

Investigation of Minimum Longitudinal Reinforcing Requirements for Concrete
Columns

By

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Provisions in the ACI 318 Building Code (Ref. 1) for minimum longitudinal reinforcement in columns are based on conclusions from testing programs that were performed several decades ago. The tests employed materials with strengths that are no longer consistent with today's materials, and so it is uncertain if the provisions are appropriate for use today. The presence of the minimum requirement for longitudinal steel was to ensure that "passive yielding" of the reinforcement would not occur. A limit of 1% limit was first published in a committee document by the American Concrete Institute - American Society of Civil Engineers (ACI-ASCE) Joint Committee 105 in 1933.

This investigation was designed to identify the minimum longitudinal reinforcement ratio for concrete columns that would preclude passive yielding of the reinforcement. It was anticipated that a lower ratio would be suitable in certain applications. This experimental program and related analyses were designed to determine the possibility of such a reduction.

Several conclusions were drawn from the data produced by this investigation. Many factors were found to affect the amount of steel needed to prevent passive yielding. Strain response predictions made using the ACI 209 method agreed reasonably well with measured data but tended to under-predict strains. In general, it appears that it may be acceptable to reduce the minimum reinforcement requirement for certain conditions, but in general, it cannot be reduced.

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Chapter 1. INTRODUCTION

1.1 Introduction

Provisions in the ACI 318 Building Code (Ref. 1) for minimum longitudinal reinforcement in columns are based on conclusions from testing programs that were performed several decades ago. The tests employed materials with strengths that are no longer consistent with today's materials, and so it is uncertain if the provisions are appropriate for use today. Section 10.9.1 of the ACI 318 code requires a minimum amount of longitudinal steel equal to 0.01 times the gross cross-section area of the column. This provision is based partly on creep and shrinkage tests performed in the 1920's and 1930's (Ref. 2-10) and has been present in the code since 1936.

The presence of the minimum requirement for longitudinal steel was to ensure that "passive yielding" of the reinforcement would not occur. Passive yielding occurs when load is transferred from the concrete to the reinforcing steel as the concrete creeps and shrinks. The 1% limit was first published in a committee document by the American Concrete Institute - American Society of Civil Engineers (ACI-ASCE) Joint Committee 105 in 1933 (Ref. 8).

The tests performed in the 1920's and 1930's used materials that had different properties than the materials used in practice today, 60 years later. During the 1920's and 1930's the ultimate strength of concrete ranged between 2,000 psi and 5,000 psi and the yield strength of reinforcing steel was 39 ksi to 54 ksi. Today the concrete used in structures usually has an ultimate strength of at least 4,000 psi and strengths near 10,000 psi are not uncommon. The nominal yield strength of typical reinforcing steel has also increased; it is now 60 ksi.

ACI-ASCE Committee 441 - Concrete Columns (Ref. 11) recently reported that it may now be appropriate to lower the minimum reinforcement requirement to 0.5%. Before such a change is made, verification of the analysis performed by the committee would be wise. An experimental study using current materials was executed to gather data to verify the analysis.

The driving force behind the proposed change to the minimum reinforcement requirement is the substantial savings that could be attained by using less steel in columns. Since the majority of bridge piers designed in Texas contain the minimum required longitudinal reinforcement, reduction in the minimum required longitudinal reinforcement would result in reduced materials costs. Additional benefits from such a change are reduced congestion in the steel reinforcing cages, which would result in savings in construction costs.

1.2 Objective and Scope of This Investigation

This investigation was designed to identify the minimum longitudinal reinforcement ratio for concrete columns that would preclude passive yielding of the reinforcement. Currently the minimum requirement for longitudinal steel is 1.0% of the gross cross-sectional area (ACI 318-95 Section 10.9.1). It was anticipated that a lower ratio would be suitable in certain applications.

The applicable section from the ACI 318-95 code is as follows:

10.9 - Limits for reinforcement of compression members

10.9.1 - Area of longitudinal reinforcement for non-composite compression members shall not be less than 0.01 nor more than 0.08 times gross area A_g of section.

An experimental program that incorporated column specimens with variable concrete strengths, reinforcement ratios, and loading conditions is described in this thesis. In addition to a testing program, analytical results consistent with a method recommended by ACI Committee 209 were compared with the experimental results to verify the Committee 209 recommendations.

a) Experimental Component

The experimental program (discussed in detail in Chapter 3 of this thesis) consisted of 38 reinforced concrete columns. Twenty-four of the columns were subjected

to sustained load while the remaining 14 were unloaded control specimens. The load applied to each column was $0.40 \cdot f_c' \cdot A_g$. This load was used because it corresponds approximately to the maximum service load possible, based on Section 10.3.5.2 of the ACI 318-95 code for a tied column and using $A_g \cdot f_c'$ as the column strength.

Column specimens, which were cast in EZ Pour cardboard tubes, had a 48 inch length and 8 inch diameter. Specimens remained in the forms for five days and were loaded 14 to 28 days after casting. Load was maintained on each column with heavy coil springs. Several strain readings were collected from each specimen over the course of the long-term testing program using a mechanical Demec gage and electrical-resistance strain gages. Humidity in the enclosures containing the specimens was reduced to the extent made possible by the research budget. Humidity and temperature were monitored throughout the testing program.

Several variables were investigated. Two nominal concrete strengths, 4,000 psi and 8,000 psi, were investigated. Four reinforcement ratios were implemented. The ratios used were 0.0000, 0.0036, 0.0054, and 0.0072. The nominal loading conditions for the specimens were concentric loading or an eccentric loading of 0.10 times the column diameter.

The properties of the materials used to construct the specimens were determined. Concrete strengths were measured using several concrete cylinders cast from the same concrete batches as the specimens. The cylinders were also used to determine the moduli of elasticity for the various concrete mixes. The concrete properties were determined at 7, 14, 28, and 49 or 56 days after casting. Both the yield and ultimate strengths of the steel reinforcing bars were also determined.

b) Analytical Component

ACI Committee 209 developed recommendations for calculating time-dependent strains in concrete structures (Ref. 12). Their report was the basis for the strains determined analytically in Chapter 5 of this thesis. One type of viscoelastic behavior that concrete exhibits is deformation that occurs in a saturated environment. Another

viscoelastic behavior exhibited is drying creep, which is the additional deformation observed in columns in non-saturated environments.

ACI Committee 209's recommendations identified several conditions that increase creep in concrete. Increased water-cement ratio, highly permeable aggregate, loading at a young age, increased ambient temperature, reduced ambient humidity, and reduced volume-to-surface area were all found to increase the creep experienced by concrete. These factors, with the exception of early loading, were also found to cause increased shrinkage.

ACI report 209R-86, entitled "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures" presents a method for determining creep and shrinkage deformations in unreinforced concrete. The values predicted with this method can then be modified for use with reinforced concrete.

It was suggested in report ACI 92-S26, entitled "Longitudinal Steel Limits for Concrete Columns" by C. H. Lin and R. W. Furlong (Ref. 13), that the minimum reinforcement limit can be reduced. One rationale used to arrive at this conclusion was a calculation of the minimum amount of longitudinal reinforcement needed to prevent passive yielding in concrete columns. As a column undergoes creep and shrinkage, compressive forces in the column cross section are transferred to the reinforcing steel. If a column creeps too much the steel will yield. The calculations suggested in report ACI 92-S26 were used to develop Table 1.1. This table shows the minimum percentage of steel required to prevent passive yielding in columns with various material properties and loading conditions based on standard median creep (ν_u) and shrinkage (ϵ_{sh}) coefficients. The ratio L/D is the live-to-dead load ratio.

TABLE 1.1 Minimum % of longitudinal reinforcement (for $\nu_u = 2.35$ and $(\epsilon_{sh})_u = 800 \times 10^{-6}$)

f'c, psi	fy, psi	L/D							
		0.0	0.25	0.5	1.0	1.5	2.0	2.5	3.0
3,000	60,000	0	0	0	0	0	0	0	0
4,000	60,000	0	0	0	0	0	0	0	0
6,000	60,000	1.70	0.05	0	0	0	0	0	0
8,000	60,000	3.40	1.35	0.06	0	0	0	0	0
10,000	60,000	5.10	2.67	1.15	0	0	0	0	0

Table 1.1 shows that for lower concrete strengths the minimum amount of steel is less than the present required 1% ratio. In addition, as the live load-to-dead load ratio increases, less steel is needed to prevent passive yielding.

A second table was created based on the calculations suggested in report ACI 92-S26. Table 1.2 shows the results of the same calculations if upper bound values are used for the creep and shrinkage coefficients.

TABLE 1.2 Minimum % of longitudinal reinforcement (for $\nu_u = 4.15$ and $(\epsilon_{sh})_u = 1070 \times 10^{-6}$)

f'c, psi	Fy, psi	L/D							
		0.0	0.25	0.5	1.0	1.5	2.0	2.5	3.0
3,000	60,000	2.55	1.28	0.53	0	0	0	0	0
4,000	60,000	3.75	2.21	1.27	0.19	0	0	0	0
6,000	60,000	6.04	4.04	2.78	1.28	0.41	0	0	0
8,000	60,000	8.19	5.81	4.29	2.44	1.34	0.60	0.07	0
10,000	60,000	10.22	7.53	5.78	3.63	2.32	1.44	0.79	0.30

Table 1.2 demonstrates that a reduction in reinforcement is appropriate only for L/D values of 0.5 and larger. The use of the upper-bound values significantly alters the

amount of steel needed to prevent passive yielding. Both of these tables were created assuming concentrically-loaded columns.

Chapter 2. REVIEW OF LITERATURE ON CREEP AND SHRINKAGE IN REINFORCED CONCRETE COLUMNS

2.1 Introduction

This chapter summarizes the pertinent prior investigations of concrete column behavior. The investigations summarized were conducted during the first half of the 20th century, starting with work performed in the early 1930's at the University of California at Berkeley. The more significant work performed at the University of Illinois and Lehigh University during the 1930's is reviewed.

2.2 Davis and Davis (Ref. 2) – University of California at Berkeley (1931)

This investigation by Davis and Davis is the earliest to study the nature of time-dependent effects on reinforced concrete columns. The fifth phase of the investigation is the most relevant to this thesis. The purpose of the fifth phase was to examine the effect of placing reinforcement in concrete columns on shrinkage and creep. Also examined was the effect of shrinkage and creep on the stresses in the reinforcement.

The investigation involved testing eight columns that had a diameter of 10 inches and a height of 20 inches. Half of the specimens were used as a control group and were left unloaded. Of the remaining four specimens, two had no reinforcement while the other two had 1.33 percent (by volume) spiral reinforcement and 1.9 percent longitudinal reinforcement. These four specimens were axially loaded to a stress of 800 psi. This stress was maintained with the use of car springs, with no compensation for unloading due to time-dependent deformations. Prior to loading, the columns were stored in 100% humidity for 50 days and then in ambient conditions for 10 days. For the 18-month duration of the test, the columns were kept in a humidity-controlled enclosure. The humidity was kept at 50 percent (+/- 1 percent) and the temperature was held at a constant 70 degrees Fahrenheit (+/- 1 degree F) inside the enclosure.

After casting the specimens, stainless steel inserts were threaded into brass plugs that were cast in the specimens. Three such inserts were placed in each specimen as well

as the gage holes in the longitudinal reinforcement. Strain was measured using a ten-inch mechanical gage.

Davis and Davis reached several conclusions. The significant conclusions are as follows:

1. Combined creep and shrinkage after 18 months for columns without reinforcement was six times the immediate deformation due to initial loading of the specimens.
2. Combined creep and shrinkage after 18 months for columns with reinforcement was four times the immediate deformation due to initial loading of the specimens.
3. Total stress in the longitudinal steel was 30,300 psi. Of this amount, 5,700 psi was due to instantaneous deformation, 11,400 psi was due to creep, and 13,200 psi was due to shrinkage.
4. Uniformly distributing the load not carried by the longitudinal steel over the remaining concrete area, stress in the concrete reduced to 300 psi from 775 psi over 18 months.
5. Conditions that produce large shrinkage deformations, and loading patterns that involve mostly dead load, necessitate the use of the yield strength of the reinforcing steel as a design consideration.

2.3 Richard and Staehle (Ref. 3) - 2nd Progress report - University of Illinois (1931)

This report dealt with a portion of a large investigation of concrete columns, which was performed by the University of Illinois and Lehigh University. The third series of these investigations involved the behavior of concrete under sustained loading.

This series of tests included 108 columns that were 8.25 inches in diameter. Sixty of the specimens were loaded for 20 weeks. The nominal concrete strengths and reinforcement ratios were varied. Forty-eight specimens were left unloaded.

The longitudinal reinforcement ratios used were 1.5, 4, and 6 percent, and the volumetric, spiral reinforcement ratios used were 1.24 and 2 percent. The nominal concrete strengths were 2000, 3500, and 5000 psi, and the actual strengths measured 56 days after casting were slightly higher.

Load was applied to the columns by using hand-tightened nuts to compress railroad car springs. No eccentricities were noted for the loads on the columns. The loads were adjusted after three months.

The specimens were cured in a high-humidity room for 56 days. After the initial 56 day period the specimens were loaded (or left unloaded) and were stored in high-humidity conditions or moved into an uncontrolled lab environment. The high-humidity room was kept at 70 degrees Fahrenheit and 100 percent relative humidity. The lab conditions varied between 70 and 90 degrees Fahrenheit and 40 and 90 percent relative humidity.

Strain readings were made after the initial casting and throughout the loading period. During the loading phase, readings were taken at 1, 3, 4, 7, 14, and 28 days after loading and every 28 days thereafter. Several individuals were employed to take the readings using a 10-inch Whittemore gage.

The authors ignored possible effects of variations in temperature.

After the 20-week loading period the authors made the following conclusions:

1. Most of the creep and shrinkage occurred during the first five months.
2. Variations in spiral reinforcement did not effect the creep and shrinkage.
3. The increase in stress in the steel was generally between 6,000 and 14,000 psi with a maximum of 14,800 psi.
4. Specimens with the least longitudinal steel (1.5 percent) had the greatest increase in steel stresses. The specimens with the greatest amount of steel (6.0 percent) had the least change in steel stresses.
5. After five months only one specimen experienced steel stresses that were one half the yield stress, and thus it was concluded that the yield point was not reached in any of the specimens.
6. Lesser amounts of creep and shrinkage were observed in specimens loaded in the high-humidity conditions as opposed to those in the lab environment.
7. The modulus of elasticity significantly increased with time for those specimens that were placed in the high-humidity environment.

2.4 Slater and Lyse (Ref. 4) – 2nd Progress report – Lehigh University (1932)

This report was the counterpart of the University of Illinois study. It too involved the study of concrete columns under sustained load. The specimens had similar dimensions to those at the University of Illinois

Storage conditions were similar to those at the University of Illinois. The high-humidity environment was the same as at Illinois, while the lab conditions varied from 60 to 95 degrees Fahrenheit and the humidity was unreported.

Load was applied using a screw-type loading machine, then was maintained with springs. The specimens were overloaded to compensate for the load loss due to elongation of the rods after removal from the loading machine.

After three months, load was adjusted on those specimens stored in the lab environment. Those specimens in the high humidity environment did not need adjustment. The average load loss in the specimens that needed adjustment was seven percent.

The same three concrete strengths of 2000, 3500, and 5000 psi were investigated. The measured strengths at 56 days were only slightly higher than the nominal strengths.

The specimens were instrumented with 40 gages each. Twenty of the gages were used to measure strain in the steel, and 20 were used to measure strain in the concrete.

The significant conclusions that the authors made were the following:

1. From two to four weeks after the application of load, a large increase in strain was observed. After this period, the rate of increase became smaller and more constant.
2. Specimens in the high-humidity environment experienced much smaller deformations.
3. Concrete strength did not affect the rate of increase in the strains. This was attributed to the higher-strength specimens having larger loads placed on them.
4. Spiral reinforcement did not affect creep.
5. Columns with lower longitudinal reinforcement ratios exhibited the greatest amount of creep.
6. Higher-strength concrete produced larger strains due to shrinkage.

7. Two columns with reinforcement ratios of 6 percent had small tensile stresses and thus, it was assumed that the steel was carrying the entire load.
8. The highest stresses were measured in the specimens that had a 1.5 percent reinforcement ratio. The measured stress was 42,660 psi, which is very close to the yield stress of 49,500 psi.

2.5 Richard and Staehle (Ref. 5) – 4th Progress report - University of Illinois (1932)

A continuation of the investigation discussed in their second progress report, this fourth progress report involved data from a lengthier loading period. Expanding on the original 20 weeks, this report involved data collected over 52 weeks. Also included were strengths and deformations at failure.

As discussed previously, the investigation involved 108 specimens that varied in reinforcement ratio. The storage conditions also varied between a high-humidity environment and a lab environment.

Load was adjusted periodically to account for load reduction due to strains in the specimens.

From the period where the last report ended (five months) to 12 months, strengths of the specimens increased and stresses in the steel stayed much the same. Cylinder strengths indicated that specimens stored in a high-humidity environment increased in strength by 30 percent while those in the lab environment increased 15 percent. Stress changes in the steel were small, and in some specimens they were negative. This may have been due to seasonal changes in humidity and temperature.

Steel stresses in the columns indicated that in most specimens the steel only reached half of yield. The largest stress observed was 30,800 psi but it was in a column designed in accordance with the New York City building code. The largest stress observed in a specimen designed in accordance with the ACI Code was 26,700 psi. The maximum deformations measured were three times that of the initial elastic deformation.

Concrete strength still did not appear to alter the rate of deformation. Although after 52 weeks the rate of deformation was very small, some specimens were observed to have sufficiently large rates to warrant further observation.

From this study several conclusions were made. They are as follows:

1. Steel stresses in the columns never reached those of yield (45,600 psi). The greatest stress attained was 30,800 psi in a column with 1.5 percent longitudinal reinforcement and with a nominal concrete strength of 3,500 psi.
2. Loaded columns had the same ultimate strength as unloaded columns.
3. The strength increase from two months to one year varied from 15 to 30 percent.

Also present in this progress report were results from Series 5 of the tests at the University of Illinois.

Series 5 was conducted to evaluate whether the specimens in Series 3 were accurate models of larger columns typically used in construction. Twenty columns with diameters that varied from 12, 20, and 28-inches and lengths that were 7.5 times that of the diameter were investigated. The nominal concrete strengths used were 2,000 and 3,500 psi, and the reinforcement ratios were 1.5 and 4 percent. From the study it was concluded that the small-scale models used in Series 3 were good models of construction sized columns with respect to strength.

2.6 Lyse and Kreidler (Ref. 6) – 4th Progress report – Lehigh University (1932)

The purpose of this report was to present additional findings from the investigation of columns under sustained loading. Presented were the results after 52 weeks of loading as well as results from tests to failure.

Of the 108 original specimens, twelve were retained to investigate further loading effects. These columns were loaded for an additional 52 weeks.

The strength of specimens stored in the high-humidity conditions increased in strength by 14 percent over 52 weeks. The modulus of elasticity also increased similarly. For those columns stored in the lab environment, strength gain was marginal.

Adjustments were made to the load to compensate for column deformations. The permanent set of the springs was found to be 4 percent.

During unloading, the columns stored in lab conditions developed transverse cracks, while those stored in the high-humidity environment did not experience such cracking.

After 52 weeks there appeared to be a correlation between nominal strength and creep. The higher strength concrete experienced larger strains due to creep. The higher strength concrete also exhibited larger strains due to shrinkage. Shrinkage increased in the drier environment, and decreased with greater amounts of reinforcing steel.

The average stress found in the steel for the 2,000 psi specimens was 30,000 psi, while the average steel stress in the 5,000 psi columns was 37,000 psi. Columns with 1.5 percent longitudinal reinforcement had much higher steel stresses than those with 6 percent reinforcement. One of the 2,000 psi specimens with 1.5 percent reinforcement had all load carried by the steel after 52 weeks.

It was found that sustained loading did not affect strengths of columns. When tested to failure, the columns that were under sustained loading had strengths that varied between 95 and 112 percent of the strength of comparable unloaded columns.

After the 52-week loading period, the authors made several conclusions:

1. Higher-strength concrete experienced larger strains due to creep.
2. Although columns with no spiral reinforcement exhibited larger creep strains, the difference between 1.2 and 2 percent spiral reinforcement was negligible.
3. Higher rates of creep were found in specimens with less longitudinal reinforcement.
4. Higher-strength concrete specimens experienced larger strains due to shrinkage.
5. Columns with less longitudinal steel experienced greater amounts of shrinkage.
6. No yield point could be seen on the load-deformation curve for any column.
7. Steel stresses in the columns with 1.5 percent reinforcement and stored in the lab environment increased from 6,000 psi to 37,000 psi. Under the same conditions but with 6 percent reinforcement, steel stresses increased from 16,000 psi to 30,000 psi. Columns stored in the high-humidity environment increased from 12,000 psi to 19,000 psi.
8. Ultimate strength was not effected by sustained loading.

2.7 Lyse (Ref. 7) – 5th Progress report – Lehigh University (1933)

In this report Lyse reported the results of several tests intended to determine the maximum load a reinforced concrete column can sustain indefinitely. This study involved 28 columns 8-1/4 inches in diameter and 60 inches long, and having 4 or 6 percent longitudinal reinforcement. The nominal concrete strength was 3,500 psi and the amount of spiral reinforcement was 0, 1.2, or 2 percent. At 56 days after casting, the columns were loaded with between 70 and 100 percent of their capacity.

To determine the capacity of each group of specimens, one of each set of three identical columns was loaded to failure. The remaining columns in each set were loaded with a percentage of the load required to fail the first column. Initial loads were placed with a testing machine, and the load was maintained by springs. Load adjustments were made by tightening bolts holding the springs in place.

Several significant conclusions were made.

1. Yield stress of the reinforcement can be maintained for strains much greater than the yield strain.
2. Ultimate strength of the columns was not effected by having longitudinal reinforcement strained beyond yield.
3. Columns are likely able to withstand 80 percent of their ultimate load indefinitely.
4. Columns with the lesser amount of spiral reinforcement were able to sustain the 80 percent load with fewer signs of distress.

2.8 Richart (Ref. 8) – Tentative Final Progress Report of Committee 105 (1933)

This report was a summary of the work carried out at the University of Illinois and Lehigh University.

The report presented a formula for the ultimate strength of reinforced columns that was applicable to concrete strengths between 2,000 psi and 8,000 psi. The formula was also limited to longitudinal reinforcement ratios between 1.5 and 6.0 percent, and steel with yield stress from 39,000 psi to 68,000 psi. In addition to this formula, formulas for the yield point of columns and ultimate strength of tied columns were presented.

The greatest increase in longitudinal steel stresses was 42,000 psi after five months of sustained loading. The average increase was 12,000 psi at the University of Illinois and 20,000 psi at Lehigh University. The increase in steel stresses during the period from 5 months to one year was 2,000 psi and the increase from one year to two years was also 2,000 psi.

Design formulas for maximum permissible load were presented. In addition, minimum longitudinal reinforcement ratios were presented. For columns with spiral reinforcement, the ratio was set at 0.01, and for tied columns it was set at 0.005. These ratios were given without any clear justification.

Alternate design formulas were given in a minority recommendation. This recommendation included the same reinforcement ratio limits.

2.9 Logeman, Mensch, DiStasio (Ref. 9) – Discussion of Committee 105 (1933) Report

This paper was concerned with the two sets of formulas presented in the report by Committee 105. Whether spiral reinforcement could be considered in design, and elastic versus plastic design was discussed. The lack of supporting evidence for the difference between the reinforcement ratio of 0.005 for tied columns and 0.010 for spirally-reinforced columns was questioned by DiStasio. He suggested that the effects of bending be investigated before design formulas are accepted.

2.10 Richart (Ref. 10) – Discussion of Committee 105 Report, Closure by Chairman, Committee 105 (1933)

The division among the committee members was again the focus. The majority discounted the contribution of spiral reinforcement since its benefits were not realized until columns had undergone large displacements.

The reinforcement ratios presented earlier were not discussed.

2.11 Conclusions

The most significant reports from previous tests were those documenting several investigations performed at the University of Illinois and Lehigh University. These investigations involved 126 columns that were loaded under constant axial stress for 52 weeks in either of two types of environments. From these tests many conclusions were reached.

The columns involved in the tests were loaded according to either the ACI Code or the New York City building code. The nominal strengths of concrete used were 2,000, 3,500, and 5,000 psi and the reinforcement ratios were 1.5, 4, and 6 percent. Yield strength of the reinforcing steel ranged between 46,600 and 51,100 psi.

The largest steel stress inferred from strain measurements was 37,000 psi, which was below the yield point. The initial steel stress in this specimen was 6,000 psi. The highest stresses occurred in specimens with reinforcement ratios of 1.5 percent that were stored in the dry air environment of the lab. Steel stresses in specimens with 6 percent reinforcement ratio increased from 16,000 to 30,000 psi.

Columns that were loaded for 52 weeks were found to have unchanged ultimate load capacities. Additional tests indicated that columns could sustain 80 percent of their ultimate load capacity indefinitely, however stability was a problem in some cases.

As a result of these investigations, Committee 105 created minimum limits for reinforcement ratios. For tied columns, the minimum reinforcement ratio was set to 0.005, and was 0.010 for spirally-reinforced columns. There was little rationalization for setting these specific limits.

Chapter 3. EXPERIMENTAL PROGRAM

3.1 Introduction

The experimental program implemented to provide data to reevaluate the minimum longitudinal reinforcement ratio requirement of 1 percent for columns is described in this chapter. The investigation involved 38 reinforced concrete columns 8 inches in diameter and 48 inches in height. Twenty-four of the columns were loaded with a constant axial load. The remaining 14 columns were left unloaded and were used as control specimens. Of the 24 loaded columns, four were eccentrically loaded. All but four unloaded columns were spirally reinforced. Also involved in this experimental program were tests to monitor material strengths.

To reevaluate the 1.0 % minimum reinforcement requirement, percentages of reinforcement and concrete strengths were varied. The environment in which the columns were stored was somewhat controlled and monitored closely. A dehumidifier operating constantly was used to keep the relative humidity generally between 30 and 60 percent. The temperature was uncontrolled and ranged between 50 and 110 degrees Fahrenheit. To reevaluate the reinforcement ratio it was necessary to load the columns for a length of time that would allow the rate of creep to approach nearly zero. This period was initially estimated to be nearly two years, but in actuality it was 15 to 18 months, depending on the specimens.

3.2 Column Details

The longitudinal reinforcement details, as well as column cross sections, are shown in Figures 3.1 and 3.2. The average circumference of the columns before loading was 25.25 inches, which yielded an average diameter of 8.04 inches. Based on this diameter, the average cross-sectional area was 50.8 square inches.

Number 2 deformed bars were used for longitudinal reinforcement in the columns. Depending on the reinforcement ratio desired, 0, 4, 6, or 8 bars were used in the specimens. Because the No. 2 bars have 0.046 square inches of cross sectional

area, the resulting longitudinal reinforcement ratios for the columns were 0.0000, 0.0036, 0.0054, and 0.0072.

Spiral reinforcement was made with number 9 annealed wire. By hand feeding the wire around a 6-inch diameter spinning tube, a spiral with 7-1/2 inch diameter and 2 inch pitch was created. The spirals were stored outside in a moist environment to allow a thin layer of corrosion to form.

The first two groups of columns that were loaded experienced some cracking at the ends. Because of this, the following two groups of columns were wrapped with fiber reinforced plastic at their ends (see Fig. 3.2). It was hoped that this would prevent cracks from forming and propagating during the lengthy loading period.

To describe each specimen, a special nomenclature was used. A digit representing the nominal concrete compressive strength in ksi follows a letter indicating the type of loading (concentric or eccentric or unloaded). Next are two digits to indicate the reinforcement ratio in hundredths of percent. If the specimen had no spiral reinforcement NS indicating no steel supplants these two digits. Lastly, a single digit was used to specify a particular specimen within a group of identical specimens. An example of this is the specimen named E4-72-2. It is an eccentrically-loaded, 4-ksi specimen with a reinforcement ratio of 0.0072, and is the second in a group of identical specimens.

The number of specimens examined with the various concrete strengths, reinforcement ratios, and loading conditions is presented in Table 3.1.

TABLE 3.1 Number of Specimens

Design concrete strength, psi (& load type)	8 bars spiral 0.0072	6 bars spiral 0.0054	4 bars spiral 0.0036	0 bars spiral 0.0000	0 bars no spiral 0.0000
8,000 (concentric)	3	3	3		1
8,000 (eccentric)	1		1		
8,000 (no load)	1	1	1	2	2
4,000 (concentric)	3	3	3		1
4,000 (eccentric)	1		1		
4,000 (no load)	1	1	1	2	2

The actual concrete strengths, reinforcement ratios, column end conditions, load eccentricity, casting dates, age at loading, and group number for all 38 specimens are listed in Table 3.2.

3.3 Reduced-Humidity Environment

The columns were stored in a reduced-humidity enclosure. Two enclosures were built using wood studs that were wrapped with a 6-mil-thick vapor barrier. These enclosures were built inside a metal-framed warehouse on the University of Texas's J. J. Pickle Research Campus. To reduce humidity, a dehumidifier that operated continuously was placed in each enclosure. Plan views of the enclosures and the specimens that they contained are presented in Figures 3.3 and 3.4.

On cold days an attempt was made to raise the temperature in the enclosures. Small space heaters were used at times in an attempt to keep the temperature above 50 degrees Fahrenheit.

Devices to record temperature and humidity were placed in the enclosures. On a regular basis the current temperature and humidity were recorded. In addition to this information the maximum and minimum temperature and humidity in the enclosures were recorded. Initially, readings were taken every two to three days. Towards the end of the loading period recordings were made when data from the specimens were gathered (approximately every two weeks). Temperature and humidity histories for the four groups of specimens are presented in Figures 3.5 through 3.8.

3.4 Materials

(a) Concrete

Two different nominal concrete strengths, 4,000 psi and 8,000 psi at 28 days, were employed.

A local ready-mix plant was the source for the concrete. Concrete was brought to the lab four separate times, once for each group of specimens. The

moisture content of the fine and coarse aggregates used in the concrete was not controlled and it was impossible to accurately estimate the values. To identify the proper mix proportions, test mixes with various slumps were used.

The coarse gravel had a 3/8 inch maximum size and consisted of river gravel. Both mixes used a retarder. At the batch plant a super-plasticizing admixture was used for the 8,000 psi mix but none was added to the 4,000 psi mix.

The ready-mix plant provided mix proportions for each of the four groups. These proportions are shown in Table 3.3.

TABLE 3.3 Concrete Mix Proportions

Group	Quantity Batched, cu. yd.	Sand, lbs.	Type II Cement, lbs.	Water, lbs.	Water added, lbs.	3/8" Rock, lbs.	Retarder, oz.	Super-plasticizer, oz.	Slump, in.
1	4.0	4160	2805	426	112	8000	86	416	7.5
2	4.0	4160	2775	730	128	7860	84	416	6.5
3	4.0	6680	1925	564	80	5200	57	-	6.0
4	4.0	6580	2005	572	144	5200	58	-	6.5

The mix for Group 2 did not seem to be identical to that of group 1; coarse aggregate for Group 2 was larger and more plentiful.

(b) Reinforcing Steel

Number 2 deformed reinforcing bars with a nominal diameter of 6 mm (2/8 in.) were used as the longitudinal reinforcement. The nominal cross-sectional area of the bar was found through liquid displacement procedures and was verified by weighing a known length of bar. The area was found to be 0.046 square inches.

(c) Fiberglass Reinforced Plastic

After loading Groups 1 and 2, longitudinal cracking at the ends of the columns was noted. To prevent this from also occurring in the specimens in Groups 3 and 4, six inches of the ends were wrapped with fiberglass reinforced plastic.

The wrapping was done by hand prior to loading. The wrap had similar properties to E-glass and was held in place by a thin resin layer.

3.5 Manufacture of Test Specimens

(a) Columns

(i) Formwork

To form the columns, cardboard tubes (EZ Pour) with an inside nominal diameter of 8 inches were used. Four-foot lengths were cut and the insides were coated to ease removal after casting. Reinforcement cages were assembled then placed inside the forms and held in place with plastic ties.

(ii) Casting

Each of the four groups of specimens were cast separately. Group 1 was cast on February 7, 1996, and Group 3 was cast on April 4, 1996. Finally, Groups 2 and 4 were cast on May 15, 1996.

The columns were cast in a vertical position on a level wooden platform. The formwork was secured to the platform during casting. Concrete placement was done with a long-handle scoop and a small mechanical vibrator. For each group, casting required approximately one hour and was performed inside a reduced-humidity enclosure.

(iii) Curing

Moisture loss was prevented by covering the ends of the columns with 6 mil vapor barrier. Three days after casting, the vapor barrier was removed and a 3/8 inch layer of hydro-stone was poured to level the top end of each column. Five days after casting, the cardboard tubes were removed and the columns were stored on the laboratory floor. Between the seventh and tenth day after casting, mechanical strain gage (Demec) points were set in the specimens.

(iv) Application of Fiber-Reinforced Plastic

A representative of Ershigs Inc. of Gatesville, Texas applied the fiber wraps to the ends of select specimens. The resin-impregnated material was wrapped five times around each column end and then was trimmed with a mat knife.

(b) Cylinders

For each group of specimens, twelve 6 x 12 inch cylinders and eighteen 4 x 8 inch cylinders were cast. Compaction was done in accordance to ASTM standards, and cylinders were sealed with plastic caps after casting. The cylinders were cast outside the enclosure, then brought inside the enclosure three days after casting. The caps were removed when the vapor barrier was removed from the columns, and the cylinder molds were removed when the cardboard forms were removed from the columns.

3.6 Testing Appurtenances

(a) Columns

(i) Test Frames

For the column creep tests, 24 loading frames were built. Figure 3.9 is a schematic drawing of a frame.

The legs of the testing frames were made from 3/16 inch thick steel tubes. Load was maintained using triple-coil springs that deflected 1-1/2 inches under 20 kips of sustained load. Eight springs were used to maintain the 162 kip load, and four springs were needed to maintain the 81 kip load. Deformations of the springs were monitored using a metal scale with an accuracy of 1/64-inch.

Four Dywidag bars were used on all the setups. The 4,000 psi specimens used 5/8 inch diameter bars and the 8,000 psi specimens used 1 inch diameter bars. The bars extended from approximately three inches above the 3 inch thick steel top plate down to 9 inches below the bottom plate. To apply load to each specimen, a hydraulic ram was placed on a 3 inch steel plate which was positioned beneath each

frame and attached to the Dywidag bars using four coupling sleeves. Once load was applied to a specimen, nuts beneath the steel plate holding the triple coil springs were snug tightened to maintain the spring deflections and thus, the load on the specimen. Using this setup, several specimens could be loaded each day.

Groups 1 and 3 had pinned-end conditions. Two 1-1/2 inch steel plates were separated by a 1-1/4 inch diameter steel rod which fit into depressions in the steel plates. This arrangement was used on both the top and bottom of the specimens. Installing these pins proved to be quite difficult because it was hard to plumb the alignment of the two pins with the imperfections in the columns and loading system. Because of these difficulties the concentrically-loaded specimens in Groups 2 and 4 were not loaded using these pins. Instead, some specimens had neoprene pads placed between the top and bottom plates and the specimens. This approach also contained inherent problems because the pads tended to "walk " on the surface of the bottom plate during loading, which resulted in eccentricity of the applied load. As a result, only two of the specimens had neoprene pads. The remainder were loaded without neoprene pads. The eccentrically-loaded specimens used the pins and had their top plates braced against lateral movement.

(ii) Strain Measurements

Both mechanical and electrical strain measurements were made for all the specimens. The mechanical measurements were made using Demec points set into the specimens. The electrical measurements employed electrical resistance strain gages.

Each specimen had four pairs of Demec points. The pairs were oriented vertically on the columns (parallel to the longitudinal axis), as shown in Figure 3.10, and placed 20 degrees off the East-West axis of the columns as shown in Figure 3.11. The points were 1-inch metal H.I.T. anchors manufactured by Hilti and were placed 400 mm apart. The anchors were placed in drilled holes 7 to 10 days after casting using an epoxy.

The mechanical Demec gages were read approximately every other day after they were installed for approximately six weeks. At that time, readings were reduced to approximately once every week.

Each specimen had several longitudinal reinforcing bars instrumented with electrical resistance strain gages with a resistance of 350 ohms. Typically four or six gages were used in each specimen. The gages were placed 14 inches from the midheight of the columns and were staggered above and below midheight, as shown in Figure 3.12, to reduce the loss of cross-sectional area. To identify individual gages the numbering scheme shown in Figure 3.13 was devised.

In addition to these electrical gages, each specimen also had a "floating" electrical gage placed 8 inches from its bottom. The floating gages were effectively 8-inch strain gages. The location of these gages are shown in Figures 3.12 and 3.13.

The electrical gages were zeroed 20 minutes after the concrete was placed in the forms. Readings were then taken once every three days for the first four weeks. After the first four weeks readings were taken once a week.

(iii) Testing Procedure

Each loaded specimen was subjected to $0.40 \cdot A_g \cdot f'_c$ of axial load, where A_g is the gross cross-sectional area of the column and f'_c is the nominal compressive strength of the concrete. The exception to this loading was specimen E8-36 which was loaded with $0.30 \cdot A_g \cdot f'_c$. The load for Specimen E8-36 was reduced due to noticeable cracking on the compression side of the column. The resulting loads were 81.2 kips for the 4,000 psi specimens and 162 kips for the 8,000 psi specimens. The eccentric columns had an eccentricity of 0.80 inches which was equivalent to the code minimum of 10% of the nominal column diameter.

Load was applied using a 300 kip-capacity hydraulic ram. Once the load was applied, Dywidag nuts were hand tightened to secure the spring deformations. Pressure in the ram was monitored using a gage accurate to 200 psi. A small additional load was applied to account for seating of the nuts on the Dywidag bars.

The exact day on which each specimen was loaded is shown in Table 3.2, and was generally between 14 and 28 days after casting.

(b) Concrete Cylinders

Small (4 x 8 in.) and large (6 x 12 in.) cylinders were tested at 7, 14, 21, 28, 42, and 56 days. A minimum of two cylinders were tested in compression on each occasion for each specimen group. Both load and deflection data were recorded for each cylinder at several stress levels. These results were then averaged and used to calculate an ultimate strength and a modulus of elasticity. Strength and modulus tests were conducted in accordance with ASTM C39-61(5) and ASTM C469-94, respectively.

(c) Reinforcing Steel

Four tensile tests were conducted on the No. 2 bars using a 60-kip capacity Tinius-Olson universal test machine. The yield and ultimate strength of the No. 2 bars were determined from these tests.

Chapter 4. EXPERIMENTAL TEST RESULTS

4.1. Introduction

The test results for individual column specimens are shown in Figures 4.1 through 4.75. These results are based on data collected from the initiation of creep tests through July 2, 1997. To enable convenient comparisons, test results from individual concentrically-loaded specimens are grouped together in Figures 4.76 through 4.83.

This thesis will only consider data collected through July 2, 1997. The rate of time-dependent strain increase has dropped to such a low level that temperature-related changes in strain now mask the time-dependent changes. The very low rate of strain increase is evident in many of the strain-vs.-time plots shown in Figures 4.1 through 4.47.

The age of the specimens on July 2, 1997 was as follows:

<u>Group</u>	<u>Casting date</u>	<u>Age of specimens at last reading</u>
1	Feb. 7, 1996	511 days
2	May 15, 1996	413 days
3	April 4, 1996	454 days
4	May 15, 1996	413 days

The strain results for all column specimens have been adjusted for temperature effects. This was done because varying temperatures in the enclosures resulted in measurable changes in column strains. The coefficient of thermal expansion used was 6.5 micro-strain per degree Fahrenheit. The largest temperature variation for either enclosure was 71 degrees. This was equivalent to a strain differential of 460 micro-strain.

Temperature inside each enclosure was typically measured at the time strain readings were made in the specimens. It is noted that the temperature readings used are

ambient temperature readings as opposed to temperature readings from inside the concrete specimens. In the event that no temperature reading was available, the author's judgement was used to provide a reasonable temperature for the day and time of the reading in question. This judgement was based on readings taken prior to and after the missing temperature data and other experience with temperatures in the enclosures.

For review, the nomenclature used to designate each column specimen is as follows:

For example, for a specimen designated, **C8-36-1**,

C = load condition (C = concentric, E = eccentric, U = unloaded)

8 = design strength at 28 days in ksi (4 or 8)

36 = longitudinal reinforcement ratio in hundredths of a percent (00, NS {no spiral}, 36, 54, or 72)

1 = number of specimen if more than one such specimen existed (nothing, 1, 2, or 3)

4.2. Individual Column Specimens

The data are presented in four plots on two pages for each specimen. The Demec (mechanical gage) data are presented first along with a plot of the running average of the Demec readings for that specimen. Data from the electrical gages are presented next. The readings from all gages in the specimen are plotted on one graph. The running average of strains from working electrical gages is presented below in another graph.

Specimen strain data are presented in three separate sets. Data for the concentrically-loaded specimens are presented first, followed by the eccentrically-loaded specimens, and then the unloaded specimens. Data for each of these groups are subdivided into eight-ksi and four-ksi groups. Each of these groups is presented in order from specimens with the least steel to specimens with the most steel.

Data from each set of specimens are accompanied by a brief discussion. The discussion is intended to clarify data and point out any irregularities in the data. Ensuing discussions primarily focus on averaged Demec data because they appeared to be the most reliable.

Specimens showed some similar trends in the data they produced. The electrical gages tended to fail early; therefore, less electrical-gage data was available compared to the amount of mechanical-gage data. The electrical gages for group 1 tended to produce strain readings approximately half those indicated by the mechanical gages. The other three groups had electrical-gage readings that were very similar to the mechanical-gage readings. Some plots for individual gages cease for a period and then resume. The reason for this is that some deformations between Demec points exceeded the range of the original Demec gage. A new gage was used, starting on day 400, so data was again collected from some of these Demec points.

A problem also occurred for some of the electrical gages. Near the 300th day after casting until the 400th day, some electrical gages indicated a reduction in specimen strains. These erroneous readings were likely caused by accidental alteration of the settings on the power supply for the electrical gages. The power supply error was detected and rectified on the 400th day. For an unknown reason, only the specimens of group 1 were affected by this problem.

(a) Concentrically Loaded Specimens

i) 8,000 psi

The 8,000 psi concentrically-loaded specimens are presented in the following figures:

<u>Reinf. ratio</u>	<u>Figures</u>
0.0000	4.1
0.0036	4.2 - 4.7
0.0054	4.8 - 4.13

The first specimen presented is C8-00 (Fig 4.1). The first mechanical gage on this specimen was incorrectly installed and therefore produced no data. Gage 2 shows much higher strains than gages 3 and 4. This is most likely due to some amount of accidental eccentricity. Had gage 1 produced data it likely would have been similar to that of gage 2 and thus the average strains would have been higher. The only electrical gage placed in this specimen failed to work; therefore, there is no electrical-gage data.

Specimen C8-36-1 (Fig. 4.2) also had Demec gage 1 fail to produce data due to improper installation. The remaining gages produced data with little scatter. The Demec points for gages 2 and 3 experienced large deflections, and thus, some data is missing as previously explained. Only two of the five electrical gages provided data (Fig. 4.3). C8-36-1 was a specimen of group 1 and thus experienced a reduction in the electrical gage readings between 300 and 400 days. This phenomenon was explained previously in this chapter. The strains indicated by the electrical gages are approximately half those measured with the mechanical gages

Specimen C8-36-2 (Fig. 4.4) had Demec gage 2 improperly installed and was unreadable. Gages 1, 3, and 4 produced reasonable data with some scatter likely due to unintentional eccentricity. This specimen was also a member of group 1, and thus, the electrical gages exhibited a reduction in the strain readings (Fig. 4.5). Gage 4 of the electrical gages failed after 200 days, and only gage 2 produced strains near those indicated by the mechanical gages. The remaining electrical gages produced data that was approximately half that of the mechanical gages so the average electrical readings were significantly smaller than the average mechanical readings.

Specimen C8-36-3's (Fig 4.6) Demec gages had significant scatter, most likely from eccentricity about a northeast-southwest axis. This would explain gages 1 and 4 having equal strains while gage 3 produced significantly higher strains and gage 2 produced significantly lower strains. Three of the electrical gages produced data that was approximately 80% that of the mechanical data (Fig 4.7).

Specimen C8-54-1 (Fig. 4.8) had two gages fail to provide data soon after loading. Gage 2 failed after 60 days and gage 4 failed after 100 days. Being a member of group 1, this specimen's electrical gages exhibited a significant reduction in strain between 300 and 400 day after casting (Fig 4.9). Two of the five original electrical gages did not work. The remaining three gages produced widely-scattered readings.

Specimen C8-54-2 (Fig 4.10) had only two of its Demec gages installed properly. The remaining gages (2 and 4) indicated a small eccentricity was present in the specimen. The five electrical gages experienced a reduction in response between 300 and 400 days but measured data that was approximately half the magnitude of the mechanical data (Fig. 4.11). The data had little scatter except that due to the small eccentricity indicated by the mechanical gages.

Specimen C8-54-3's (Fig. 4.12) Demec gages produced very good data with little scatter. Electrical gages 2, 4, 5, and 6 did not produce data (Fig. 4.13). Gage 1 failed around day 100, and the data produced after that day was not used in calculating the average strains. The average electrical data indicated a final strain much higher (approximately 25%) than the strain indicated by the mechanical gages.

Specimen C8-72-1 (Fig. 4.14) had two Demec gages improperly installed. The remaining Demec gages produced data with little scatter. This specimen was also a member of group 1 and has the reduction in its electrical data between 300 and 400 days (Fig. 4.15). Two of the five electrical gages failed to provide data, but the remaining three gages produced data with little scatter. Again, the electrical gages indicated strains approximately half those of the mechanical gages.

Specimen C8-72-2's (Fig. 4.16) Demec points for gage 1 quickly deflected beyond the capacity of the Demec gage, and thus, stopped producing data after 70 days. The remaining three gages produced consistent data with lower strains, and so the data from gage 1 was likely erroneous. The gage 1 data was not used in the averaged results. This specimen was a member of group 1 and, like the other group 1 specimens, experienced a reduction in electrical gage data (Fig. 4.17). Two of the electrical gages

failed. The remaining three gages produced data with little scatter. The average electrical strains were again approximately half those indicated by the mechanical gages.

Specimen C8-72-3's (Fig. 4.18) Demec gages produced data that indicated the presence of some eccentricity. Several of the electrical gages either never worked or failed soon after the specimen was loaded (Fig 4.19). Three gages remained functional throughout the test and their running average strain is comparable to the strains measured with the mechanical gages.

ii) 4,000 psi

The 4,000 psi concentrically loaded specimens are presented in the following figures:

<u>Reinf. ratio</u>	<u>Figures</u>
0.0000	4.20 - 4.21
0.0036	4.22 - 4.27
0.0054	4.28 - 4.33
0.0072	4.34 - 4.39

Specimen C4-00 (Fig 4.20) had Demec gage 2 improperly installed so that it was unreadable. The other three gages produced data that showed good agreement. The only electrical gage in the specimen failed after 140 days and produced unreliable data (Fig. 4.21).

Specimen C4-36-1's (Fig. 4.22) Demec gages produced data with little scatter. All five of this specimen's electrical gages produced data (Fig. 4.23). Gages 1 and 2 would occasionally stop producing data for a period only to resume later. The reason for this is unclear but the few data points produced by gage 2 appear to be consistent with other electrical data. The electrical gages measured strains similar to those measured with the mechanical gages.

Specimen C4-36-2's (Fig. 4.24) Demec gages produced data with little scatter. Electrical gages 1 and 4 produced unreasonable data and were not used in determining the running average electrical strains (Fig. 4.25). The other three gages produced reasonable data with a running average similar to that for the mechanical gages.

Specimen C4-36-3 (Fig. 4.26) likely had some eccentricity as indicated by the Demec gages. The electrical gage data show a large amount of scatter with many gages failed or producing erratic results (Fig. 4.27). Gage 4 seemed to produce the most reasonable results.

Specimen C4-54-1's (Fig. 4.28) Demec data have some scatter most likely due to an eccentricity of the applied axial load. All five of the electrical gages initially produced reasonable data, but gages 4 and 1 failed (probably experienced an electrical short) after approximately 75 days (Fig. 29). The average electrical strains were similar to the average mechanical strains.

The Demec data of specimen C4-54-2 (Fig. 4.30) indicated significant eccentricity of the applied axial load. All the electrical gages produced data, but only gage 5 produced reasonable values throughout the loading period (Fig. 4.31). The electrical data from gage 5 is slightly less than the average data from the mechanical gages.

Specimen C4-54-3's (Fig. 4.32) Demec gage 1 deformed beyond the capacity of the gage. Before during so, gage 1 produced data that indicated an eccentricity was present. Because gage 1 produced the highest strains, the average strains after gage 1 failed would likely have been larger. Five of the seven electrical gages produced data but only gage 6 produced reasonable data throughout the loading period (Fig. 4.33). The electrical strain readings from gage 6 were similar to the average mechanical strains.

The Demec data from the four gages on specimen C4-72-1 (Fig. 4.34) indicated a large eccentricity of the applied axial load about the north-south axis. Of the five electrical gages, gage 1 and gage 4 failed soon after loading (Fig. 4.35). The remaining gages produced data similar to the data produced by the mechanical gages.

Specimen C4-72-2's (Fig. 4.36) Demec gage 1 showed a sudden reduction in strains soon after loading. This inconsistency is not compatible with the readings of the other three gages and is likely erroneous; therefore gage 1 was not used in computing the average strain response. Four of the five electrical gages present in the specimen (all but gage 4) failed within the first 100 days (Fig. 4.37). The electrical data from gage 4 was 85% the average from the mechanical gages.

Specimen C4-72-3 (Fig. 4.38) had Demec gage 4 installed improperly. The remaining three gages indicated an eccentricity that would have resulted in gage 4 measuring less-than-average strains. This means that the true average was probably less

than that indicated in Fig. 4.38. All seven electrical gages produced erroneous data (Fig. 4.39).

iii) General Discussion

The Demec mechanical gage readings generally provide what appears to be reliable data. When eccentricity is apparent in the results, the loading apparatus has been visually inspected and, in most cases, the eccentricity can be confirmed.

The electrical gage data are not nearly as reliable as the Demec gage data. Specimens from group 1 all have electrical gage readings approximately half those from the mechanical gages. Specimens in the other groups have average gage readings that are very similar to average readings from the mechanical gages.

The electrical gages of groups 2 and 4 had a higher failure rate than the gages of groups 1 and 3. This may be attributed to two separate problems: the possibility of poorer workmanship in water proofing the gages at the time of application, and the possibility that gages were somehow contaminated when allowed to sit for several months after having been applied to the reinforcing bars prior to casting in the concrete specimens. All strain gages were applied prior to January, 1996. Specimens in groups 2 and 4 were not cast until May 15, 1996. The experience of other researchers suggests that humidity can adversely affect the gages if they are left in place on the reinforcing bars for a long period prior to casting.

(b) Eccentrically Loaded Specimens

i) 8,000 psi

The 8,000 psi eccentrically-loaded specimens is presented in the following figures:

<u>Reinf. ratio</u>	<u>Figures</u>
0.0036	4.40 - 4.41
0.0072	4.42 - 4.43

Specimen E8-36's (Fig. 4.40) Demec gage readings clearly indicate effects of the eccentric axial load intentionally placed on the specimen. The Demec points for gage 4

deflected beyond the capacity of the gage after 80 days. Data up to 80 days from gages 3 and 4 were similar and thus, the average of readings from gages 3 and 4 would be expected to be similar to what is presented in Figure 4.40 if gage 4 had not stopped producing data. Of the five electrical gages, gages 2, 4, and 5 produced data (Fig. 4.41). Gages 2 and 4 produced unreliable data; measured strains should have been much greater than the strains measured by gage 5 which appeared reasonable because they were located near high the compressive-stress-side of the column. It should be noted that this is the specimen that was loaded to only $0.30*f_c'*A_g$, as compared with all other specimens which were loaded to $0.40*f_c'*A_g$.

All of specimen E8-72's (Fig. 4.42) Demec gages produced reasonable data. The effect of the eccentricity in applied axial load is evident in the data. The electrical gages produced widely-scattered data (Fig. 4.43). Gages 4, 5, and 6 were located in the high-strain portion of the column. It was difficult to determine if gage 6 failed after 100 days or if data from the gage were correct. Gage 6's data were used in the running average but, had they not been included, the average strains would have been much greater.

ii) 4,000 psi

Strain responses for the 4,000 psi eccentrically-loaded specimens are presented in the following figures:

<u>Reinf. ratio</u>	<u>Figures</u>
0.0036	4.44 - 4.45
0.0072	4.46 - 4.47

The effect of the intentional eccentricity of the applied axial load is apparent in specimen E4-36's (Fig. 4.44) Demec readings. The Demec data are reasonably consistent and have little scatter. Only one of the five electrical gages provided meaningful data (Fig. 4.45). The working gage was gage 1 which was in the middle of the specimen near the neutral axis. This resulted in strains measured by gage 1 being between strains indicated by the mechanical gages on each side of the column.

The intentionally-eccentricity load is also apparent in specimen E4-72's (Fig. 4.46) Demec readings. There also seems to be an additional unintended eccentricity that caused the strains measured by gage 1 to be larger than the strains measured by gage 2, and the strains measured by gage 4 to be less than the strains measured by gage 3. Gage 1's Demec points deflected beyond the capacity of the gage and so data is missing after day 70. Data from gage 1 would have likely continued to be greater than those from gage 2 and thus, the average should be higher than presented. Only one of the electrical gages produced data (Fig. 4.47). This gage also appears to have failed after 90 days.

iii) General Discussion

For these eccentrically-loaded columns, the Demec data appears to be very reliable. The eccentricities are clearly indicated by the Demec data.

The electrical-gage data for all four eccentrically-loaded specimens are not very reliable. Very few gages produced reasonable data. The data that was produced came mainly from gages in the center of the specimens and therefore did not provide data that would aid in the interpretation of creep behavior of eccentrically loaded columns.

(c) Unloaded Specimens

i) 8,000 psi

The 8,000 psi unloaded specimens are presented in the following figures:

<u>Reinf. ratio</u>	<u>Figures</u>
0.00NS	4.48 - 4.51
0.0000	4.52 - 4.55
0.0036	4.56 - 4.57
0.0054	4.58 - 4.59
0.0072	4.60 - 4.61

Specimen U8-NS-1 (Fig. 4.48) had both its Demec gages produce reasonable data. The electrical gage in this specimen was in the center of the column and produced data that was approximately half the strains measured by mechanical gages (Fig. 4.49). This specimen was a member of group 1 and seemed to suffer from the same problems with the electrical gages as the loaded specimens from group 1.

Specimen U8-NS-2's (Fig. 4.50) Demec gages produced data with significant unexplained scatter. The only electrical gage in this specimen produced just two data points (Fig. 4.51).

Specimen U8-00-1's (Fig. 4.52) Demec gages produced data with little scatter. Both the electrical gages placed in the specimen produced data that experienced the

reduction between 300 and 400 days associated with the specimens in group 1 (Fig. 4.53). Gage 1 was in the center of the specimen and produced data similar to the mechanical gages. Gage 2 was placed on the spiral reinforcement and thus, was not considered in the average data.

Both of Specimen U8-00-2's (Fig. 4.54) Demec gages produced data but the data showed significant scatter. Electrical gage 1 appeared to fail at approximately 80 days (Fig. 4.55). Gage 2, which was placed on the spiral reinforcement, did not produce data.

Some scatter is present in the data for Specimen U8-36 (Fig. 4.56). All five electrical gages produced data (Fig. 4.57). However, gage 4 failed quickly and gage 3 failed at approximately 100 days. The remaining three gages produced data with little scatter but with a running average much different from that for the mechanical gages.

Specimen U8-54's (Fig. 4.58) Demec gages produced data with significant scatter, although the data from the two gages followed the same trend. The five electrical gages present in the specimen produced data with little scatter (Fig. 4.59). However, as was typical of other group 1 specimens, the average of electrical gage responses was much smaller than the mechanical gage average.

Both the Demec gages in specimen U8-72 (Fig 4.60) were readable but they produced data with significant scatter. All but one of the five electrical gages produced data (Fig. 4.61). These electrical data also exhibited the typical quirks of data from group 1 specimens.

ii) 4,000 psi

Strain responses from the 4,000 psi unloaded specimens are presented in the following figures:

<u>Reinf. ratio</u>	<u>Figures</u>
0.00NS	4.62 - 4.65
0.0000	4.66 - 4.69
0.0036	4.70 - 4.71
0.0054	4.72 - 4.73
0.0072	4.74 - 4.75

Specimen U4-NS-1's (Fig. 4.62) Demec gages provided strain data with little scatter. The only electrical gage cast in the specimen failed after 20 days (Fig. 4.63).

Only one of specimen U4-NS-2's (Fig. 4.64) Demec gages was readable. The only electrical gage placed in this specimen produced data similar to the mechanical gage data (Fig. 4.65).

Specimen U4-00-1's (Fig. 4.66) Demec gages produced data with significant scattered. Two electrical gages were placed in this specimen (Fig. 4.67). Gage 1 was cast in the center of the column and appeared to fail after 120 days. Gage 2 was placed on the spiral reinforcement in the column.

The two Demec gages in specimen U4-00-2 (Fig. 4.68) produced data with little scatter. The two electrical gages in the specimen produced data that appeared not too meaningful (Fig. 4.69). Gage 1 was positioned in the center of the column. Gage 2 was placed on the spiral reinforcement.

Specimen U4-36's (Fig. 4.70) Demec data from the two gages in the specimen had little scatter. Only two of the five electrical gages in the specimen produced data (Fig. 4.71). Of those two gages, it is difficult to determine if one failed or if there was simply significant scatter. In any case, the running average of the electrical data is much less than the running average of the mechanical data.

Specimen U4-54's (Fig. 4.72) Demec gages produced data with very little scatter. Three of the five electrical gages produced data but it is difficult to determine if any of those failed during testing (Fig. 4.73). The running average of the electrical data is much less than the running average from the mechanical data.

Both Demec gages from specimen U4-72 (Fig. 4.74) produced very consistent data. Only two of the five electrical gages present in the specimen produced data (Fig. 4.75). Both of these gages appear to have malfunctioned.

iii) General Discussion

The data provided by the Demec gages appears to be very reliable and reasonably consistent. Nearly all of the gages on the unloaded specimens were readable. Very little scatter was observed in the data for most specimens.

Contrasting this is the data from the electrical gages. Very few of the electrical gages were useful throughout the test. Those that were useful produced inconsistent and much smaller strains than indicated by the mechanical gages.

3. Comparison of Concentrically Loaded Specimens

In comparing the strain responses for various specimens, only the average Demec readings were considered. This was done to simplify the necessary graphs. Electrical-gage data was not used further because of its inconsistent and questionable nature.

For purposes of comparison, the results for the concentrically-loaded specimens are shown together in Figures 4.76 through 4.83. The data for the specimens are presented as follows:

<u>Concrete Strength</u>	<u>Reinf. Ratio</u>	<u>Figure</u>
8,000 psi	All	4.76
8,000 psi	0.0036	4.77
8,000 psi	0.0054	4.78
8,000 psi	0.0072	4.79
4,000 psi	All	4.80
4,000 psi	0.0036	4.81
4,000 psi	0.0054	4.82
4,000 psi	0.0072	4.83

To enable a more meaningful comparison the data have been plotted with day zero corresponding to the day each specimen was loaded. Specimens from groups 1 and 2 make up the 8,000 psi specimens. Groups 3 and 4 contain the 4,000 psi specimens. Because groups 1 and 2 were cast on different dates and from different concrete batches, the variation in specimen responses between these two groups is of added interest. This difference is also present between groups 3 and 4, although to a lesser degree. To aid in seeing the variations between the different groups, strain data from specimens in groups 2 and 4 have been plotted with triangles as their symbols. All specimens have had their data corrected for strain variations due to temperature.

i) 8,000 psi

The average strain responses for the 8,000 psi concentrically-loaded specimens are presented in Figure 4.76. Because the specimens of Group 2 had lower concrete strengths than the specimens in group 1, yet were loaded the same, they tended to experience greater strains. This difference is most notable early in the loading period. The specimens from group 2 then appeared to creep at a slower rate than specimens from group 1. It should be noted that the average strain from all nine specimens reached beyond 2,070 micro-strain, the nominal yield strain of 60 ksi steel.

To clearly see the difference between the specimens with the same longitudinal reinforcement ratios, specimens with equal reinforcement ratios were plotted separately.

Figure 4.77 shows the response of specimens having a reinforcement ratio of 0.0036. Specimen C8-36-3 is from group 2 and clearly has a greater initial strain than specimens C8-36-1 and C8-36-2. The other two specimens had concrete strengths significantly greater than their design strengths yet still experienced total average axial strains well above 2,070 micro-strain.

Figure 4.78 illustrates average strain responses from those specimens with a reinforcement ratio of 0.0054. The response of group 2 specimen C8-54-3 and group 1 specimen C8-54-1 was nearly the same for the first 100 days of loading. The response of

C8-54-1 then dropped suddenly due to a failed gage as explained in Section 4.2. All three specimens experienced maximum average strains greater than 2,070 micro-strain.

Figure 4.79 shows the average strain responses of specimens with reinforcement ratios of 0.0072. Specimen C8-72-3 was a group 2 specimen with the lower strength concrete, and had significantly larger initial and maximum strains as expected. All three specimens experienced average strains greater than 2,070 micro-strain.

ii) 4,000 psi

A summary of the average axial strain responses for the 4,000 psi concentrically-loaded specimens is presented in Figure 4.80. Because the group 4 specimens had lower concrete strengths than group 3 specimens, yet were loaded the same, they tended to experience greater strains. This difference is less noticeable than for the 8,000 psi specimens because the difference in strengths was much smaller for the 4000 psi specimens. It should be noted that the maximum average strain response for only one of the nine specimens exceeded 2,070 micro-strain, and that specimen (C4-72-3) appeared to do so based on what appears to be an erroneous reading. Some of the specimen responses came very close to reaching 2,070 micro-strain and likely would have exceeded it given a few more months of loading.

To clearly see the differences between responses of specimens with the same longitudinal reinforcement ratios, responses of specimens with the same reinforcement ratios were plotted separately.

Figure 4.81 shows the average response of specimens with a reinforcement ratio of 0.0036. Specimen C8-36-3 was from group 4. The slightly lower concrete strength of group 4 specimens was not reflected by larger strains for specimen C8-36-3. All three specimens experienced average strains below 2,070 micro-strain, although specimen C4-36-2 very nearly reached this level of response.

Figure 4.82 shows the response of specimens with a reinforcement ratio of 0.0054. Again, any significant differences related to concrete strength between the response of the specimen from group 4 and the specimens from group 3 were not evident. All three specimens experienced maximum average strains well below 2,070 micro-strain.

Figure 4.83 shows the response of specimens with a reinforcement ratio of 0.0072. Specimen C8-72-3 was the group 4 specimen with lower-strength concrete and had a significantly larger initial and maximum strain than specimen C4-72-1, as expected. Surprisingly, the group 3 specimen C4-72-2 experienced strains similar to the lower-strength specimen, C4-72-3. This may have been due to a missing Demec or gage to

application of too much load. Only the response specimen C4-72-3 is exceeded 2,070 micro-strain, but C4-72-2 was very near that level.

iii) General Discussion

The concrete strength and modulus differences from groups 1 and 2 were clearly noticeable in responses of the 8,000 psi specimens. The smaller differences in strength and modulus for groups 3 and 4 were not perceptible. Because load used for the specimens was a function of the nominal concrete design strength, the 4,000 psi specimens experienced small elastic strains.

There was some concern that specimens had been mislabeled and caused some of the inconsistent strain responses (e.g. specimen C4-2-2 {Fig. 4.83} having such high strains). This possibility was investigated and refuted. The number of electrical gages in the specimens was a function of the number of No. 2 bars. The concern about mislabeling was dismissed by counting the number of wire leads exiting the questioned specimens.

4. Concrete Cylinders

Table 4.1 presents the average strength of concrete cylinders from each of the castings. The average moduli of elasticity are presented in Table 4.2. Strengths and moduli were determined at various dates after casting. For groups 3 and 4 testing was done on day 49 instead of day 56.

TABLE 4.1

Group	Design Strength 28 days, psi	Small Cylinders, (4 inch x 6 inch)			Large Cylinders, (6 inch x 12 inch)		
		Actual Strength 14 days, psi	Actual Strength 28 days, psi	Actual Strength 56 days, psi	Actual Strength 14 days, psi	Actual Strength 28 days, psi	Actual Strength 56 days, psi
1	8,000	9,645	10,368	10,044	8,419	9,182	9,415
2	8,000	6,900	7,527	6,789*	6,524	6,918	7,026*
3	4,000	5,060	5,502	5,486	4,896	5,389	5,661
4	4,000	4,117	4,443	4,635*	4,222	4,461	4,686*

* Test performed on day 49.

TABLE 4.2

Group	Predicted M.O.E. 28 days, ksi	Small Cylinders, (4 inch x 6 inch)			Large Cylinders, (6 inch x 12 inch)		
		Actual M.O.E. 14 days, ksi	Actual M.O.E. 28 days, ksi	Actual M.O.E. 56 days, ksi	Actual M.O.E. 14 days, ksi	Actual M.O.E. 28 days, ksi	Actual M.O.E. 56 days, ksi
1	5,098	5,435	5,088	5,203	-	5,401	5,417
2	5,098	4,963	4,659	4,481*	4,888	5,104	4,486*
3	3,605	4,044	4,247	3,046	4,240	4,257	4,161
4	3,605	3,593	3,532	3,882*	3,919	3,677	4,014*

* Test performed on day 49.

Figures 4.84 through 4.87 present this information graphically.

5. Reinforcing Steel

Yield and ultimate strengths were determined from four, 18 inch long bar samples. The average yield stress for the reinforcing steel was 68 ksi, indicating a yield strain of 2,344 micro-strain, and the average ultimate stress was 74 ksi.

Chapter 5. COMPARISON OF EXPERIMENTAL RESULTS WITH ACI 209R-86

5.1 Introduction

This chapter presents an analytical method for estimating long-term strains present in concrete, and summarizes the results of calculations of strains in the column specimens. The method used was recommended in report ACI 209R-86 entitled “Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures”. The predictions that were made based on this method are compared in this chapter to the measured experimental results presented in Chapter 4.

5.2 Summary of ACI 209R-86 Procedure

The ACI 209 procedure utilizes estimates of basic creep and shrinkage coefficients, then applies several correction factors to account for the effects of age at loading, differential shrinkage, length of initial moist curing, ambient relative humidity, average thickness of member or volume-to-surface area ratio, ambient temperature, slump, fine aggregate percentage, cement content, and air content.

The correction factors were determined for each specimen and were applied to the basic creep and shrinkage predictions to determine ultimate creep and shrinkage strains for the columns. The correction factors used, and the ultimate values computed are presented in Table 5.1 for the 8,000 psi specimens and in Table 5.2 for the 4,000 psi specimens.

5.3 Comparison of Predicted and Experimental Results

To perform a comparison between the predicted results based on ACI 209-86 and the experimental results presented in Chapter 4, it is necessary to include the effects of the reinforcing steel in the columns on the predicted results. Four calculations must be performed for each specimen to include the effects of the reinforcement. The initial strain, modified creep strain, modified shrinkage strain, and sum of these three values are needed.

The transformed area method was used to predict the initial elastic strains due to load. The measured compressive strength was used to predict the 28-day modulus of elasticity by assuming $E_{28} = 57,000 * (f'_c)^{0.5}$. The modulus of elasticity for the reinforcement was assumed to be 29,000 ksi. For each specimen the initial strain was calculated as:

$$\epsilon_{\text{initial}} = \frac{P}{[A_g (1 - \rho_g) + n\rho_g A_g] \cdot E_{ci}} \quad (\text{Equation 5-1})$$

where

$\epsilon_{\text{initial}}$ = initial strain in reinforced concrete specimen due to applied load

P = applied load

A_g = gross cross-sectional area of concrete column

E_{ci} = modulus of concrete at time of loading (taken as 28-day modulus)

A_{st} = total area of longitudinal reinforcement

n = modular ratio (E_{st}/E_{ci})

E_{st} = modulus of reinforcing steel

ρ_g = A_{st}/A_g

To compute the strain in the specimens after a period of time, the effective modulus of elasticity was substituted for E_{ci} in equation 5-1. The effective modulus was calculated as:

$$E_{\text{eff.}} = \frac{E_{ci}}{(1 + \nu_t)} \quad (\text{ACI-209 3-1})$$

where

- $E_{\text{eff.}}$ = Effective modulus of concrete at time considered after loading
- E_{ci} = modulus of concrete at time of loading (taken as 28-day modulus of concrete)
- ν_t = creep coefficient at time t
- t = time after load applied (in days)

Therefore,

$$\epsilon_{\text{initial}} + (\epsilon_{\text{creep}})_t = \frac{P}{[A_g(1 - \rho_g) + A_g\rho_g n] \cdot E_{\text{eff.}}} \quad (\text{Equation 5-2})$$

where

- $(\epsilon_{\text{creep}})_t$ = strain in reinforced concrete specimen due to creep at time considered after loading
- $n_{\text{eff.}}$ = modular ratio at time after loading considered ($E_{st}/E_{\text{eff.}}$)

To predict the shrinkage strains in a specimen it is necessary to apply the restraining force in the longitudinal steel to the transformed area of the specimen. The force present in the reinforcement due to the specimen attempting to shrink is approximately:

$$P_{\text{resisting}} = (\varepsilon_{\text{sh}})_t E_{\text{st}} A_g \rho_g \quad (\text{Equation 5-3})$$

where

$P_{\text{resisting}}$ = restraining force developed in longitudinal reinforcing steel due to shrinkage of concrete

An equation for strain due to shrinkage is then found by applying this restraining force to the transformed area of the specimens as:

$$(\varepsilon_{\text{shrinkage}})_t = (\varepsilon_{\text{sh}})_t - \left\{ \frac{(\varepsilon_{\text{sh}})_t E_{\text{st}} A_g \rho_g}{[A_g (1 - \rho_g) + A_g \rho_g n] \cdot E_{\text{ci}}} \right\} \quad (\text{Equation 5-4})$$

where

$(\varepsilon_{\text{shrinkage}})_t$ = strain in reinforced concrete specimen at time considered

Finally, the total strain is calculated by summing the initial creep and shrinkage strains.

$$(\varepsilon_{\text{total}})_t = [\varepsilon_{\text{initial}} + (\varepsilon_{\text{creep}})_t] + (\varepsilon_{\text{shrinkage}})_t \quad (\text{Equation 5-5})$$

where

$(\varepsilon_{\text{total}})_t$ = total strain in reinforced concrete specimen at time considered after loading

These equations were used to predict strain histories for the specimens by varying the time after loading and calculating the total strain at that time. These data are

presented with measured strain data previously presented in Chapter 4 of this thesis. The results of this comparison are presented in the following figures:

Design Concrete <u>Strength</u>	<u>Group</u>	<u>Figure</u>
8,000 psi	1	5.1
8,000 psi	2	5.2
4,000 psi	3	5.3
4,000 psi	4	5.4

The curves in these figures plotted with solid lines represent the measured Demec strains modified for temperature, and with the loading day plotted as day zero. The dashed curves represent the analytical data created using ACI 209-86 considering the measured conditions (e.g. temperature and slump).

5.4 General Discussion

a) 8,000 psi Specimens

The analytical results and experimental measurements for the specimens in group 1 (Figure 5.1) appear to correspond reasonably well. The calculated strains fall within the scatter of the measured results.

For the specimens of group 2 (Figure 5.2) the experimental results indicate strains slightly higher than those predicted by the ACI 209 method. Measured strains tended to increase at a higher rate than the predicted strains. This may be due to the specimens in group 2 being in an environment with high temperature during the first few months of loading. Only a single average temperature was used in the ACI 209 procedure. However, the average temperature during the first two months of loading was

91 degrees Fahrenheit. The average temperature used in the ACI 209 procedure was 75 degrees Fahrenheit.

b) 4,000 psi Specimens

The measured strains in the columns of group 3 (Figure 5.3) are significantly higher than the predicted strains. The cause of this discrepancy might be attributed to the manner in which measured data were corrected for temperature changes. Because the measured data were altered based on ambient temperature measurements and not the temperature inside the specimens, actual temperature differentials from one date to another may have been lower. If this were true the temperature-corrected measured strains would have been smaller and thus in better agreement with the predicted strains. This may have effected the specimens in group 3 more than the other specimens because early in their testing period larger temperature variations were present as compared to the variations the other specimens experienced early in their testing periods.

Measured and predicted strains for specimens in group 4 (Figure 5.4) exhibit fairly good agreement. Initially, the measured strains are greater than the predicted strains, but after 200 days the predicted strains fall within the scatter of the measured strains. The early differences may again be due to the higher initial average temperature (90 degrees Fahrenheit) compared to the average temperature during the testing program used to predict the strains (75 degrees Fahrenheit).

Chapter 6. SUMMARY AND CONCLUSIONS

6.1 Introduction

This investigation was carried out to examine the long-term behavior of axially loaded reinforced concrete columns and to determine if longitudinal reinforcement requirements for columns can be reduced. Specifically, it has been hypothesized that the current code minimum longitudinal reinforcement ratio for reinforced columns could be reduced from the current one percent requirement. This experimental program and related analyses were designed to determine the possibility of such a reduction. All Columns in this study had reinforcement percentages less than one percent and were loaded for longer than 12 months. Plots of the measured strains versus time (Figures 4.76 through 4.83) indicate that the rate of strain increase after twelve months of loading was very small. Because of this small rate of strain, additional strains in the future are expected to be small. The results of the experimental investigation were compared to strain predictions based on an analytical method reported by ACI-209.

6.2 Experimental Investigation

For this investigation, 38 concrete columns were cast. The columns were nominally 8 inches in diameter and 48 inches tall. Twenty-four of the columns were subjected to sustained axial load with a load equal to $0.40 \cdot f_c' \cdot A_g$, which is approximately the largest possible service load based on ACI 318 load requirements. To maintain the load, large coil springs were employed. Column specimens were loaded between 14 and 28 days after casting, and were contained in reduced humidity enclosures. The effects of several variables were investigated, including:

1. Concrete Strength

Nominal strengths of 4,000 psi and 8,000 psi.

2. Reinforcement Ratio

Reinforcement percentages of 0.36%, 0.54%, and 0.72%.

3. Eccentricity

No eccentricity and eccentricity of the axial load equal to 0.10 times the column diameter.

6.3 Comparison of Experimental Results with Predicted Analytical Results

The experimental results are presented and compared with the analytical results in Chapter 5. An analytical method recommended by ACI 209 was used to predict the strain responses of the specimens. Age at loading, ambient humidity, ambient temperature, concrete strength, and several other parameters were considered in the analysis. The predicted values were either equal to or slightly less than the measured values.

Temperature effects were the probable cause of discrepancies between measured and predicted results. Because only a single average temperature was used in the ACI 209 method, elevated temperatures experienced by specimens in groups 2 and 4 during the initial two months of loading were not reflected in the computed responses. The early high temperatures tended to increase early strains measured in the specimens. Additionally, the measured data were corrected for temperature differentials, but the temperatures used were ambient temperatures, not internal column temperatures.

6.4 Conclusions

Although the specimens had not ceased creeping when data collection was discontinued, the rate of creep had dropped to a sufficiently low level to encourage confidence in any conclusions drawn from the recorded data. Several conclusions can be made from the experimental and analytical results.

- 1) Temperature and humidity affect creep and shrinkage significantly.
- 2) Strain response predictions made using the ACI 209 method agreed reasonably well with measured data but tended to under-predict strains when higher temperatures were encountered early in the loading period.

3) It was necessary to correct measured strains for temperature effects to produce reasonable results.

4) As the ratio of dead-to-live load is increased, the amount of steel required to prevent passive yielding is increased.

5) As concrete compressive strength is increased, the amount of steel needed to prevent passive yielding is increased.

6) If the conditions which cause creep in concrete are at the standard values as defined by ACI Committee 209, then for many material strengths and live load-to-dead load ratios, the minimum percentage of longitudinal reinforcement could be reduced below one percent. This is demonstrated in Table 1.1 in Chapter 1.

7) If the conditions which cause creep in concrete are at the upper-bound values as reported by ACI Committee 209, then for many material strengths and live load-to-dead load ratios, the minimum percentage of longitudinal reinforcement could not be reduced below one percent. This is demonstrated in Table 1.2 in Chapter 1.

8) From conclusions 6 and 7, it appears that it may be acceptable to reduce the minimum reinforcement requirement for certain conditions, but in general, it cannot be reduced. To permit the minimum amount of steel to be reduced, an equation or table that takes into account material strengths, live load-to-dead load ratio, and creep and shrinkage factors could be developed. Unfortunately, the factors that affect creep and shrinkage are usually out of the designer's control (e.g. loading age, temperature, humidity, air content, and cement content). If the worst case is assumed for these factors, the minimum amount of steel that is needed in nearly all cases to preclude passive yielding of longitudinal reinforcement is not less than one percent.

9) The compression steel in all eccentrically-loaded columns reached strains well beyond yield strain. None of the columns failed, but significant visible curvature is present in all four. Test specimens had significant concrete cover ($3/8''$ or $1.5 d_b$) and transverse reinforcement ($\rho_s = .025$). Had the specimens been fabricated with less cover and reduced transverse reinforcement with longer spacing, it is possible that spalling of

cover and instability of longitudinal reinforcement might have been observed for the curvatures experienced in these specimens.

6.5 Further Research

A set of specifications could be developed to reduce the amount of creep in columns. These specifications could require low water-cement ratios (possibly employing super-plasticizers), low-permeability aggregates, and require longer periods before loading. Based on these specifications a worst-case scenario for creep effects could be determined for various loading conditions and material strengths. Recommendations could then be made for reduced percentages of steel when these specifications are followed. A testing program should also be developed to verify the performance of columns designed with these specifications.

Further research could also be directed towards determining the behavior of longitudinal reinforcement after yielding occurs. The amount of cover required to prevent a bar from buckling between transverse ties could be determined.

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