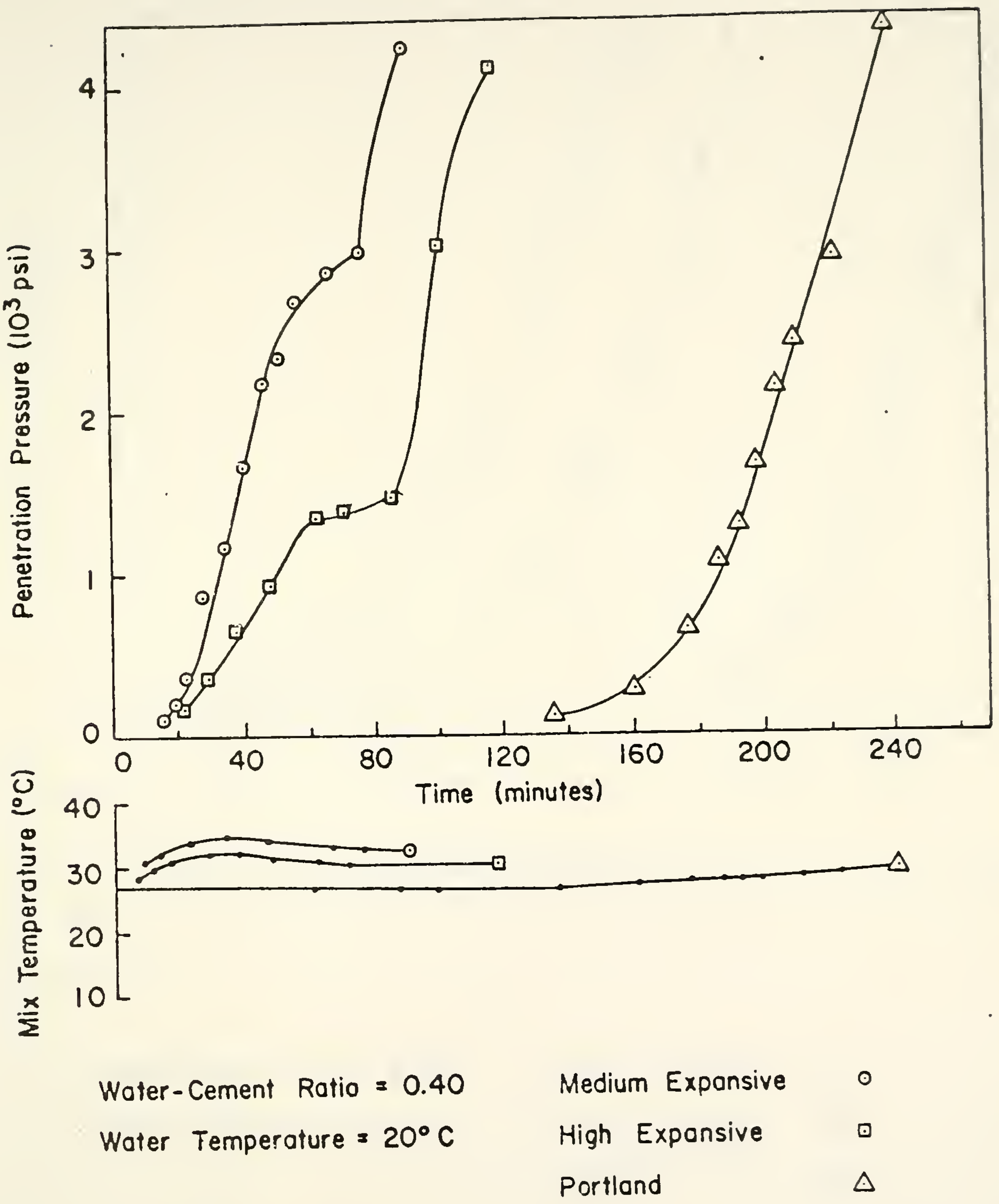
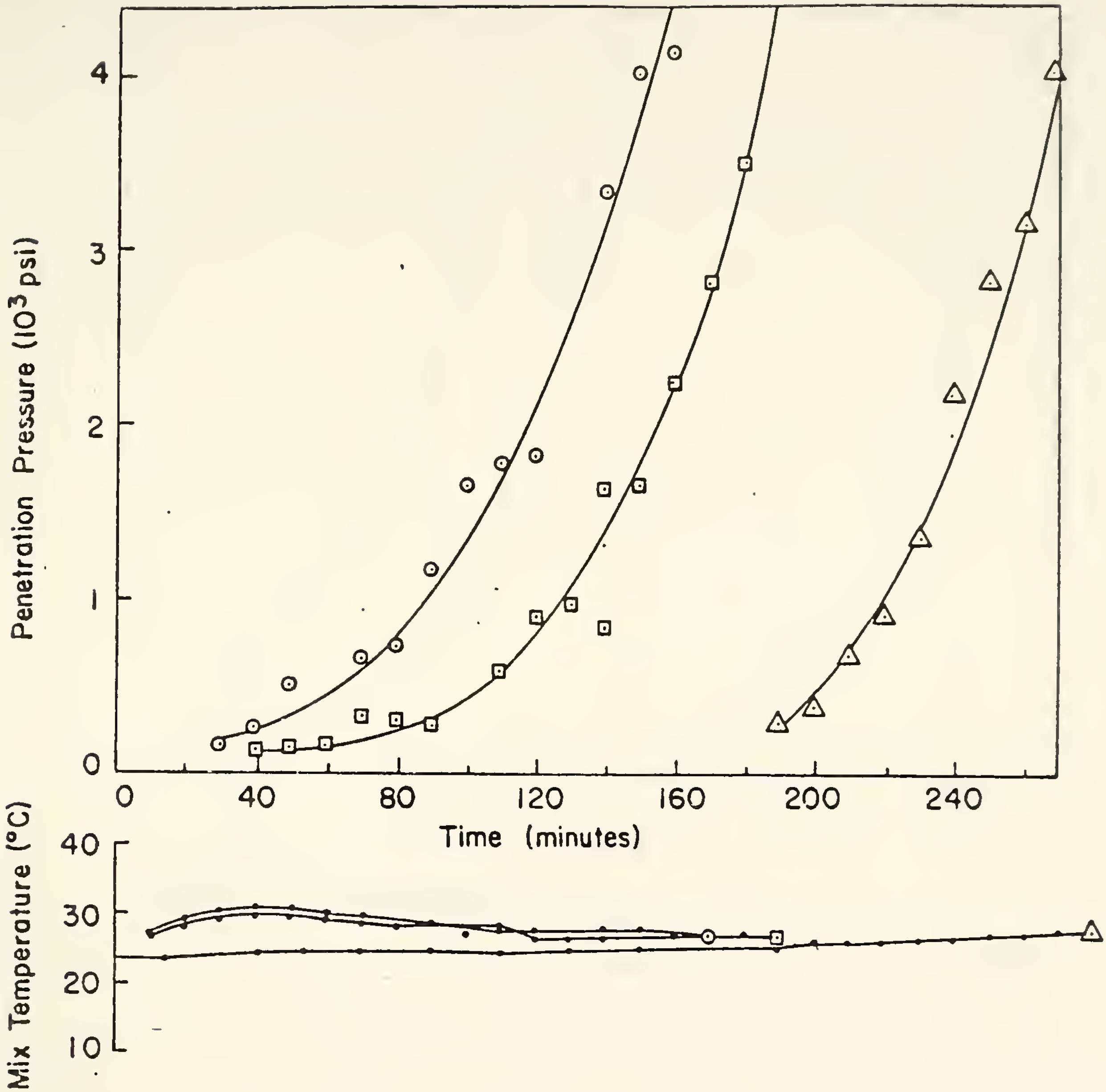


Figure 8-23 Time of Set for Medium Expansive, High Expansive, and Portland Cement Mortars; W/C = 0.40, Water Temp. = 20°C



Water-Cement Ratio = 0.42

Water Temperature = 10°C

Medium Expansive ⊙

High Expansive ⊡

Portland △

Figure 8-24 Time of Set for Medium Expansive, High Expansive, and Portland Cement Mortars; W/C = 0.42 Water Temp. = 10°C

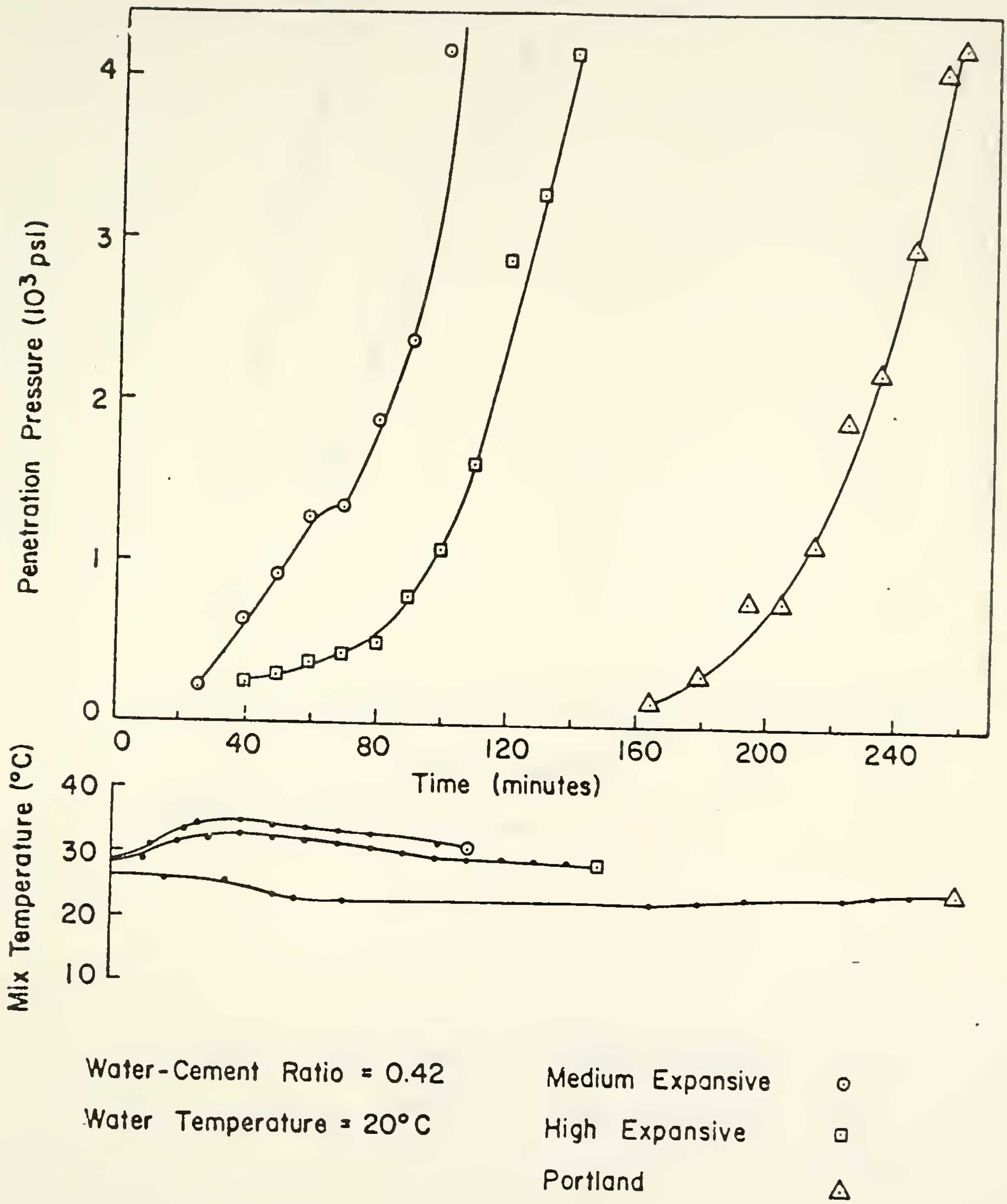
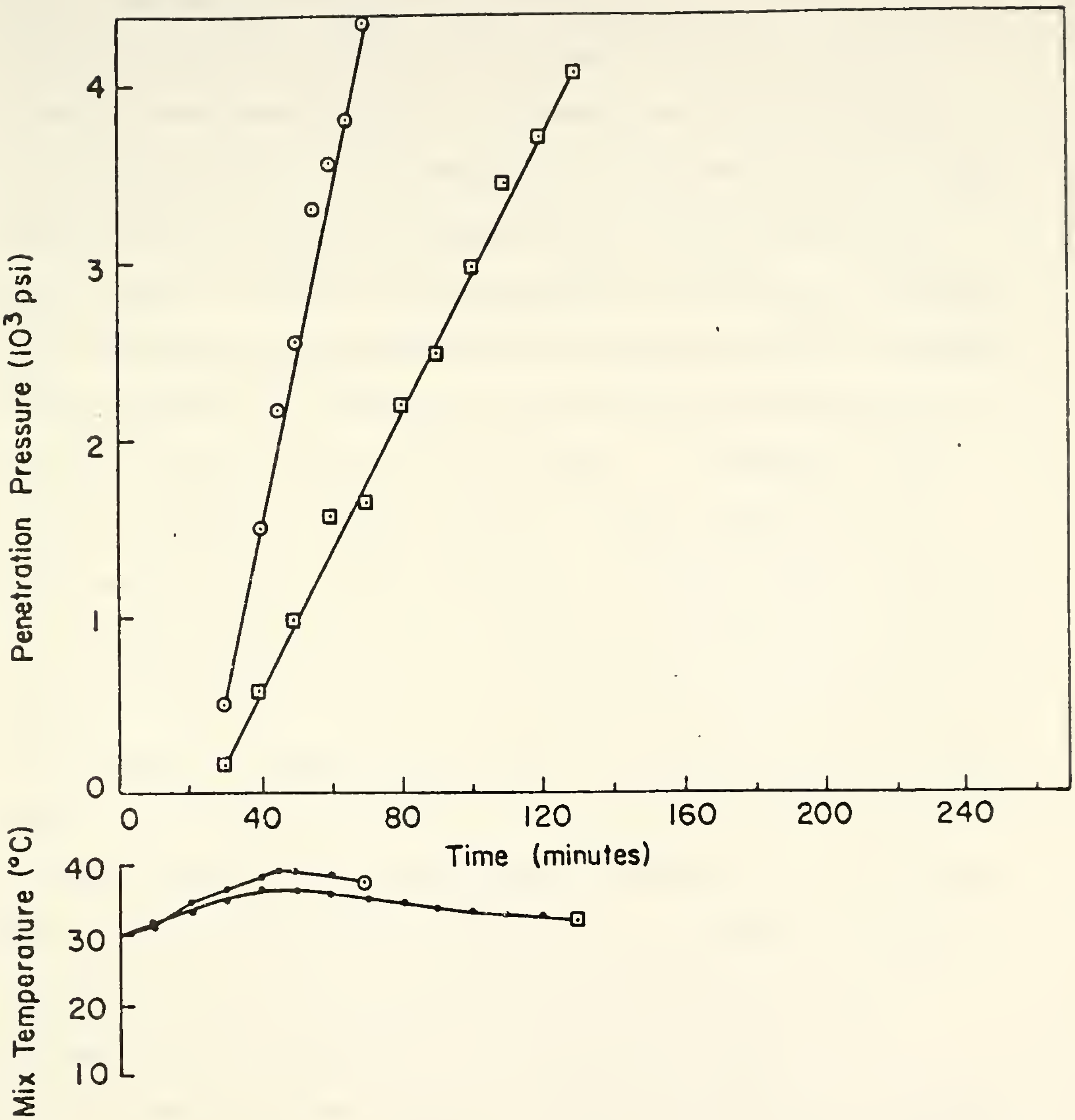


Figure 8-25 Time of Set for Medium Expansive, High Expansive, and Portland Cement Mortars; W/C = 0.42, Water Temp. = 20°C



Water-Cement Ratio = 0.40

Water Temperature = 20°C

Medium Expansive ⊙

High Expansive ⊡

Figure 8-26 Time of Set for Medium Expansive and High Expansive Mortars Mixed with Reagent Grade Gypsum; W/C = 0.40, Water Temp. = 20°C show the mortar temepratures from mixing to final set. The portland cement mortars showed a general cooling toward room temperature. Then when the mortars began to harden about a 2°C temperature rise would occur. The portland cement mortars appeared to be quite passive until the last active 100 minutes of the setting process. Both the high and medium expansive cement mortars showed large amounts of chemical activity from the moment the mortars were fresh out of the mixer. In general, the medium expansive cement mortars heated up more rapidly and set up faster than the high expansive cement mortars. The only exception to this occurred with water to cement ratios of 0.40 and mix water temperatures of 10°C. In this case the temperature readings of the high expansive mortar were higher and the final set occurred a little sooner than for the medium expansive mortar. The higher percentage of calcium aluminate in the medium expansive cement may have been the factor contributing to its faster setting times in all the other cases.

It was observed that two conditions could be imposed on any of the mortars to delay setting time. One was an increase in the water to cement ratio and the other was a decrease in the mix temperature. In this investigation the latter was accomplished by controlling the mix water temperature.

Comparison of the effectivenesses of the two grades of gypsum showed no conclusive results. The medium expansive mortar with the reagent grade gypsum yielded a set time 19 minutes quicker than the mortar with the pharmaceutical grade gypsum. On the other hand the high expansive mortar yielded a set time 13 minutes slower. The tests showed no pattern in the use of the two different grades of gypsum and no advantage was found in using one over the other except the lower cost of pharmaceutical grade.

Conclusions for Time of Set Tests

Strict care should be exercized when using expansive cement concretes. The material sets up much more rapidly than portland cement concretes. Measures should be taken to keep the fresh concrete cool and to get it placed and worked as quickly as possible. Further delay of set could be achieved by increasing the water to cement ratio, but the resulting concrete quality would be reduced. An investigation of chemical set retarders would be desirable before field use of an expansive cement concrete mix.

CHAPTER 9

SUMMARY OF RESULTS

The following observations were made in this investigation:

1. The workabilities of the expansive concretes were poorer and the consistencies were more harsh than for the portland cement concretes. Increasing the water to cement ratio and decreasing the mix water temperature were two ways of successfully improving the consistency of the expansive concretes.
2. Highest expansion strains and self-stresses were obtained using the spiral reinforcement arrangement and expansive concrete without air entrainment. Air entrainment considerably reduced the expansion capacity of the expansive concrete.
3. Creep in the expansive concrete specimens occurred causing a reduction of the self-stress obtained in expansion.
4. The unrestrained cover concrete of the expansive specimens developed many cracks, was of very poor quality, and provided little protection against corrosion of the reinforcement.

5. The strength of the triaxially restrained expansive concrete was comparable to that of the portland cement concrete of the same cement content and roughly the same water to cement ratio. Due to the triaxial prestress caused by expansion the ductility of the expansive concrete specimens was much greater than that of the portland concrete specimens.
6. The strength of the expansive concrete was reduced 50 to 70 percent when entrained with air.
7. With a rectangular arrangement of steel the ultimate moment was higher and the ductility better for the beams specimens when expansive concrete was used instead of portland cement concrete.
8. Air entrainment was able to be obtained in the same manner for both the expansive and the portland concretes.
9. The freeze-thaw durability of expansive concrete, even with air entrainment, was extremely poor.
10. The time of set of the expansive concrete was extremely fast. Reducing the mix temperature and increasing the water to cement ratio retarded the set of the expansive concrete.

CHAPTER 10

CONCLUSIONS

The conclusions drawn here are pertinent only to the conditions of the tests and materials used in this investigation.

1. Expansive concrete should not be subject to exterior exposure. Corrosion of the reinforcing steel and deterioration due to freezing and thawing are critical when using expansive concretes.
2. The use of air entrainment with expansive cement concretes is not recommended because no significant improvement in durability results and significant reduction in strength occurs.
3. Circular spiral reinforcing is superior over tied reinforcing for providing lateral restraint of expansion.
4. Consideration of time of set is very important with expansive concrete and precautions should be taken to prevent flash set.
5. Creep in expansive concrete can be large and should not be disregarded.

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APPENDIX I

PROPERTIES OF REINFORCING STEEL

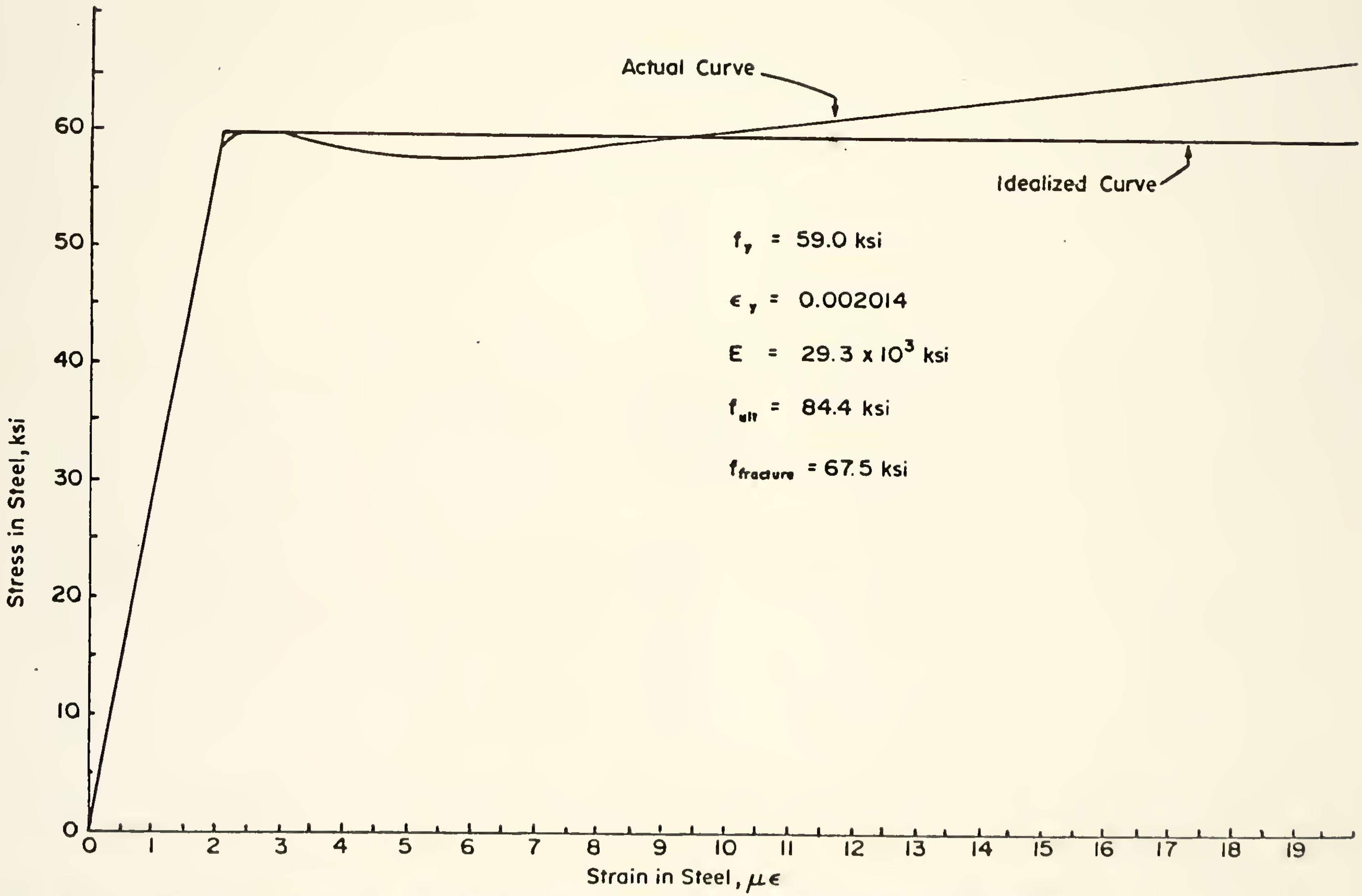

FIGURE AI. PROPERTIES OF # 3 REINFORCING BAR FROM TENSION TEST.

APPENDIX II

STRENGTH OF PORTLAND CEMENT CONCRETE

Four 3 x 6 inch cylinders were tested in compression and four were tested in split tension for each of the five mixes. The means and standard deviations of these values appear in Table A1.

Table A1. Strength of Portland Cement Concrete.

Mix Code	Compressive Strength	Standard Deviation of Compressive Strength	Split Tensile Strength	Standard Deviation of Tensile Strength
1	7772 psi	230 psi	787 psi	26 psi
3	7940 psi	117 psi	810 psi	35 psi
7	6985 psi	226 psi	794 psi	28 psi
9	7463 psi	229 psi	776 psi	106 psi
12	7675 psi	49 psi	811 psi	68 psi

APPENDIX III

OUTLINE OF CALCULATIONS FOR SELF-STRESS

Calculations for self-stress in the concrete of an expansive specimen were performed using the following equation:

$$\sigma_p = \frac{\varepsilon_e \ E_s \ A_s}{A_c - A_s} \ , \ \varepsilon_e \leq \varepsilon_y \text{ in all cases}$$

σ_p = Calculated compressive self-stress in concrete.

ε_e = Measured expansion strain in the specimen (tension positive).

E_s = Modulus of elasticity of the longitudinal steel.

A_s = Cross-sectional area of longitudinal steel at mid-section of the specimen. (The reduction of steel area due to lathing of the deformations for gage mounting was taken into account.)

A_c = Cross-sectional area of concrete within the outside dimensions of the lateral reinforcement.

ε_y = Yield strain for the longitudinal steel.

APPENDIX IV

OUTLINE OF CALCULATIONS FOR CONCRETE STRESS

Calculations for concrete stress* due to applied loads on a specimen were performed using the following equations:

$$\sigma = \frac{P_a - E_s \varepsilon_s A_s}{A_g - A_s} \text{, for regular concrete and } \varepsilon_s \leq E_y$$

$$\sigma = \frac{P_a - E_s \varepsilon_y A_s}{A_g - A_s} \text{, for regular concrete and } \varepsilon_s > \varepsilon_y$$

$$\sigma = \frac{P_a - E_s \varepsilon_s A_s}{A_c - A_s} \text{, for expansive concrete and } \varepsilon_s \leq \varepsilon_y - \varepsilon_e$$

$$\sigma = \frac{P_a - E_s (\varepsilon_y - \varepsilon_e) A_s}{A_c - A_s} \text{, for expansive concrete and } \varepsilon_s > \varepsilon_y - \varepsilon_e$$

σ = Stress in concrete due to loading.

P_a = Applied load.

E_s = Modulus of elasticity of longitudinal steel.

ε_s = Measured in steel during loading. (For the expansive specimens $\varepsilon_s = 0$ at the prestrained condition immediately before loading and compression is positive.)

A_c = Cross-sectional area of concrete within the outside dimension of the lateral reinforcement.

A_g = Gross area of concrete.

A_s = Cross-sectional area of longitudinal steel at mid-section of the specimen. (The reduction of steel area due to lathing of the deformations for gage mounting was taken into account.)

ε_y = Yield strain for the longitudinal steel (compression positive).

ε_e = Measured expansion strain of specimen immediately before loading (compression positive).

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* Zero for concrete stress in the expansive specimens was taken as the self-stressed conditions. Therefore, the stress carrying capacity of the self-stressed concrete was compared to that of the regular portland concrete. For all the concrete stress calculation perfect bond between the steel and concrete was assumed.

PART III:

STRUCTURAL BEHAVIOR OF STEEL TUBE COLUMNS FILLED WITH EXPANSIVE CONCRETE

CHAPTER 11

INTRODUCTION

Expansive concretes with the potential of self-stressing may become a competitive material in structural applications. In this investigation a type M expansive cement concrete shall be observed for its structural performance as a column while confined inside structural steel tubing. As a composite expansive concrete and steel column significant mechanical improvements may be provided mutually for each material. From the investigation by H. Gowda (1) it is apparent that expansive concrete with self-stressing potential is worthless without any confinement. In this report the expansive concrete is observed as a structural material totally confined. The effects of curing temperature on the expansions and self-stress and finally on the structural performance of the tubular columns also follow. Comparison is made against columns filled with type I portland cement concrete, normal concrete.

Expansive Concrete Properties

There are several unknowns about expansive concretes. Because of the mechanical tendency to crumble during the advanced stages of expansion, the unconfined compressive strength can not be measured directly. Similarly, the modulus of elasticity can not be obtained. The absence of these two parameters so common in structural analysis makes it very difficult to predict the material behavior and structural behavior. Analysis and interpretation of the structural behavior must be based on empirical relationships of the composite element. Expansive concrete will never be found performing apart from some kind of confinement.

Methods of Confinement

Unconfined expansive concrete will disintegrate. Therefore, confinement is essential in order to maintain the material's mechanical continuity and also to achieve the self-stressing effect. Confinement is possible in three different modes: axial, biaxial and triaxial.

Triaxial confinement would be the mode in which the working section of the concrete is restrained by reinforcement in all directions. Conventional patterns of reinforcement for reinforced concrete beams and columns may satisfy this type of confinement. For example, a concrete beam with longitudinal steel and closed stirrups regularly spaced provides triaxial confinement to the beam's concrete core. Similarly, reinforced concrete columns with ties or spiral reinforcement confine triaxially the concrete core. The effect can be good. However, the total concrete section is not confined. A concrete shell covers the reinforcement. Under high loads with large deflections this shell spalls off providing no structural strength.

Triaxial confinement of the total concrete section may be achieved by placing the expansive concrete within structural steel tubing. Thus, the entire composite section remains continuous in service up through its ultimate loading.

Confinement by Steel Tubing

Confinement of expansive concrete by means of circular structural steel tubing has advantages as well as disadvantages. Advantages are:

1) The expansive concrete is totally confined. Thus no material spalls off at high loads.
2) Fabrication is simplified. Only the end plates or connections need to be handworked.
3) No formwork is needed.
4) Moist curing is of no concern. There is no exposure to allow drying.

Disadvantages are:

1) Steel tubing is very expensive.
2) Steel pipe may be substituted at a lower expense admitting more nonuniformity in the steel wall thickness.
3) The exposed steel needs protection from weather and fire.

Objectives and Scope of the Investigation

The objectives of this investigation were to compare the structural performance of composite tubular columns filled with normal portland cement concrete against columns filled with expansive concrete. Related to this for the columns with expansive concrete, the influence of the curing temperature on the expansion history and corresponding self-stress needed to be observed.

The scope of the column investigation was limited to the use of one type and size of steel tubing, one mix design for the portland cement concrete, two mix designs, medium and high expansions, for the expansive concrete, and three different curing temperature conditions. The plan for this column investigation is illustrated in Table 11-1.

Table 11-1 Plan of Column Tests.

Column Numbers	Type of Concrete	Curing Temperature	Age at Column Test
1 - 2	HEX	84°F	14 days
3 - 6	HEX	70°F	> 14 days
7 - 13	HEX	70°F	Expansion Completed
14	HEX	50° - 70° - 90°F	Expansion Completed
15 - 17	HEX	50°F	35 days
18 - 21	HEX	90°F	21 days
22 - 28	PCI	84°F	> 14 days
29 - 31	MEX	70°F	Expansion Completed

HEX: High Expansive concrete

MEX: Medium Expansive concrete

PCI: Portland Cement concrete, Type I

CHAPTER 12

MATERIALS AND PREPARATION

Materials

Cements and Gypsum

The Type I portland cement used in all the experiments was from a single clinker batch. The designation for the portland cement was number 323 in the Joint Highway Research Project Concrete Laboratory at Purdue University.

The calcium aluminate cement was obtained in one shipment and used throughout all the experiments.

The gypsum was a fine ground food and pharmaceutical grade hydrous calcium sulfate. This differs from previous expansive concrete experiments performed at the Joint Highway Research Project at Purdue in that reagent quality gypsum had then been used.

Aggregates

The coarse aggregate used throughout the experiments was a limestone of size number 11 (designation by the Indiana State Highway Standard Specifications). The fine aggregate was a local glacial-alluvial sand. The gradation and absorption were found according to ASTM Standard Tests and were assumed to remain constant throughout all the mixes.

Water

Ordinary tap water was used in all the mixes.

Concrete Mix Proportions

Three types of concrete were used in the columns. The mix designs for these three are shown in Table 12-1. The mixes for the portland, high expansive and medium expansive all had the same aggregate content and cement factor. The quantities shown are for a one cubic yard batch. The medium expansive mix had a slightly larger water-cement ratio. The terms "high expansive" and "medium expansive" were coined by Hanume Gowda (1). The mix design is his with the exception of the modified medium expansive water-cement ratio. The modified water-cement ratio was necessary to provide a workable medium expansive mix.

Steel Tubing

Each tubular column was taken from a nominal 4 inch finish annealed structural steel circular tubing meeting grade designation 1020 of ASTM A519 standards for seamless steel. The measured outside diameter was 4.002 inches with the inside diameter being 3.810 inches yielding a wall thickness of 0.096 inches. The load strain curve for the steel section is shown in Figure 12-1 developed from an axial compression test on a 4 inch long

Table 12-1 Concrete Mix Designs.

Concrete Type	Portland	High Expansive	Medium Expansive
Water-Cement Ratio	0.40	0.40	0.42
Cement Content/Cubic Yard	855. lb	855. lb	855. lb
% Portland	100%	67.5%	65.0%
% Calcium Aluminate	0	17.5%	20.0%
% Gypsum	0	15.0%	15.0%
Coarse Aggregate/yd^3	1119. lb	1119. lb	1119. lb
Fine Aggregate/yd^3	1397. lb	1397. lb	1397. lb

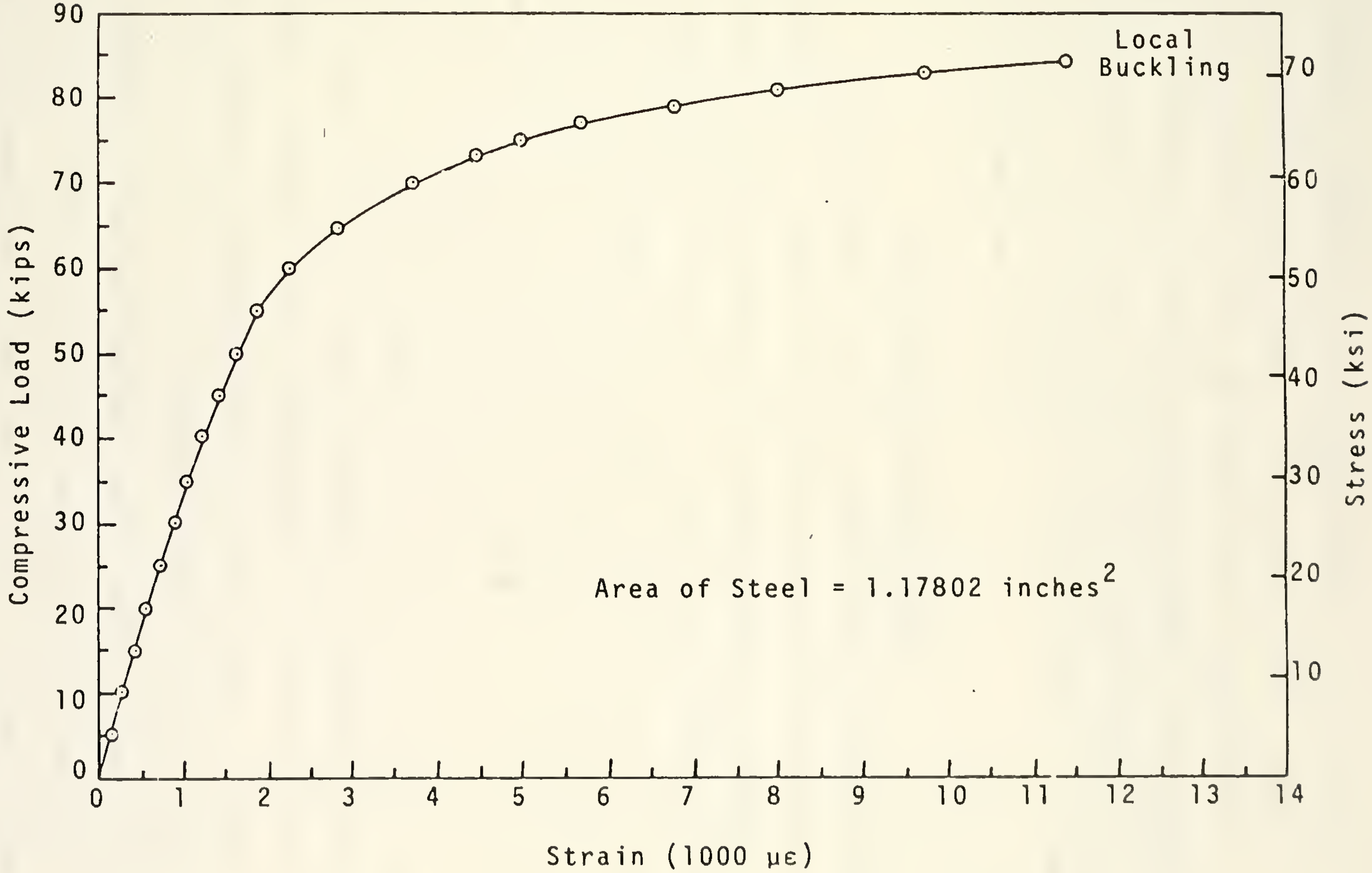

Figure 12-1 Steel Tubing Load Strain Relationship.

specimen. Table 12-2 also shows the data for this test. Figure 12-2 shows the steel specimen after being tested. Load strain tests were performed only in compression. For purposes of analyzing tensile stress, it is simply assumed that the steel exhibits the same behavior in tension. At the longitudinal strain of 3000 $\mu\varepsilon$, Poisson's ratio averaged out to 0.29.

Fabrication of the Tubing

The steel tubes used for each column were fabricated identically. Each tube was 16.00 inches long with the constant outside diameter of 4.002 inches and inside diameter of 3.810 inches. Six 3/8 inches diameter holes were located symmetrically about the perimeter 3/8 inches from each end. Through these holes were attached the end plates. Each end plate consisted of a 1/2 inch thick, 3.809 inch diameter machined steel disk fixed to a five inch square 1/4 inch steel plate. The disk section could be inserted into the tube to provide the bearing surface against the concrete. The attached 1/4 inch plate provided the bearing surface against the steel tubing. The end plate and tube could be clamped together by two steel ring clamps fixed by six 3/8 inch diameter bolts tapped into the inserted disk. Each end plate was identical with the exception of the grout holes. For each column the top end plate had three 1/8 inch diameter holes through the disk and plate to allow for release of excess

Table 12-2 Properties of the Steel Section.

SECTION PROPERTIES

AREA OF STEEL	=	1.17802 INCHES**2
AREA OF CONCRETE	=	11.40092 INCHES**2
STEEL MOMENT OF INERTIA	=	2.24797 INCHES**4
CONCRETE MOMENT OF INERTIA	=	10.34355 INCHES**4

STEEL TUBE PROPERTIES

LOAD (KIPS)	STRESS (PSI)	AXIAL STRAIN (MICRO IN/IN)	TRANSVERSE STRAIN	POISSONS RATIO	SECANT MODULUS (PSI)	TANGENT MODULUS (PSI)
-0	0	-0	-0	-.303	0	29221366.
5.0	-4244.4	-145.2	44.0	-.303	29221366.	28343262.
10.0	-8488.8	-295.0	81.0	-.275	28775617.	28248942.
15.0	-12733.2	-445.2	124.5	-.280	28597890.	30317168.
20.0	-16977.6	-585.3	163.0	-.279	29009165.	27164182.
25.0	-21222.0	-741.5	206.5	-.278	28620388.	26362755.
30.0	-25466.4	-902.5	250.0	-.277	28217641.	28202016.
35.0	-29710.8	-1053.0	295.0	-.280	28215408.	23979681.
40.0	-33955.2	-1230.0	342.5	-.278	27605876.	23844963.
45.0	-38199.6	-1408.0	394.0	-.280	27130420.	19469741.
50.0	-42444.0	-1626.0	454.5	-.280	26103342.	16515189.
55.0	-46688.4	-1883.0	530.0	-.281	24794710.	10480009.
60.0	-50932.8	-2288.0	660.5	-.289	22260857.	7446322.
65.0	-55177.2	-2858.0	829.0	-.290	19306244.	4823186.
70.0	-59421.6	-3736.0	1099.5	-.294	15896642.	3432132.
73.0	-61968.3	-4480.0	1326.5	-.296	13832208.	3215457.
75.0	-63666.1	-5008.0	1470.0	-.294	12712870.	2358002.
77.0	-65363.8	-5728.0	1730.0	-.302	11411280.	1576380.
79.0	-67061.6	-6805.0	2010.0	-.295	9854750.	1397334.
81.0	-68759.3	-8020.0	2465.0	-.310	8573483.	960272.
83.0	-70457.1	-9788.0	2970.0	-.303	7198314.	677021.
84.3	-71560.6	-11418.0	0	-.303	6267354.	0

Fig. 12-2. Tubular Steel Stub after Compressive Load Test.

grout. Figure 12-3 shows the fabricated column.

Mixing and Casting

The methods used for the mixing and casting of the concrete for the composite tubular steel columns are herein described. Experience gained through the investigation and lessons learned from bad results prompted a change of method for mixing the expansive concrete and also initiated the observation of the time of set as described in the following chapter.

Throughout the investigation a total of thirty-one tubular specimens were cast. The steel tubes were ready for casting when the bottom end plate was fixed to the tube. Seven of the specimens were cast with portland cement concrete which was mixed according to ASTM C-192 standard for machine mixed concrete. The fresh concrete was placed into each steel tube in two lifts. Each lift was vibrated for 15 to 20 seconds using an internal vibrator. The second lift came up to a level of approximately 3/4 inch from the top ridge of the steel tube.

Twenty-four specimens were cast with expansive concrete. Modification was made to the mixing procedure after finding six of the first eight specimens cast unsatisfactorily. Specimens #1 and #2 were cast from the first high expansive mix. The mixing procedure followed ASTM C-192. A three minute mix, three minute rest and another two minute mix was the schedule. Therein, the

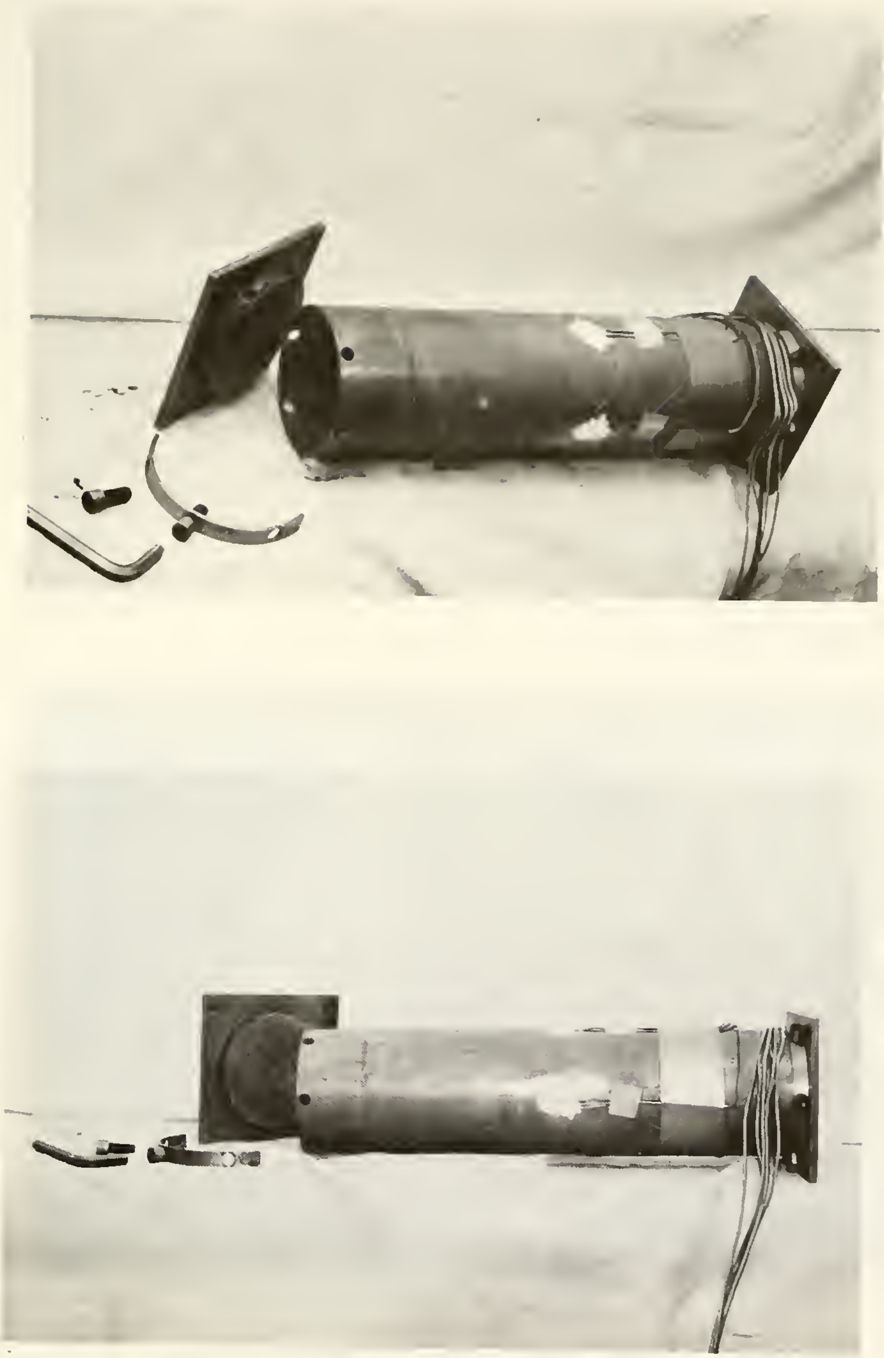

Figure 12-3. Fabricated Tubular Steel Columns.

Figure 12-4. End View of Expansive Concrete Consolidated during Flash Set.

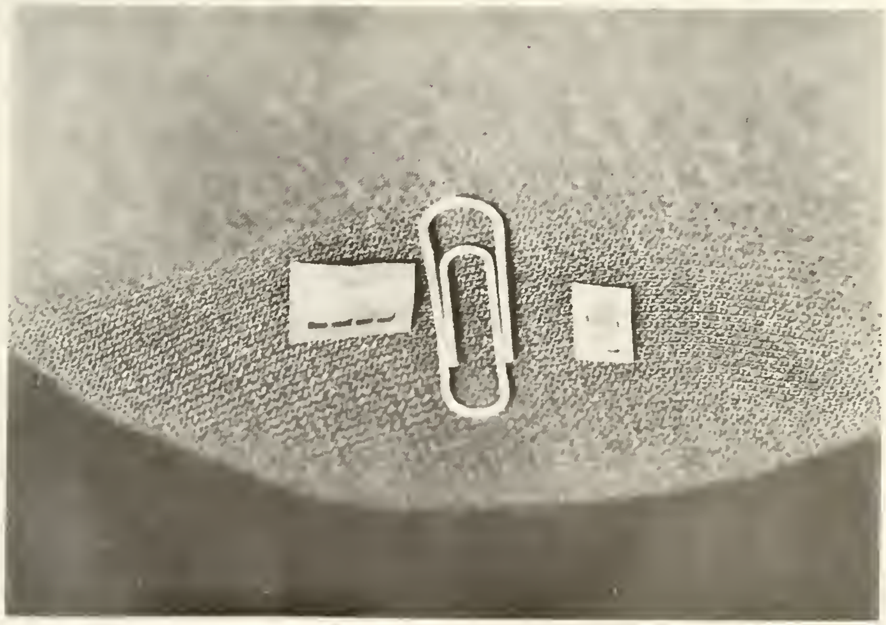

Figure 12-5 One Directional and Two Directional Elect Resistance Strain Gauges.

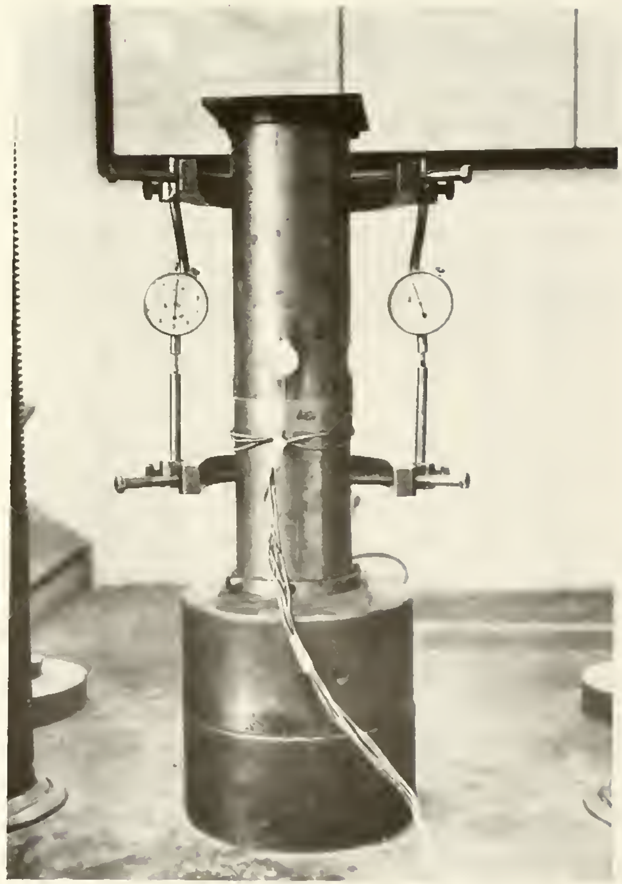

Figure 12-6 Cage for Mechanical Gauging of Column Deflections.

slump test was performed showing a slump of 3/4 inch. Then the first lift was placed in each tube and consolidated. Flash set began to take effect as the second lift was being placed and consolidated. The material became difficult to work, and consolidation was impaired by its harshness.

The second expansive concrete mix from which specimens #3, #4, #5 and #6 were cast was performed again according to ASTM C-192. No special treatments were made to prevent flash set. However, the specimens were cast before the slump test to save time. Immediately upon completion of the mixing, effort was made to cast the specimens. Set was very fast, and attempts to attain sufficient consolidation with the internal vibrator were in vain. The slump test showed a no slump concrete. Figure 12-4 shows how poor the consolidation in the second mix actually was because of the flash set.

Specimens #14 and #15 were cast from the third mix which was made again according to ASTM C-192. However, there was one change. This time cool, 17°C, tap water was used in the mix versus the room temperature, 24°C, water used in the first two mixes. The results were considerably improved with a 1/2 inch slump. Casting and consolidation were easier because of the delayed set.

At this point a permanent modification was made in the mixing procedure. First, the mixing water was always cooled to lower than 15°C. Second, rather than follow the schedule of mixing for three minutes, covering and resting for three minutes, and then mixing another two minutes, the time in the mixer was cut in half. The concrete materials were simply mixed for four minutes non-stop. Each mix of expansive concrete followed this procedure, and workable mixes were obtained regularly.

To complete the casting of each composite tubular specimen, a grout was used to fill the void between the concrete and the top end plate. For all specimens the grout was a portland cement paste with a water-cement ratio of 0.37. This paste was mixed according to ASTM C-305 Standard Method for Mechanical Mixing of Hydraulic Cement Pastes. Before putting the end plate on, this paste was placed so as to cover in excess the concrete inside the tube which had been levelled off inside the tube at approximately 3/4 inch below the tube's top ridge. When the end plate was applied by inserting the steel disk into the tube, the excess grout escaped through the three grout release holes in the top end plate. Thus all the voids were filled, and the concrete was completely confined.

Instrumentation

The Electrical Resistance Strain Gauges. The electrical resistance strain gauges came from Micro Measurements of Romulus, Michigan. Two styles of the same family of gauges were used. Both styles were constantan strain gauges with open faced construction on tough, flexible polyimide film backings as shown in Figure12-5. The first labelled EA-06-125AD-120 was capable of measuring strains in one direction. The second labelled EA-06-125TM-120 had two directional capabilities for strains 90° apart. The gauges were applied and protected according to methods recommended by the manufacturer (2).

Gauge Location and Orientation. Four electrical strain gauges were mounted on each tubular specimen at its mid-height. Two of the four were the one directional gauges mounted diametrically opposite each other. Then at a 90° rotation around the tube from these, two of the two-way gauges were similarly mounted diametricly opposite each other. The orientation was such that four longitudinal readings could be made while two hoop or transverse directional strains were measured. With the strains labelled A, B, C, D, BT and DT, the same orientation was used on all tubular columns including the four inch steel tubing stub observed for the steel properties.

Mechanical Strain Gauging

A mechanical strain gauge was employed during the column load tests to check the performance of the electrical strain gauges. It consisted simply of a steel cage mounted about the tubular columns. It had two Ames dial gauges calibrated to units of 0.0001 inch, and both were fixed on the cage to be on opposite sides of the column specimen. This gauge is shown mounted in Figure 12-6. With this unit, the total longitudinal deflection was measured over a ten inch span length.

CHAPTER 13

EXPERIMENTAL WORK AND RESULTS

The experimental methods and results for the several investigations are discussed in this chapter. The first investigation was on the effects of curing temperature on the expansions in the confined concrete tubes. The strain histories are plotted for each temperature grouping of columns made with expansive concrete. Further comparison of the temperature effect is made in the discussion of the column tests. Finally observations are made on the time of set of expansive concrete.

Expansion Tests on Steel Tubes Filled with Expansive Concrete

Monitoring the Expansions

The expansive growth of the twenty-four expansive concrete specimens was monitored regularly. Electrical resistance strain gauges were used with readings being taken within the hour of casting and then each day through the first week. As the rate of growth diminished the frequency of readings also was decreased. Specimens were monitored until the day of column testing.

Curing Conditions

The expansive concrete specimens were cured continually in three separate temperature controlled situations. Three high expansive specimens were cured at 50°F. A refrigerated ice box housed these specimens throughout their expansive history. The ice box was situated so that the expansive strains could be monitored without moving the columns. Four high expansive concrete columns were cured immersed in a hot water bath at 90°F. Similarly, this unit was located so that the expansions could be observed without any transport. Finally, thirteen high expansive and three medium expansive composite columns were cured at 70°F. This condition was maintained by locating the specimens under a thermostaticly controlled air conditioner. These columns needed to be moved each time the strains were monitored. During the readings these specimens were exposed to 84°F temperatures for an average of five minutes.

The seven composite columns filled with portland cement concrete were cured at the room temperature of 84°F.

Expansive Strain Histories

When possible, expansive strains were monitored until the expansion was complete. However, in the cases of the column groups cured at 50°F and 90°F, expansive column specimens were tested before expansions reached their peak. Such tests proceeded only after good trends were established for each specimen. Uniformity for the age of test was maintained for specimens cured at the same temperature.

High Expansive Concrete Cured at 50°F. Four high expansive concrete specimens #14, #15, #16 and #17 were cured at 50°F. Out of these, one was used to observe the effects of temperature change on the expansion through an extended period. Thus, until the age of sixteen days all four specimens cured at 50°F. Thereafter only three of them remained at 50°F until the time of test. The average of the results are plotted on Figure 13-1. This figure shows the percentage of expansion in both the longitudinal and transverse directions through the age of thirty-five days.

The expansion in the transverse direction was much larger than that in the axial direction. This should be expected as the percentage of hoop reinforcement was 5.04% while the axial reinforcement represented 10.33% of the concrete area. The expansions were not complete at the time of test, however, the rate of growth had slowed down so much that the trend indicated that the material would not be able to attain any more significant amounts of self-stress. The average transverse expansion was 0.0455% while the longitudinal expansion reached 0.0177%. The ratio of transverse strain to longitudinal strain was 2.57.

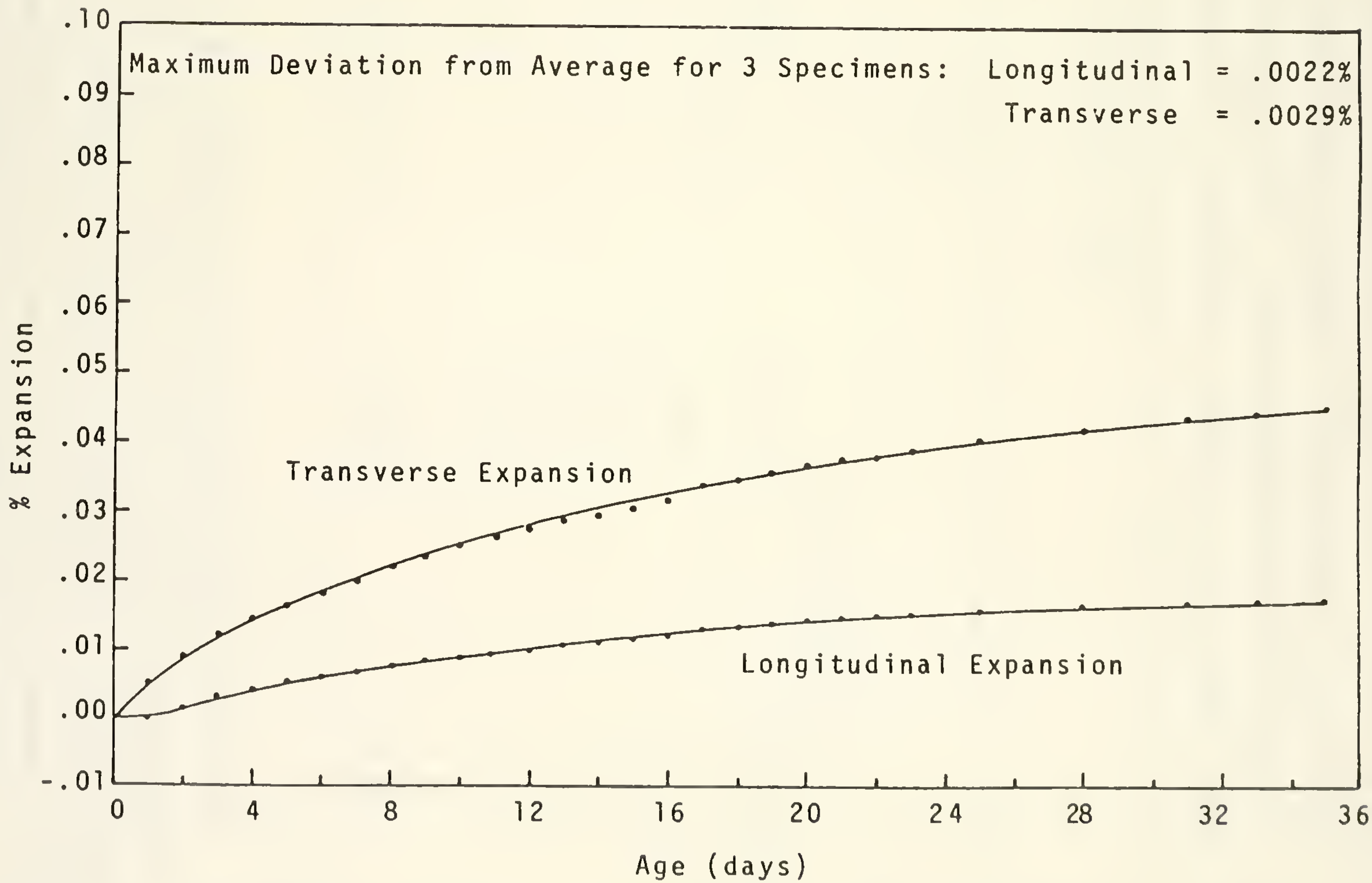

Figure 13-1 Expansion of Tubular Columns Filled with High Expansive Concrete Cured at 50° F.

On Figure 13-1 only the average results are plotted. The deviation between the expansions for specimens #15, #16 and #17 were so small that graphical illustration would be difficult. The maximum deviation from the average for longitudinal expansions was 0.0022%, and in the transverse direction it was 0.0029%. Table 13-1 shows the conformity of data.

Table 13-1. Expansions on High Expansive Specimens Cured at 50°F.

	Maximum Expansion	
Specimen	Transverse	Longitudinal
15	0.0482 %	0.0184 %
16	0.0448 %	0.0189 %
17	0.0435 %	0.0159 %
Average	0.0455 %	0.0177 %
Maximum Deviation -	0.0029 %	0.0022 %

<u>High Expansive Concrete Cured at 70°F</u>. Not all results were so uniform. Seven specimens were cured at 70°F. The range of data was much larger. Figure 13-2 shows the averages for the expansion in these specimens. The figure also shows the range of data. The average

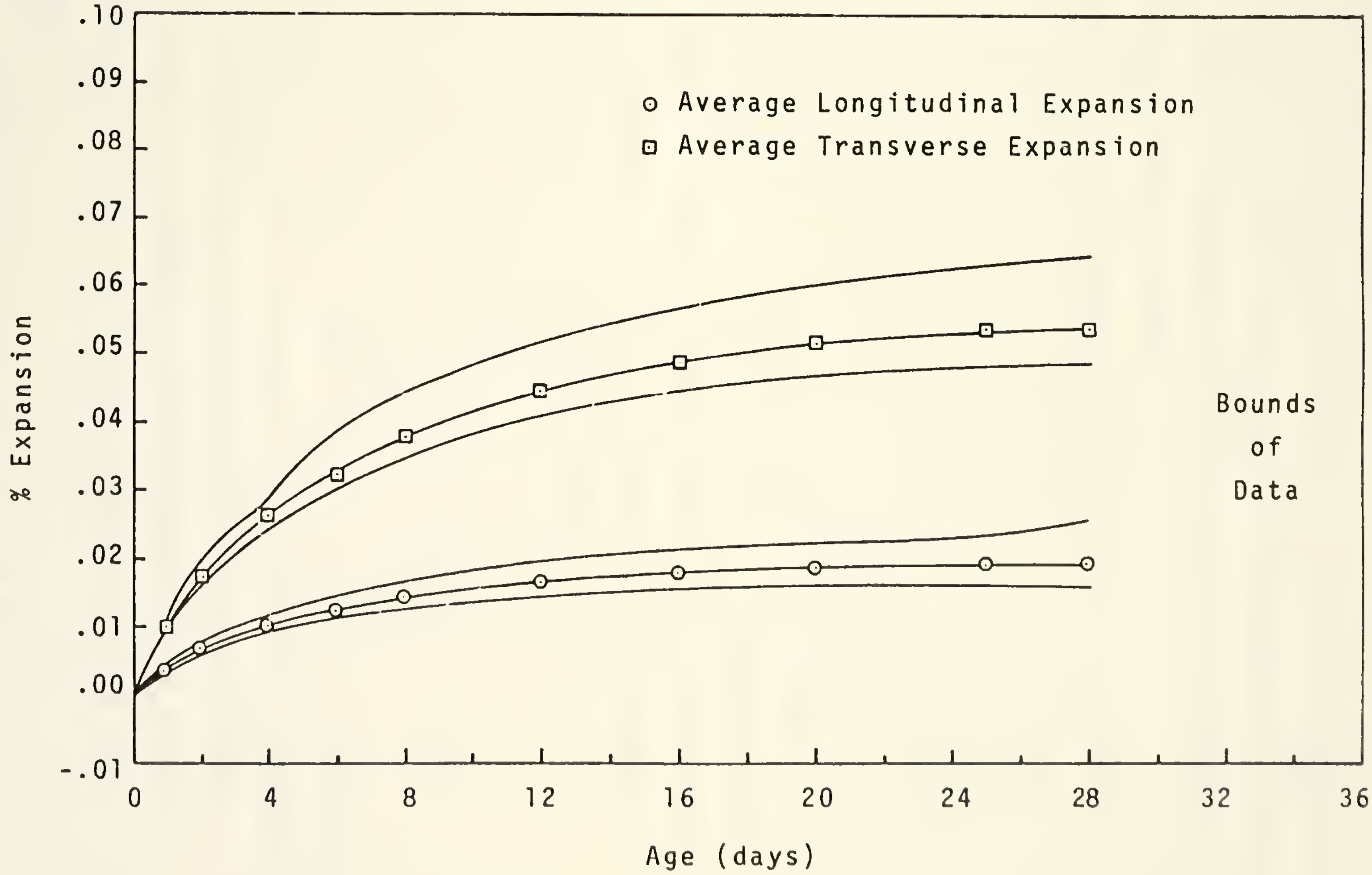

Figure 13-2 Expansion of Tubular Columns Filled with High Expansive Concrete Cured at 70° F.

longitudinal expansion reached 0.0195% and the transverse expansion reached 0.0535%.

Similar to the specimens cured at 50°F, the transverse strain was 2.7 times as large as the longitudinal expansion. Such a ratio is expected since the ratio of longitudinal to transverse steel is 2.05.

High Expansive Concrete Cured at 90°F. Four high expansive specimens were cured immersed in a hot water bath at 90°F. Out of these four, two specimens showed suspicious readings for expansive strains. A flutter on the electrical strain indicator indicated that some shorting was occurring on the gauges of two columns. The protective coatings over the electrical resistance strain gauges simply were not providing total waterproofing for either the leadwires or the constantan foils. Because of this, the strain readings for specimens #20 and #21 were discarded as poor data.

Specimens #18 and #19 provided good data without any of the suspicious readings being observed as on specimens #20 and #21. Expansions were observed occurring quite rapidly. Only two hours after casting and immersion in the 90°F bath the expansion equalled one-fourth the total expansion attained by the specimens cured at 50°F for thirty-five days. In twelve hours this mark was surpassed.

These 90° specimens displayed some unique behavior. First, the ratio of transverse expansion to longitudinal was approximately one. See Figure 13-3. Hence, the higher effective amount of steel restraint in the longitudinal direction appeared to make no difference on the rapid expansive movement. Secondly, after twenty-one days of curing, the expansion appeared to be far from complete.

Curing was terminated, and column tests were performed at the age of twenty-one days. The average transverse expansion attained at this time was 0.1818%. Meanwhile the average longitudinal expansion reached 0.1899%

<u>High Expansive Concrete Cured at 50°, 70° and 90° Sequentially</u>. Specimen #14, which was mentioned under the category of specimens cured at 50°F, is the one specimen observed under this unique curing sequence. The intent was to see what effect increased temperature had on a specimen whose expansion was approaching the dormat stage.

After curing for sixteen days at 50°F, as illustrated by Figure 13-4, this specimen was exposed to the 84°F room temperature for two hours. Then it was returned again to the 50°F environment. Up to this point the expansion was slowly tapering off. However, that two hour warming caused a distinct increase in the expansive rate, observed for the two following days. By the fourth day after this event the growth appeared to be complete again. This was the twentieth day after casting.

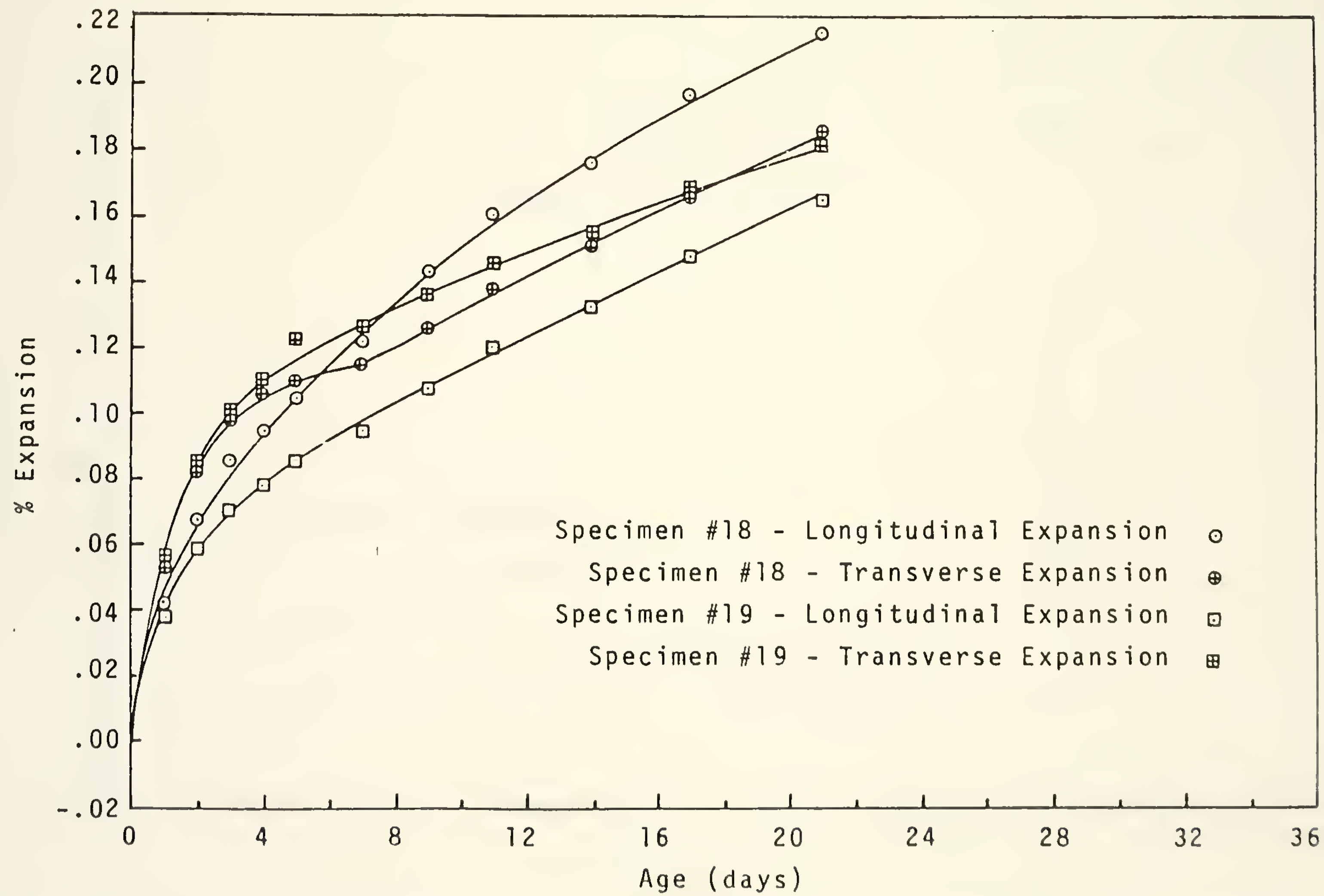

Figure 13-3. Expansion of Tubular Columns Filled with High Expansive Concrete Cured at 90° F.

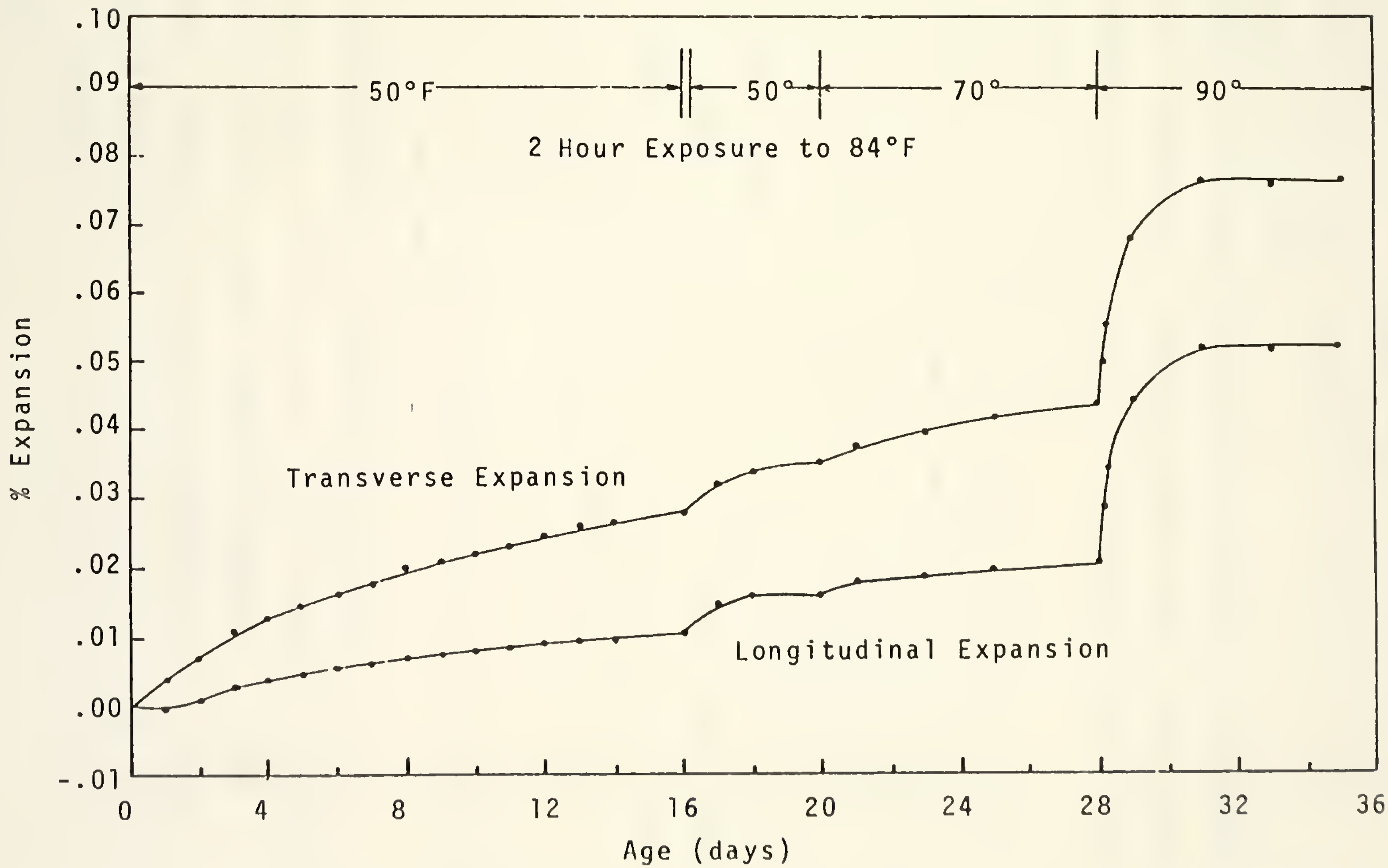

Figure 13-4. Expansion of a Tubular Column Filled with High Expansive Concrete Cured at 50°, 70° and 90°F Sequentially.

At the age of twenty days, the specimen was transferred to be kept at 70°F. Through the ensuing eight days expansion was revived. However, the effect was not particularly dramatic.

When the twenty-eighth day was reached the specimen was again transferred. Curing at 90°F, the column gave a significant spurt in growth. This growth however was complete after only three days.

The trend for expansion to occur primarily in the transverse direction reappeared here only in the early stages. The ratio of transverse to longitudinal expansion was 2.72 at the end of sixteen days. On the twenty-eighth day it was 2.14. However, the expansion occurring during this 90°F exposure period was 0.0324% transversely and 0.0315% longitudinally. Thus, again during the last rapid period of growth, no bias toward the transverse direction appeared.

The final expansions reached were 0.0762% and 0.0520% transversely and longitudinally.

Medium Expansive Concrete Cured at 70°F. Three specimens, #29, #30 and #31 were cast of medium expansive concrete to observe their expansive behavior. These specimens in Figure 13-5 showed considerable bias toward transverse expansion. The ratio of transverse to longitudinal expansion was 2.90 when the maximum strain was reached.

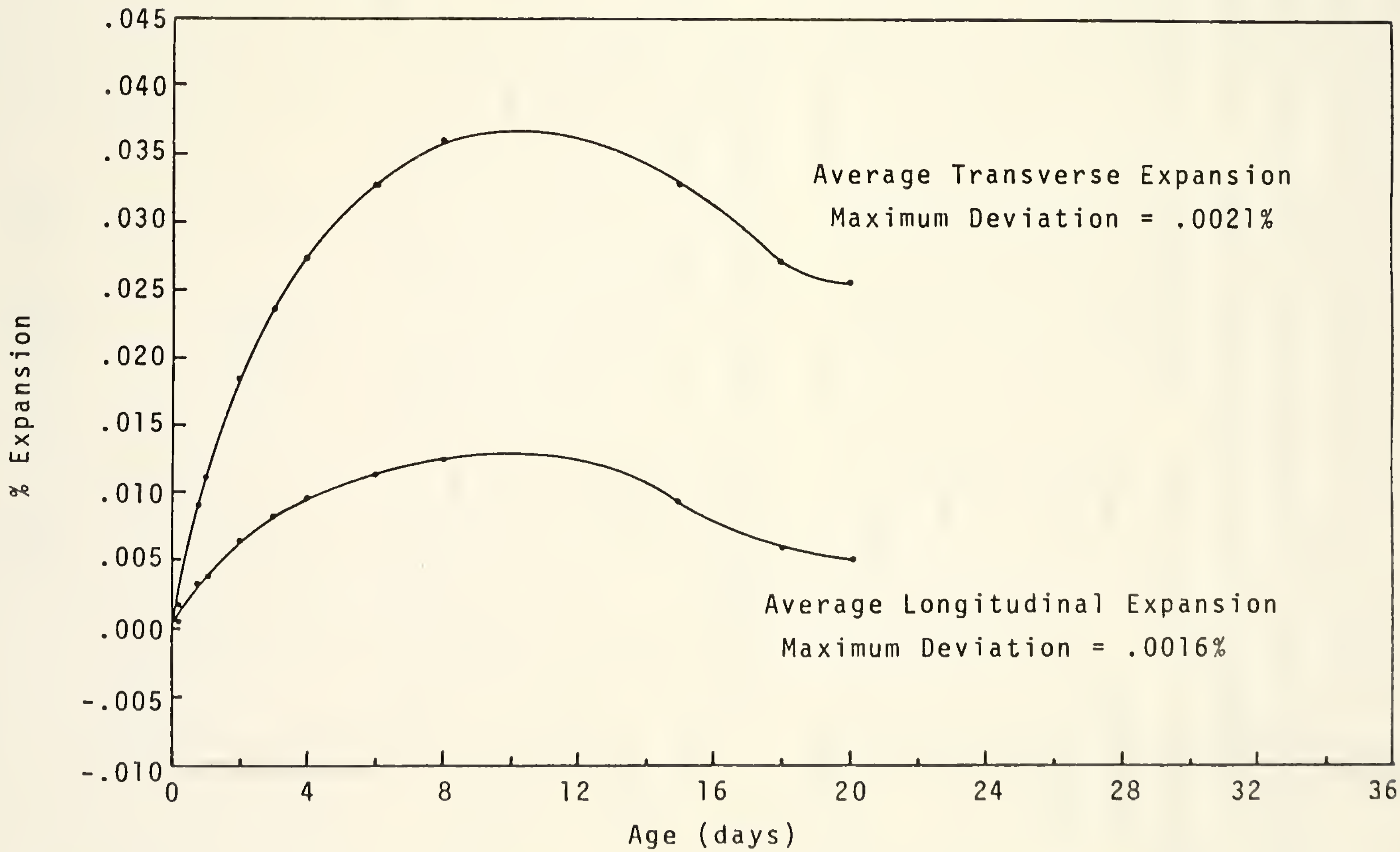

Figure 13-5. Expansion of Tubular Columns Filled with Medium Expansive Concrete Cured at 70° F.

The overall growth was rather small. Maximum growth was achieved in only eight days. However, the effects of creep soon negated this early growth. By the age of twenty days, creep reduced the total expansion to 0.0256% and 0.0053% transversely and longitudinally.

Summary of the Expansions. The effects of temperature on expansion are very important. Figure 13-6 illustrates the transverse expansive strain histories of the several groups of expansive column specimens. Because of the sensitivity to temperature, the amount of self-stress developed can only be duplicated through very strict control of the material proportions and curing temperature.

Self-Stress Developed Through Expansion

Very sizeable stresses were induced in both the steel tubing and the expansive concretes. The stresses in the concrete can be determined from the stresses in the steel, which in turn can be analyzed from the steel's basic stress-strain relationship and the expansive strains developed. The following relations shall be used:

$$\sigma_L^s = \frac{E_s}{1-\nu^2} (\varepsilon_L + \nu\, \varepsilon_T)$$

and

$$\sigma_T^s = \frac{E_s}{1-\nu^2} (\varepsilon_T + \nu\, \varepsilon_L)$$

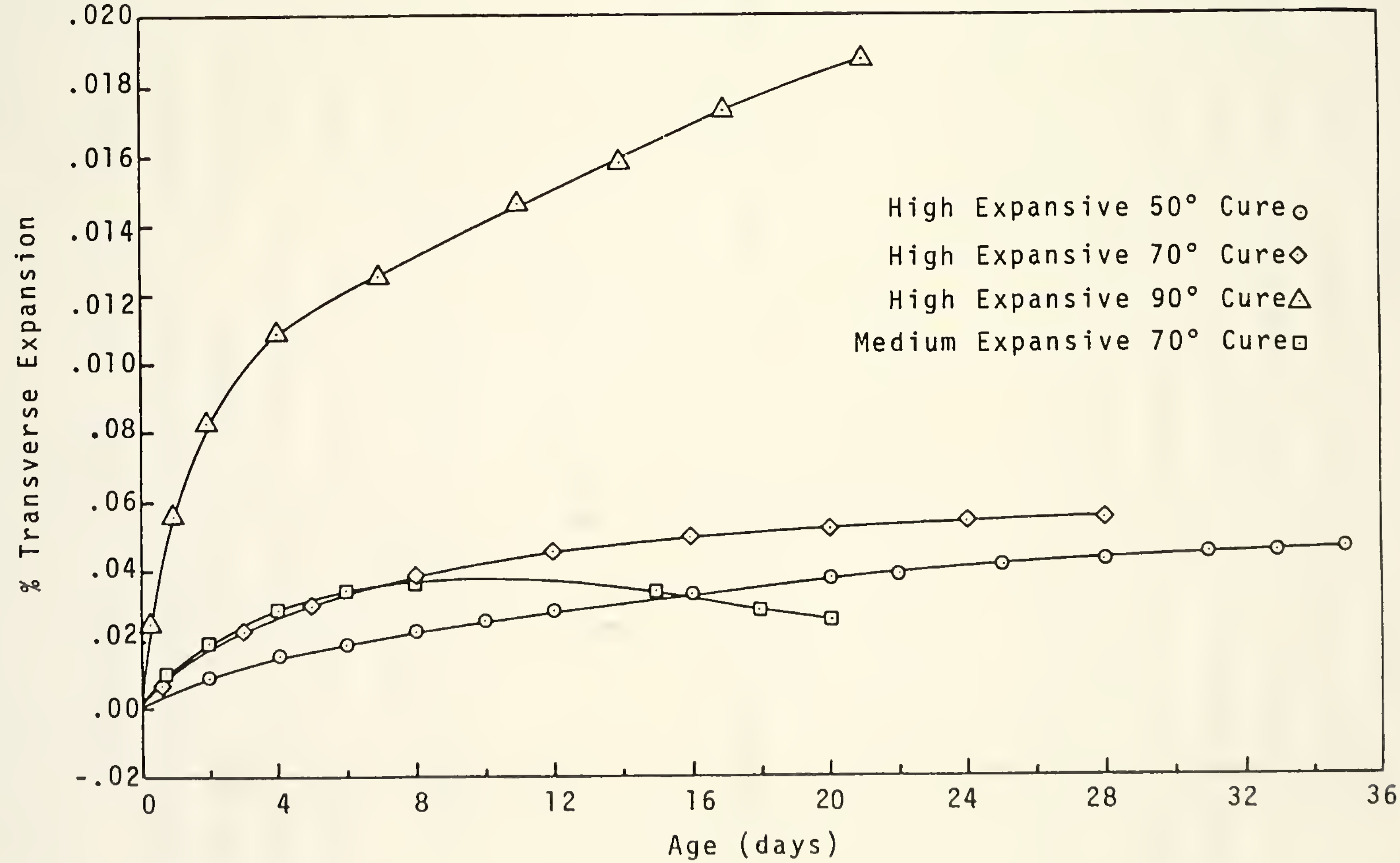

Figure 13-6. Summary of Transverse Expansions for Expansive Columns Cured at 50°, 70° and 90°F.

The longitudinal and transverse stresses in the steel, σ_L and σ_T, are dependant upon the steel modulus, E_s, poisson's ratio, ν, (equal to 0.29) and the strains induced longitudinally and transversely, ε_L and ε_T.

The stresses in the concrete can be evaluated based upon the requirement of internal equilibrium. The longitudinal ratio of steel to concrete was 10.33%. Thus the axial stress in the concrete is 10.33% of the steel longitudinal stress.

$$\sigma_L^c = -0.1033\ \sigma_L^s$$

Similarly, the transverse concrete stress follows this expression:

$$\sigma_T^c = -0.0504\ \sigma_T^s$$

The self-stresses developed in each of the column groups are tabulated in Table13-2. Special attention should be drawn to the high expansive concrete columns cured at 90°F. The expansions strained the steel into the nonlinear range. Therefore, the modulus E_s used was the secant modulus for the strain of 1818$\mu\varepsilon$. Also, since ε_L and ε_T were so large and close, an approximating assumption was made that the expansions were uniform in all directions. The steel stresses were then analyzed

Table 13-2. Self-Stress Developed Through Expansion.

				Stresses (psi)			
	Expansive Strains ($\mu\varepsilon$)		E_{sec}	Steel		Concrete	
Column Group	ε_L	ε_T	10^6psi	σ_L^s	σ_T^s	σ_L^c	σ_T^s
High Expansive, 50°F	177	455	28.75	9,700	15,890	1,000	800
High Expansive, 70°F	195	535	28.75	10,990	18,570	1,135	935
High Expansive, 90°F	1899	1818	25.13	64,350	64,350	6,650	3,245
High Expansive, 50° - 70° - 90°F	526	702	28.75	23,260	28,650	2,400	1,445
Medium Expansive, 70°F	53	256	28.75	3,990	8,520	410	430

based on a transverse and longitudinal strain of 1818$\mu\varepsilon$ for each.

The Column Load Tests

Specimen Ages at the Time of Test

It was sought to test the expansive columns at the age in which all expansion was complete. This was possible only in two cases, the high expansive specimens cured at 70°F and the medium expansive specimens also cured at 70°F. These tests were performed at twenty-eight and twenty days respectfully. Within any one group of specimens the test ages were the same.

The high expansive concrete specimens cured at 50°F and 90°F had to be tested before expansion was complete. However, good trends were established for each group, and they were tested after thirty-five and twenty-one days of curing respectively.

The seven portland cement concrete specimens were cured and scheduled to be tested at the age of fourteen days. However, only two of the seven were tested at this age. When the third column, specimen #23, was being tested on a different day, mechanical failure was discovered in the universal testing machine. The column was accidently loaded without any readings being indicated. When released, the residual strains were recorded. Column #23 was retested six days later beginning at the residual strain readings. Column #23's results were not included in the discussion, but its test results are listed in the Appendix. Because of the mechanical failure, tests on the remaining four portland cement specimens were postponed until the ages of eighteen and nineteen days. It was assumed that the concrete strength did not grow appreciably during these four and five extra days.

Column Test Procedure

Each composite concrete filled tubular column was tested under axial compression to determine its load strain relationship. The loads were applied in regular increments by a universal testing machine. At each increment of load, the loading was temporarily halted so that the several strain readings could be taken. Because of the great ductility of the columns, loading was continually applied as long as the strain gauges performed well. If any local buckling appeared, the loading was terminated. Typically, if buckling occurred, it was at the end connection where the portland cement grout had been placed. Figure 13-7 shows this form of failure. Figures 13-8 and 9 demonstrate the kinds of ductility possible. Specimen #14 had been tested until local buckling appeared. Specimen #15 had not yet been tested.

The strains were measured by means of the four electrical resistance strain gauges mounted at positions

Figure 13-7. Local Buckling at the Grout Filled End of the Composite Column.

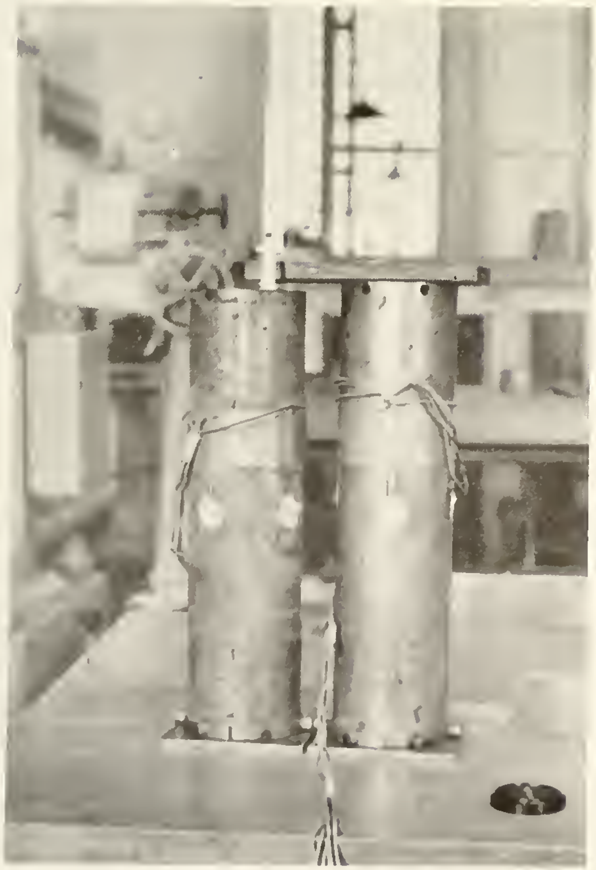

Figure 13-8. Composite Tubular Columns Before and After the Load Test.

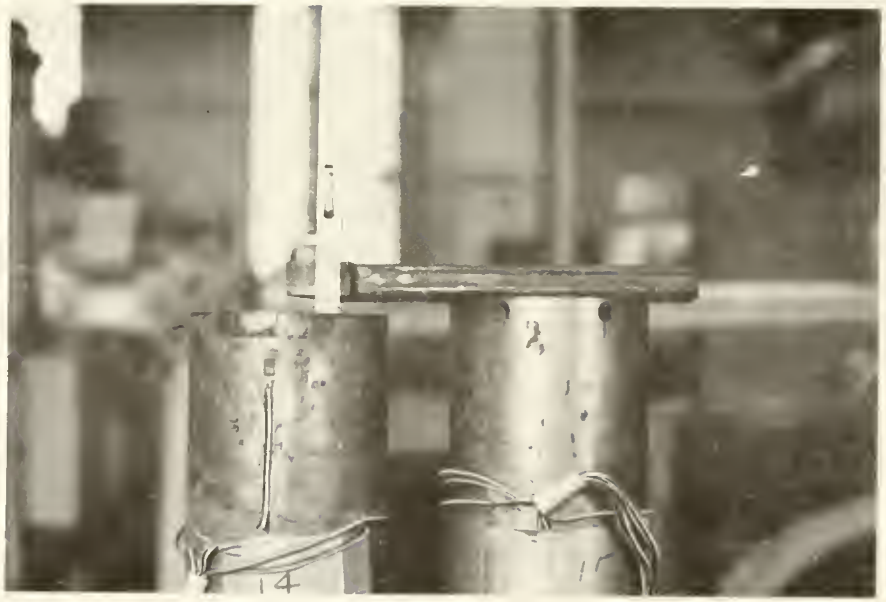

Figure 13-9. Ductility Demonstrated by a 10/32 inch Deflection.

A, B, C and D as described in Chapter 12. The six strain readings were balanced to zero micro-strains after each column had been seated and released three times under a 10,000 lb load. Four of the strain gauges gave longitudinal readings, and two of them gave transverse readings.

The mechanical cage was used to measure deflections on all of the composite columns with expansive concrete except specimens #14 and #15. The cage in operation in Figure 13-10 was used simply as a check on the electrical strain gauges. Readings were taken with it until the column reached a strain of 0.2% or 2,000 $\mu\varepsilon$. It was then dismounted to guard against any possible damage to the cage.

Results of the Column Tests

The results of the column tests are shown in Figures 13-11 through 15. These results show the spread of the data for each particular group of specimens. The actual tabulated data for each column is listed in the Appendix.

It was found that the electrical strain gauges performed well. Therefore, the results of the mechanical cage deflection readings are not listed. Particular attention was drawn to specimens #20 and #21. These columns were considered useless during the expansion tests of columns immersed in the 90°F water bath. The gauges had been giving suspicious readings during immersion. However, when the gauges were completely air dried,, they worked well.

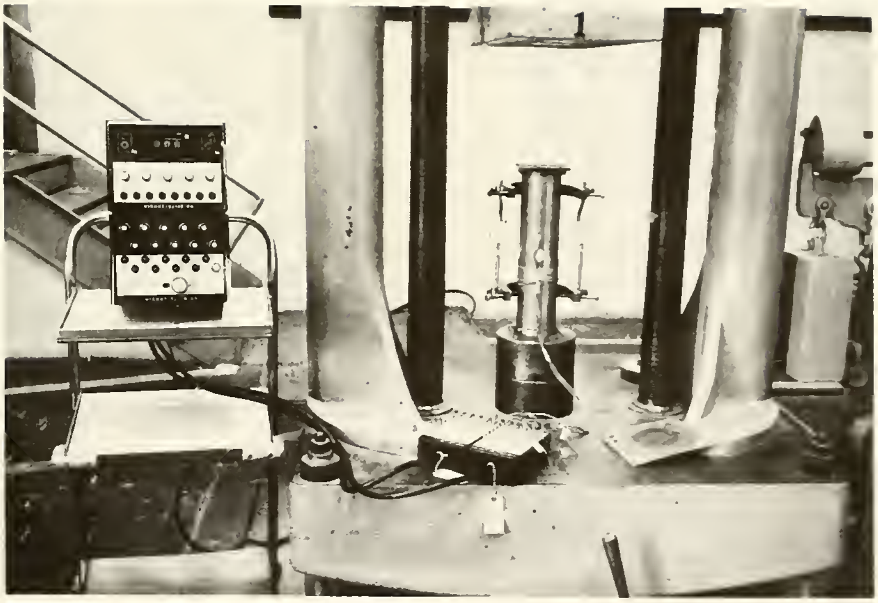

Figure 13-10. Mechanical Cage Checking Electrical Strain Gauge Readings.

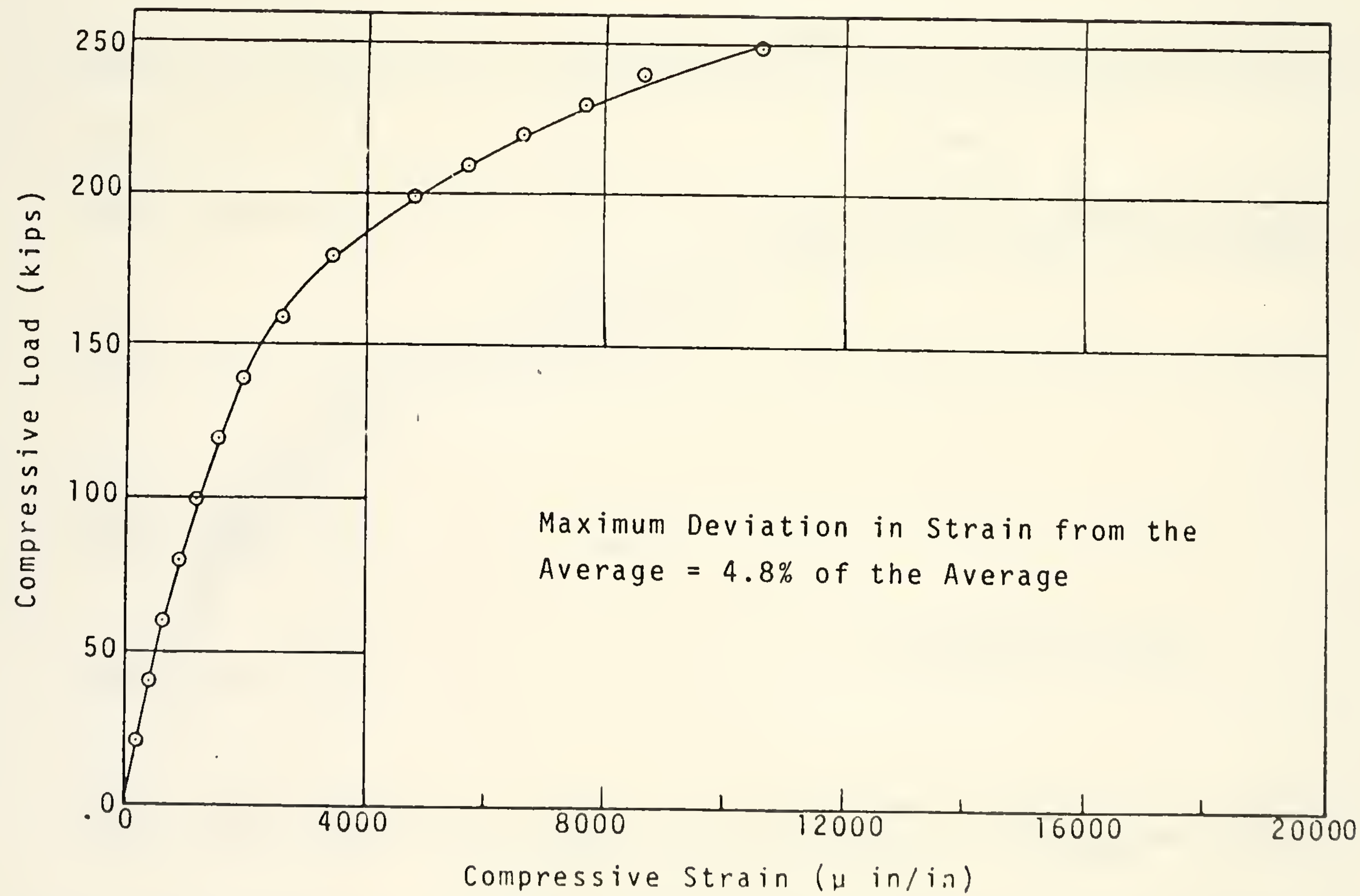

Figure 13-11. Load Strain Relationship for Tubular Columns Filled with High Expansive Concrete Cured at 90°F.

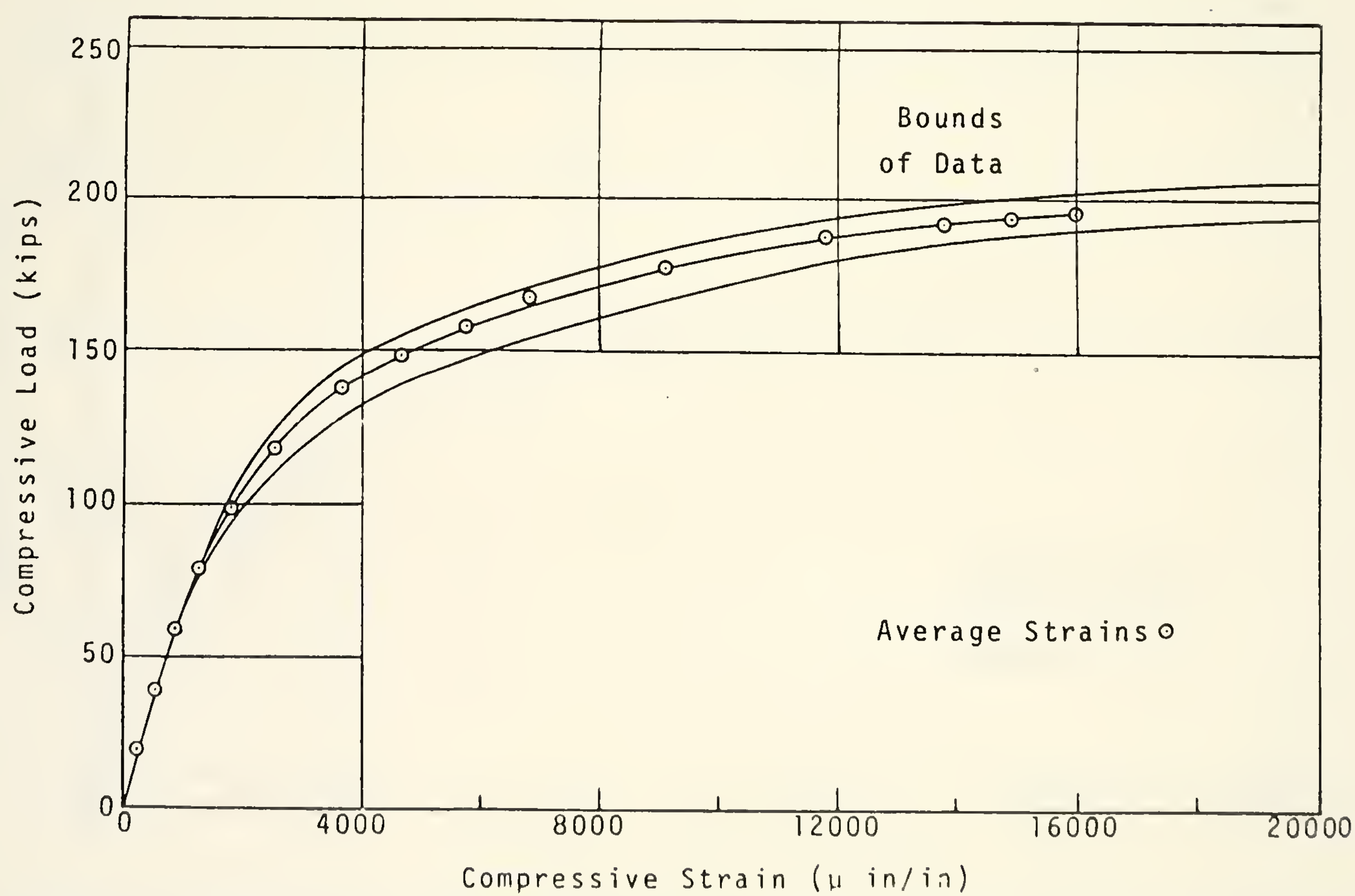

Figure 13-12. Load Strain Relationship for Tubular Columns Filled with High Expansive Concrete Cured at 70°F.

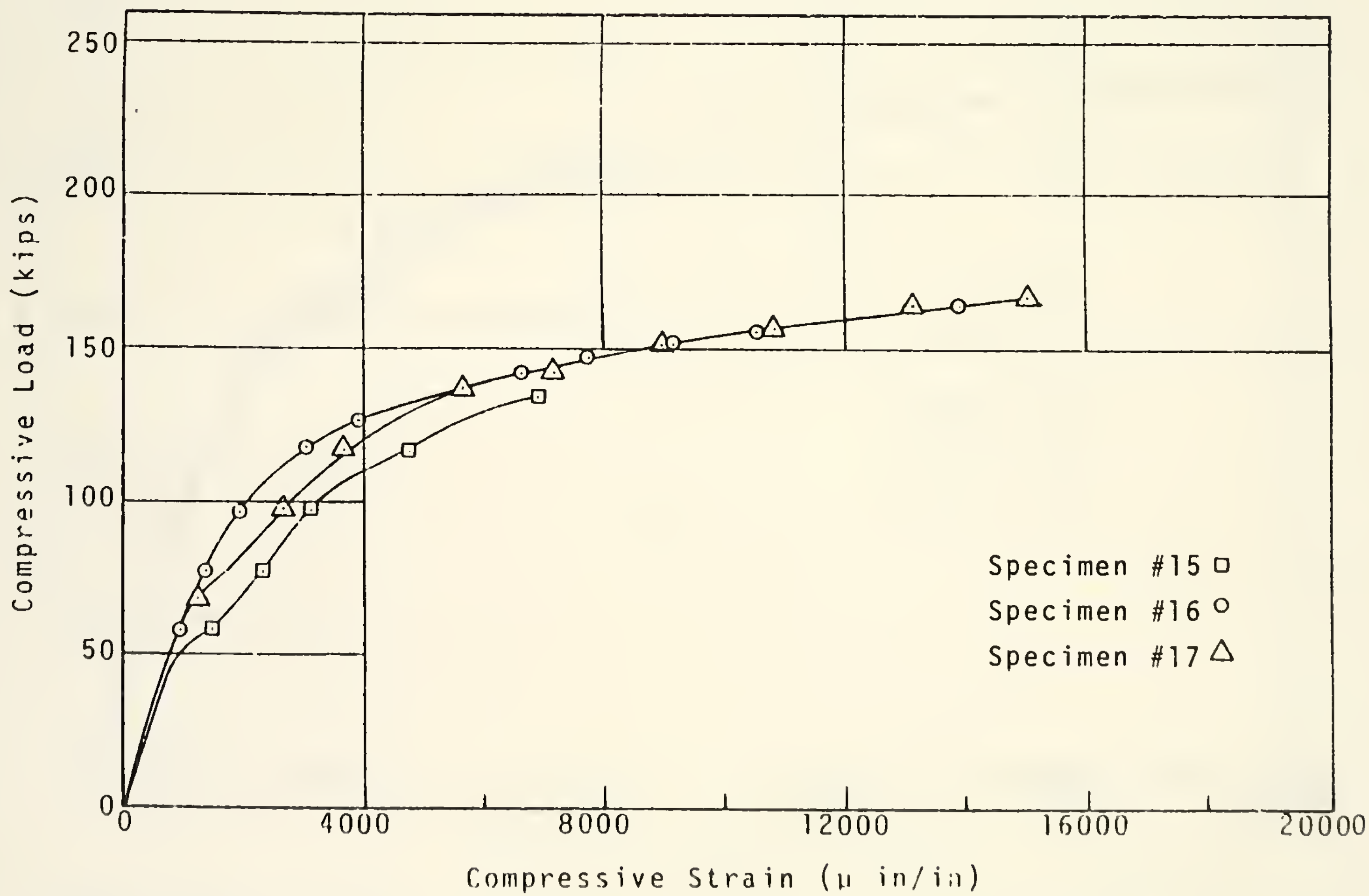

Figure 13-13. Load Strain Relationship for Tubular Columns Filled with High Expansive Concrete Cured at 50°F.

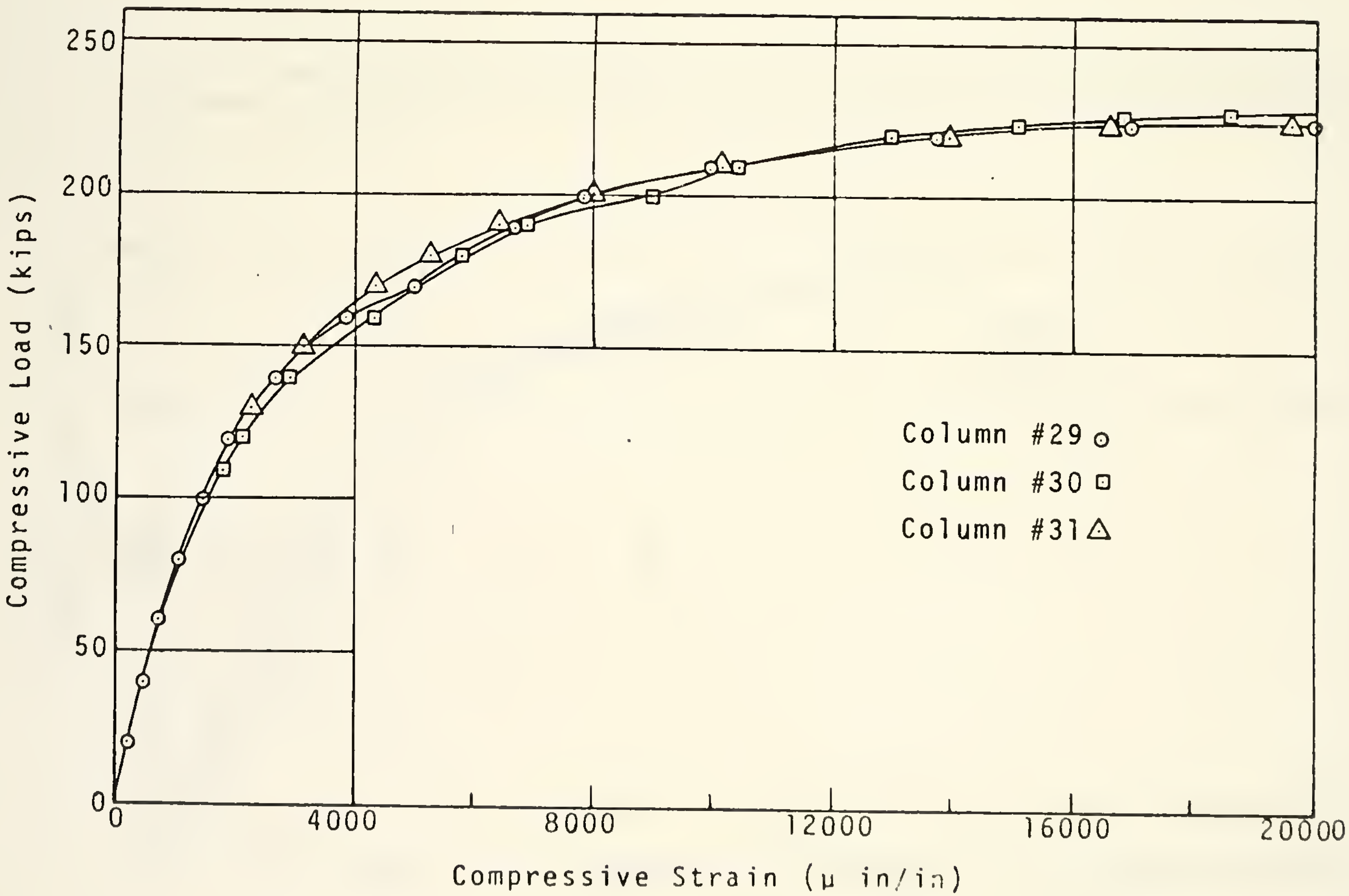

Figure 13-14. Load Strain Relationship for Tubular Columns Filled with Medium Expansive Concrete Cured at 70°F.

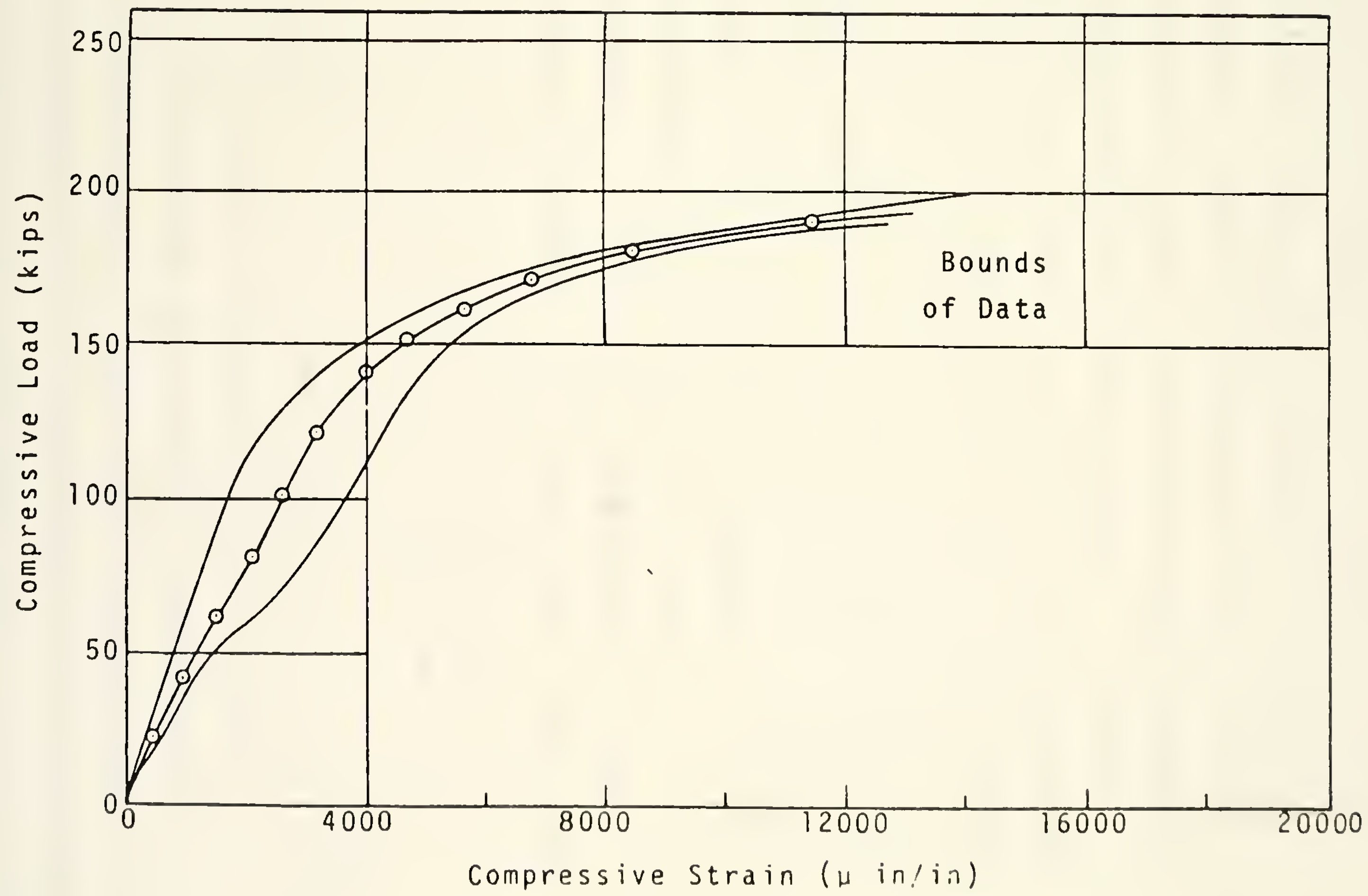

Figure 13-15. Load Strain Relationship for Tubular Columns Filled with Portland Cement Concrete Cured at 84°F.

<u>Uniformity of the Column Test Results</u>. The column load tests on the four specimens cured at 90°F produced extremely uniform results. For this reason, the range of data could not be graphically illustrated. Of all the load levels, the largest deviation in strain from the average was found to be only 4.8% of that average.

The three specimens cast from the medium expansive concrete also displayed uniform results. In fact at the lower and higher ends of the load testing, the load-strain relationships for specimens #29 and #31 coincided.

The range in data for the high expansive 70°F cure is shown as an average with boundaries within which all test results fell. The results of the three separate columns cured at 50°F are plotted together. All of this data was quite uniform at the low load levels.

The tubular columns filled with normal portland cement concrete displayed scattered results. The spread of data was largest in the lower working range of the columns, but at the upper levels of load, the columns began to close in on the average. The effect was opposite to the expansive specimens which were uniform at the lower levels and diverged at the larger levels of load.

The confinement in the expansive specimens was continually active at all loads. This continuity of interaction between the steel tubing and expansive concrete was the reason for such reproduceable results at the lower loads. However, for the columns with portland cement concrete, the confinement effects in the various specimens were probably occurring at different points along the way. That is, the confinement was discontinuous. The confinement probably went from tensile stresses at the interface to no stresses and then to compressive confining stresses. This is because Poisson's ratio for the steel tubing is 0.29. For the concrete it is between 0.15 and 0.20 at its lower strain levels (3). Thus, under axial compression the circumferential strain of the steel tubing would be larger than that in the concrete core. The steel tube then tends to spread apart from the concrete. This effect plus the effect of concrete shrinkage would promote separation of the tube from the concrete. Which point this occurs at would be hard to predict since the separation must first overcome the concrete's tensile bonding force to the steel tube. Assuming material separation does occur, then the two materials would be acting independently in sharing the compressive load. There would be zero confining stresses.

In general, as concrete approaches it ultimate unconfined compressive strain in the neighborhood of 3,000 to 4,000 $\mu\varepsilon$, then the negative volume change of the concrete under additional load dissipates. Simultaneously, Poisson's ratio begins to approach its limiting value of 0.5. Then during the larger axial compressive loads the lateral deflection of the enclosed concrete becomes greater than the steel tube. Soon the passive confinement by the tube is attained.

With the lateral confinement in force, the concrete becomes more ductile and capable of taking higher axial loads. In turn the steel tubing gains the benefit of continuous lateral support thus bolstering it against local buckling.

Effect of Curing Temperature. The structural performance of the high expansive columns was dependent upon the curing temperature. The specimens cured at 50°F were the poorest both in the working range and at ultimate load. However, this strength was increased markedly by exposing the column to higher temperatures. Figure 13-16 shows the improvement for specimen #14 which was cured at 50°F and then subsequently exposed to 70°F and 90°F environments.

The group of high expansive specimens cured at 70°F did not develop much more self-stress than the 50°F group, but it did reflect marked strength improvements. The 90°F cured columns had temendous strength. In the tests of this latter group the ultimate strength was not reached because of the failure of the electrical strain gauges. Yet the trend of the load-strain curve indicated that it could reach 260 kips. This would be a strength improvement over the 50° group by 60% and over the 70°

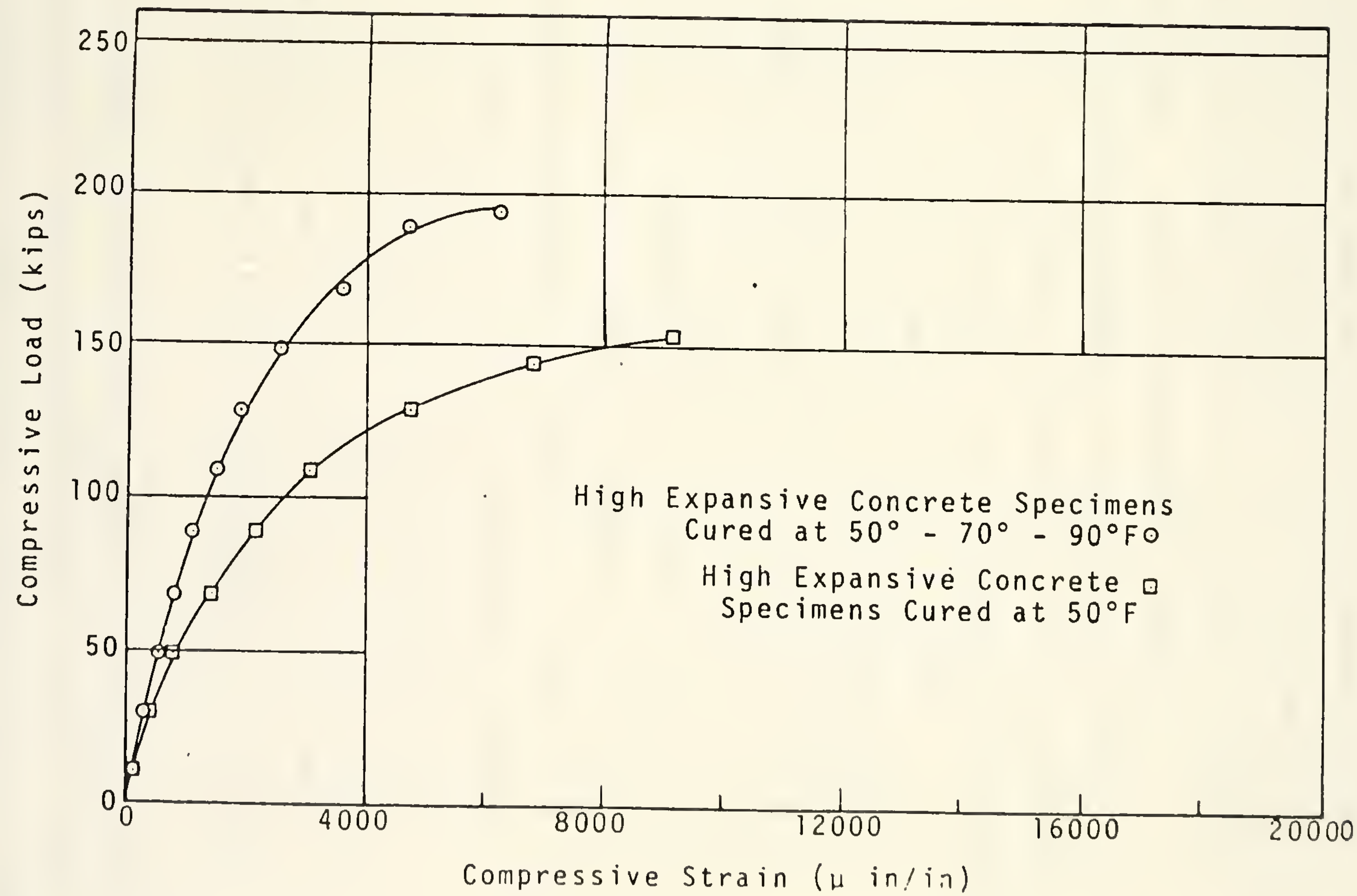

Figure 13-16. Load Strain Relationships for Tubular Columns Filled with High Expansive Concretes Cured at 50°F and 50° - 70° - 90° Sequentially.

group by 20%. All of the results are summarized in Figure 13-17.

In the lower working range of the columns emphasized by Figure 13-18, the same ranking in performance existed. Table 6 shows the longitudinal strains required to resist the compressive load of 60 kips. A relative stiffness is also shown. For this an arbitrary value of 60 kips/863 $\mu\varepsilon$ was assigned the relative stiffness of one. Thus, the relative stiffness of the high expansive group cured at 70°F was one. The 50°F group was 23% less stiff while the 90°F group was 28% more stiff.

Comparison of the Portland, High Expansive, and Medium Expansive Concrete Columns. The high expansive concrete columns have already been discussed. However, they need to be compared with the medium expansive and normal portland cement concrete columns. The results are presented in Figures 13-17 and 18 and in Table 13-3.

The medium expansive group showed good strength. Cured at 70°F and compared to the high expansive 70°F group, it was much better. In spite of having a lower circumferential confining stress, it was still 14% stiffer and had an ultimate strength approximately 13% higher.

The normal portland cement concrete tubular columns yielded such a range of results at the low working loads that producing a discreet number to represent the stiffness

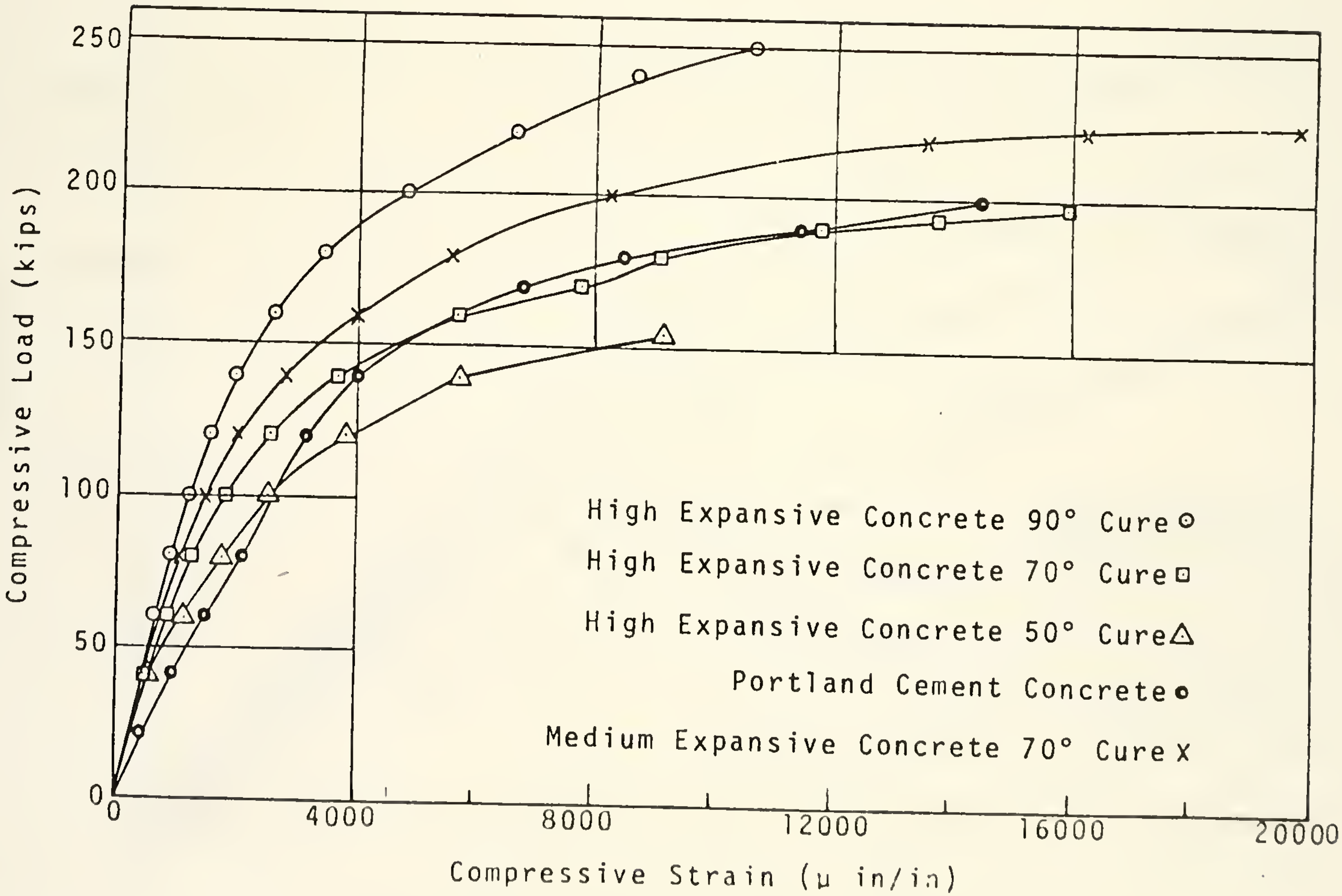

Figure 13-17. Load Strain Relationships for Composite Tubular Columns.

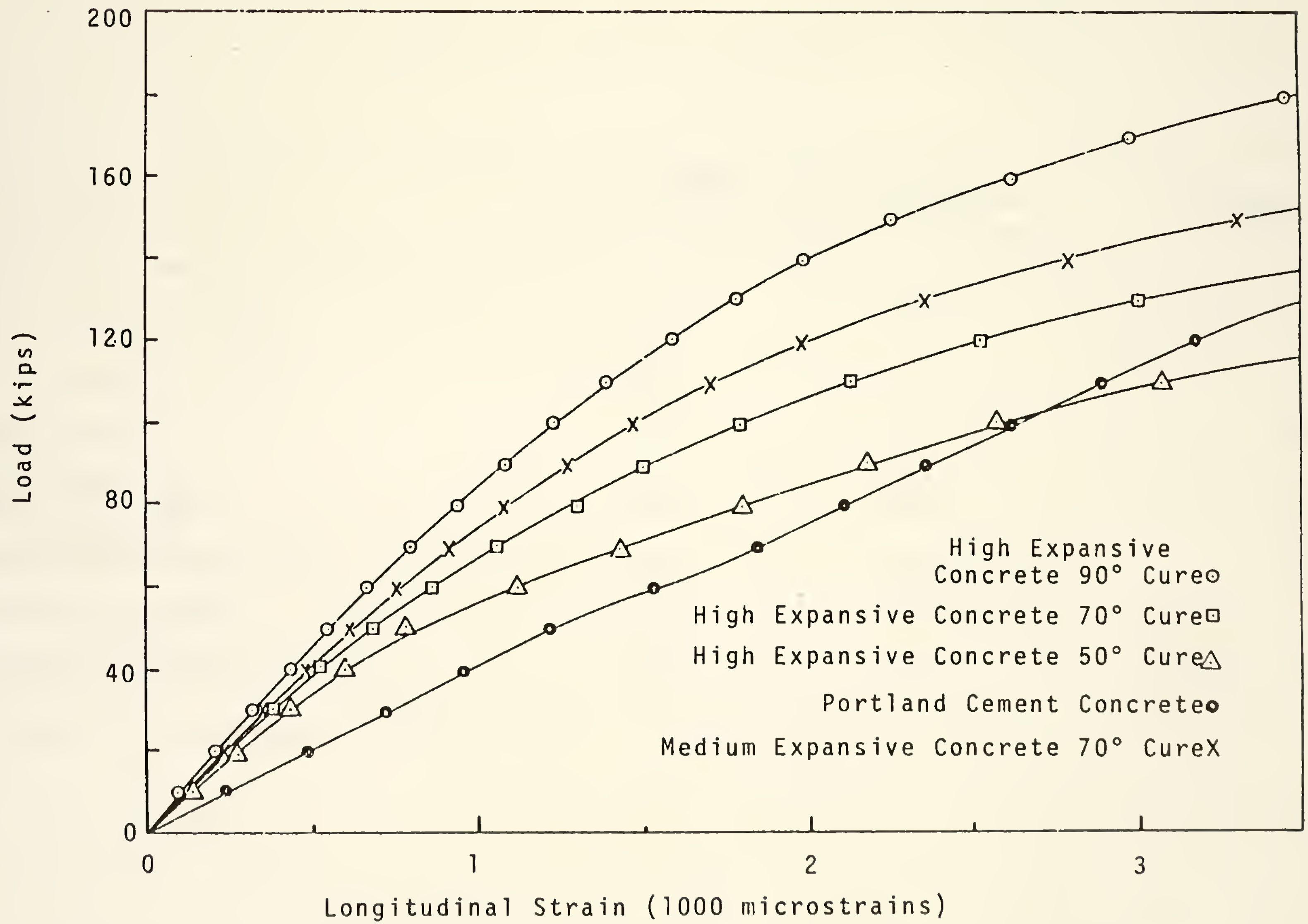

Figure 13-18. Load Strain Relationships for Composite Tubular Columns at the Working Levels.

Table 13-3 Column Load Responses.

Column Group	Response Under 60 kips		Ultimate Load (kips)
	Strain ($\mu\varepsilon$)	Relative Stiffness	
High Expansive, 50°F	1119	0.77	approx. 160
High Expansive, 70°F	863	1	200
High Expansive, 90°F	673	1.28	260
High Expansive, 50° - 70° - 90°F	692	1.25	195
Medium Expansive, 70°F	759	1.14	226
Average Portland	1527	0.57	approx. 210
Strongest Portland	1040	0.83	approx. 210

could be somewhat misleading. Carefully looking at the average results, the portland cement columns performed miserably, far worse than expected. As discussed earlier, the variation and reduction in stiffness was probably due to the unpredictable internal lateral tension forces developed by bond because of the concrete shrinkage and the different poisson's ratios for the steel and concrete. The transverse tensile forces weakened the axial compressive resistance of both materials.

The two stiffest normal columns reacted to the loads as expected. Their relative stiffnesses were both 0.83 at 60 kips. They were only 17% less stiff than the 70°F high expansive columns and slightly more stiff than the 50°F cure high expansive group.

CHAPTER 14

FINDINGS

The following findings are pertinent only to the tests and materials used in this investigation.

1. The total expansion of expansive concrete is quite sensitive to the mix proportions.
2. The rate and extent of expansion can be increased markedly by increasing the curing temperature. Expansion is very slow at cool temperatures.
3. Under triaxial confinement, slowly expanding concrete expands primarily in the direction of least restraint. Expansion is distributed so that the confining stress is constant in all directions.
4. When the rate of growth is very large, the expansion tends to become uniform in all directions regardless of the degree of restraint.
5. The expansion process when totally confined covers a period of up to thirty-five days.
6. Composite tubular steel columns cast with concrete are very ductile.
7. Composite tubular steel columns filled with concrete can be stiffened and strengthened considerably when filled with expansive concrete rather than portland

cement concrete. As far as strength is concerned, no benefit is found unless the expansive concrete is cured at temperatures above 70°F. Yet expansive columns cured at 50°F are still stiffer than the normal composite columns. In general, higher curing temperatures yield columns of greater stiffness and strength.

8. Expansion, self-stressing, stiffening and gain in strength can be induced by exposing an expansive concrete column to higher curing temperatures. Yet the expansion which can be achieved is dependent upon the curing history.
9. The time of set of expansive concrete is very short. Flash set occurs unless precaution is taken to cool the mix ingredients.
10. Composite tubular steel columns filled with expansive concrete represent a good means for improving the stiffness, strength and ductility of each constitutive material.

CONCLUSIONS
FOR PART III

In conclusion, it has been found through this investigation that expansive (self-stressing) concrete can be used to provide significant structural improvement to tubular steel columns filled with this material. The existence and amount of improvement structurally is quite sensitive to the curing temperature. Strength is best achieved when the columns cure at higher temperatures. As long as the average curing temperature during the first month is greater than 70°F, then expansive concrete placed in tubular columns will be much stronger and stiffer than the same column cast with portland cement concrete.

The mechanical performance requires the prerequisite of workability for casting consolidation. The expansive concrete will make this impossible by its flash set unless precautionary steps are made during mixing and casting.

With these conditions properly enforced, the tubular steel column filled with either high expansive or medium expansive concrete will provide greater stiffness, strength and ductility than the column cast with portland cement concrete.

LIST OF REFERENCES
FOR PART III

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2. "Tech Tips from Micro-Measurements, New Methods and Helpful Techniques in Bonded Gage Technology," Micro-Measurements Div., Romulus, Michigan.

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PART IV

PROJECT SUMMARY AND RECOMMENDATIONS

The purpose of this project was to gain a better understanding of the behavior of self-stressing reinforced concrete structures. In order to accomplish this it was necessary to develop an expansive cement source, and it was most feasible to prepare such a material in the laboratory insuring its freshness and reproducibility. The cement developed was sufficient to accomplish this purpose.

A second objective was to determine the economic savings possible with the effective use of expansive cement under practical construction conditions. For this objective, the expansive cement was found to be insufficient to allow potential savings to be accomplished.

This investigation considered both the properties of the expansive concrete and the performance of the self stressed units. Since the characteristics of the expansive concrete were somewhat unique and did have a limiting influence on the structural performance they were first discussed.

<u>Analysis of the mechanical properties and concrete performance.</u>

Foremost among these properties was that of expansive potential. This was found to be controllable and for purposes of this work, the amount of expansion, or expansive force assuming it was restrained was most adequate. However, the achievement of a large expansion potential was done with a sacrifice to compressive strength and modulus of elasticity. This relationship was hypothesized to be minimized by utilizing three dimensional prestressing of the concrete. This work is described in Chapter 5; however, it should be emphasized that the strength of Type I portland cement concrete, 6,670 psi, was greater than the greatest strength achieved in the triaxially reinforced expansive concrete 6310 psi. The latter occuring with a developed self stress of 246 psi. Likewise, the modulus of elasticity was greater for conventional concrete 4.36×10^6 psi compared to 4.18×10^6 psi. The result was that improved mechanical properties by three dimensional prestressing were more than negated by losses in the capacity of the expansive concrete to withstand loads. Were this not true, or if to a lessor extent, superior mechanical properties might be realized. One must realize that the expansive cement concrete giving the above values had a gypsum content of 15 percent in addition to that already present in the portland cement.

The concrete has several other undesirable characteristics in addition to the lower compressive strength and modulus of elasticity. In the plastic state the consistency and time of set was found to be extremely susceptible to the temperature of the mix. Figure 8-22 thru 8-26 show this behavior. It was essential that expansive concrete be produced at a cool (less than $20^{o}C$) temperature for a rapid temperature rise was detected. Cool water, $10^{o}C$, was sufficient for the laboratory work, however in field conditions, extensive use of artificial cooling or ice would be required to cool ingredients if they were in excess of $20^{o}C$.

The freeze thaw durability of the air entrained expansive concrete was found to be unsatisfactory. Hence, such concrete should not be used in a situation where it might be both moist and exposed to freezing. Possibly other procedures would reduce the problem, but it was so severe that one must have serious reservations about self stressing cement concrete in freeze-thaw exposures such as pavements or bridge decks.

Determination of Optimum Reinforcement Ratio and Effect of Eccentric Restraint.

Determination of an optimum longitudinal/lateral steel ratio was an objective of this investigation. As discussed in Chapter 3, the existence could be reasoned and the limited results from the literature indicated its existence. However, this investigation found an increase whenever the lateral steel was increased. This was true up to what was considered to be the maximum practical value, 0.84%, of lateral steel.

Another objective was to determine the effect of eccentric restraint. This is the topic of Chapter 4. Eccentric reinforcement does cause non-uniform stress distributions to develop, in fact, a shortening on the surface nearest the reinforcement was noted. This must be recognized and taken into account in self stressed concrete applications.

Behavior of Typical Structural Members

To analyze the behavior of typical structural members, was a major goal of the investigation. Small beams were evaluated and reported in the interim report and are the subject of Chapters 3 and 4 of this report. An advantage of prestressing is a reduction in the amount of steel required. For the simple beams of this investigation, an analysis based upon equal deflection showed that a 20 percent reduction in steel quantities was possible.

At the conclusion of the work on small beams an evaluation of the most promising structural applications was made and columns were selected for further investigation. Reinforced concrete columns are the subject of Part II (Chapters 6 thru 8) of this report and steel tube columns filled with concrete are the subject of Part III, (Chapters 11 thru 14).

The most promising practical application of self stressing concrete appears to be in the steel column filled with expansive concrete. Two reasons exist for this promise: first, the material is protected from aggressive outside conditions such as freezing and thawing and secondly, the stiffness of the columns are appreciably increased. This is shown in Table 13-3. A major advantage of such columns is their capability to take extreme deformations without a large loss in their load bearing capacity. For siesmic exposure, for instance, they would be superior to conventional reinforced or concrete filled steel pipe columns.

Such application would result in a more resistant structure, however, its concrete construction procedure would require several special requirements. Concrete must be placed at a temperature below 20°C to insure a consistency which will allow good compaction, and for maximum expansion it must subsequently be cured at warm temperatures, preferably above 30°C. Such requirements obviously restrict the practical use of self stressing cement.

Economic savings under practical construction conditions do not appear obtainable with the expansive cement used. If an expansive cement which did not have the sensitivity to temperature in both placement and curing existed, it would be more feasible to utilize the material. Also, if such an expansive cement would produce a concrete with strength comparable to a conventional portland cement, the economic savings in materials, primarily steel, would be appreciable. However, the Type M cement utilized in this study does not offer economic savings when considered for practical application.

Recommendations for future work

The first requirement for future development of expansive, "self stressing" cement concrete should be the development and verification of an expansive cement which will exhibit high compressive strength and modulus of elasticity. Furthermore, for cast-in-place construction it should be one in which the performance of the concrete is not so susceptible to temperature at the time of placement and during the curing process.

If such a cement were developed, further structure evaluations of self-stressed concrete would be justified; however, the performance in wet, freezing and thawing exposures, might still be unsatisfactory.

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