MINIMUM MILD REINFORCING REQUIREMENTS

FOR CONCRETE COLUMNS

by

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Thesis

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DEDICATION

This thesis is dedicated to my grandmother, Mrs. Mabel Ziehl.

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This on-going investigation involves the response of reinforced concrete columns under sustained axial load. It is being conducted to establish the feasibility of reducing the minimum longitudinal reinforcing steel percentage as currently required by the ACI code. A review and discussion of previous studies is included.

A total of 38 reinforced concrete columns have been fabricated and are being maintained in reduced-humidity enclosures. Twenty-four of the 38 columns are being subjected to near-constant axial load. The remaining 14 are unloaded control specimens. Of the loaded columns, four are loaded with an axial eccentricity of approximately 10% of the column diameter.

The variables investigated were concrete strength and percentage of longitudinal reinforcement. It is intended that the columns will be maintained for a period of at least two years from the time of casting.

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Chapter 1. INTRODUCTION

1.1 Background

Since the 1930's a minimum reinforcement ratio of 1% (based on cross-sectional area computed using gross-section dimensions) has been required in reinforced concrete columns and piers. This minimum quantity of reinforcement was intended to prevent "passive yielding" of the longitudinal reinforcement which occurs when load is transferred gradually from concrete to steel as the concrete deforms (creeps) under sustained axial load.

The 1% minimum reinforcement ratio was based on tests conducted during the 1920's and 30's[1-9] using low to medium-strength materials; nominal concrete compressive strengths were in the range of 2,000 to 5,000 psi and the steel yield was in the range of 39 to 54 ksi. The 1% limit was first published as part of a committee document by the American Concrete Institute - American Society of Civil Engineers (ACI-ASCE) Joint Committee 105 in 1933[7], and was adopted in the Building Code[10] published by ACI in 1935. These documents are discussed in Chapter 2.

Today, structural concrete compressive strengths below 4,000 psi are uncommon and can easily range up to 10,000 psi. In addition, today's common reinforcing steel has a nominal yield strength of 60 ksi. As a result, it is likely that the tests conducted more than 60 years ago and the code limit for minimum column longitudinal reinforcement are no longer valid for today's columns constructed with modern construction materials.

Recent analysis of the minimum reinforcement ratio in reinforced concrete columns (ASCE-ACI Committee 441 - Concrete Columns) indicates that it may now be appropriate to reduce the minimum reinforcement ratio to approximately 0.5%. However, before such a change is made it would be prudent to verify this limit with a comprehensive experimental study.

Because a substantial percentage of all bridge piers require less than the minimum 1% longitudinal reinforcement to satisfy strength demands, use of the current minimum reinforcement requirements may result in nearly twice as much longitudinal reinforcement in these piers as may really be needed to withstand the effects of creep. Reduction of the minimum longitudinal reinforcement requirement would result in economic savings in the form of reduced material and related transportation costs, savings in labor costs resulting from fewer longitudinal bars to be placed, and the modest added benefit of less congestion in piers.

1.2 Objective and Scope of This Investigation

The objective of this investigation is to determine the behavior of reinforced concrete columns which are reinforced below the current code-required minimum longitudinal reinforcement ratio of 1.0% (ACI 318-95[11] Section 10.9.1). It is hoped that this lower limit of 1.0% may be reduced for certain cases. The applicable section is quoted below:

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.

The work described herein involves experimental tests that incorporated variable concrete strengths, reinforcement ratios, concentric versus eccentric application of axial loads, and comparison of preliminary experimental results with long-term responses predicted by the analytical method recommended by ACI Committee 209R-86[12] .

a) Experimental

A total of 38 conventionally reinforced concrete columns have been cast and are currently being tested. Each column has a nominal cross-sectional diameter of 8 inches and is 4 feet long. Of the 38 columns, 24 are subjected to axial load. The applied axial load is $0.40 * f_c' * A_g$ (with one exception). This load is the maximum service load which can be derived from ACI 318-95[(12)](Section 10.3.5.2 (Eqn. 10-2) for required strength of a tied column (using the approximation $A_g * f_c'$ equal to the strength of the column). The load is maintained with heavy coil springs. The columns were cast in cardboard forms, and the forms were stripped five days after concrete placement. Columns were loaded between 14 and 28 days after casting. Strain measurements are being made using a mechanical Demec gage and electrical strain gages. Ambient humidity in the enclosures containing the test specimens has been reduced as much as was practical and affordable within the budget of the research project. Details of the experimental program are discussed in greater detail in Chapter 3.

The following variables were investigated:

1. Concrete Strength

   Nominal design strengths (at 28 days) of 4,000 psi and 8,000 psi.

2. Reinforcement percentage

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

3. Eccentricity

   No eccentricity and eccentricity equal to 0.10*column diameter.

To determine material properties, several 4 by 8 inch and 6 by 12 inch cylinders were cast with each group of columns. Cylinders were tested for modulus of elasticity and compressive strength evaluation at 7, 14, 28, and 56 days after casting. Longitudinal steel specimens were tested for yield and ultimate strength.

b) Analytical

Concrete exhibits pronounced visco-elastic behavior during loading and immediately thereafter. This visco-elastic behavior tends to decrease with time, and after several years the deformation under sustained stress tends to a limiting value[13].

This visco-elastic behavior is commonly referred to as creep. Two types of creep are generally discussed. The first is referred to as basic creep. This is creep which occurs without moisture exchange (i.e. the creep that would occur if specimens were stored in a saturated environment). The second type is drying creep. Drying creep may be thought of as shrinkage enhanced by applied stress[14].

The following conditions tend to increase creep in concrete[14]:

1. Increased water-cement ratio
2. High permeability aggregates
3. Early loading
4. Increased ambient temperature
5. Reduced ambient humidity, and
6. Reduced volume-to-surface area ratio

These same factors (with the exception of early loading) tend to increase shrinkage.

The report ACI 209R-86[12] entitled "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures" presents an analytical procedure to predict creep and shrinkage strains in unreinforced concrete. These predicted values can then be applied to reinforced concrete members, and ultimate values for creep and shrinkage strains in the member can be obtained. The ACI 209R-86 method is described in detail and is applied to the test columns in Chapter 5. It is noted that the ACI 209 procedure does not distinguish between basic and drying creep.

In the report ACI 92-S26 entitled, "Longitudinal Steel Limits for Concrete Columns" by C.H. Lin and R. W. Furlong[15], two rationales are proposed for establishing a lower limit for longitudinal reinforcing steel. The first rationale is developed on the basis of limiting the size of tension cracks in the concrete column.

This rationale does not rely on creep and shrinkage effects and is not considered in this thesis. The second rationale is based on the prevention of passive yielding of longitudinal reinforcement. Passive yielding will occur if strains in the concrete column due to load, creep, and shrinkage surpass the yield strain of the reinforcing steel. It is with this rationale (prevention of passive yielding) that this thesis is concerned.

The following is a brief summary of the rationale developed in the report "Longitudinal Steel Limits for Concrete Columns" for preventing passive yielding of the reinforcing steel.

As concrete undergoes shrinkage and creep, compressive force is transferred to the longitudinal reinforcing steel. Total stress in the longitudinal reinforcement is the sum of the stress under service load and the stresses due to creep and shrinkage. For the prevention of passive yielding, it is necessary to limit the lower-bound of the steel ratio.

Figure 1.1 is of use in understanding the relationships developed.

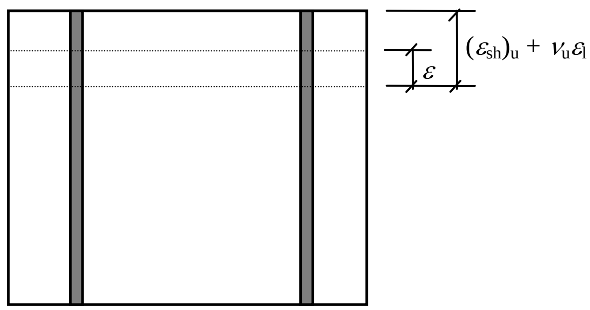

FIGURE 1.1 Schematic of Reinforced Concrete Column

The deflection due to dead load (ε_l) can be obtained from the following relationships:

$$1.4D + 1.7L = \varphi P_o \qquad \text{(Eqn. 1.1)}$$

becomes

$$1.4D + 1.7XD = \varphi P_o \qquad \text{(Eqn. 1.2)}$$

where:

D = service dead load

L = service live load

X = live load-to-dead load ratio = (L / D)

φ = capacity reduction factor
(0.70 is used for tied columns [the columns used in the test program are spirally reinforced but do not meet the ACI code requirements for spiral columns])

P_o = design axial load strength of column for zero eccentricity

$\varphi P_o = 0.80\varphi\,[0.85 * f'_c\, A_g\,(1-\rho_g) + A_g\,\rho_g\, f_y]$

(the factor of 0.80 is included to account for accidental eccentricities and roughly corresponds to the older code requirement of accidental eccentricity = 0.10*h for tied columns)

A_g = gross area of reinforced concrete column

ρ_g = ratio of longitudinal reinforcement = (A_{st} / A_g)

A_{st} = total area of longitudinal reinforcement

f_y = yield stress of longitudinal reinforcement

therefore,

$$D = \frac{\varphi P_o}{(1.4 + 1.7X)} \qquad \text{(Eqn. 1.3)}$$

The strain due to dead load (ε_l) is simply the dead load divided by the transformed area of the column multiplied by the modulus of concrete at time of loading:

$$\varepsilon_l = \frac{D}{\left[A_g E_{ci}\left(1 + \rho_g (n-1)\right)\right]} \qquad \text{(Eqn. 1.4)}$$

where all terms are as defined previously and

E_{ci} = modulus of concrete at time of loading

n = modular ratio at time of loading = (E_{st} / E_{ci})

E_{st} = modulus of reinforcing steel

The following equation gives the value for ε (refer to Figure 1.1). It is derived by applying the force developed in the longitudinal steel as the concrete tries to shorten to the transformed area of the reinforced column.

$$\varepsilon = \frac{n\rho_g(\varepsilon_{sh})_u}{(1-\rho_g+n\rho_g)} + \frac{n_{eff.}\rho_g\nu_u\varepsilon_l}{(1-\rho_g+n_{eff.}\rho_g)} \qquad \text{(Eqn. 1.5)}$$

where

ε = strain in reinforced column when force developed in longitudinal steel is applied to the transformed area (refer to Figure 1.1)

$(\varepsilon_{sh})_u$ = ultimate strain in unreinforced concrete due to shrinkage (normal range = 415 x 10^{-6} to 1070 x 10^{-6})

ν_u = ultimate creep coefficient (normal range = 1.30 to 4.15)

$n_{eff.}$ = effective modular ratio = $n(1+\nu_u)$

In this equation, the modular ratio is modified for creep strains only as opposed to being modified for both creep and shrinkage strains. This is done to be consistent with the experimental approach in which creep and shrinkage strains are commonly evaluated separately and then added to initial strains to arrive at total strains.

The total stress in the longitudinal reinforcing steel (including live load) is:

$$f_s = E_s\,[(\varepsilon_{sh})_u + \nu_u\,\varepsilon_l - \varepsilon\,] + nE_c\varepsilon_l\,(1+X) \qquad \text{(Eqn. 1.6)}$$

where

f_s = total stress in longitudinal reinforcing steel

The maximum value of f_s allowed can be expressed as:

$$f_s = Rf_y \qquad \text{(Eqn. 1.7)}$$

where

$0 < R < 1.0$

If the L/D ratio is known, Equations 1.3 through 1.6 represent a series of four equations with four unknowns (D, ρ_g , ε_l and ε) which can be readily solved. If a value of 1.0 for R is used (no safety factor) then tables can be generated for different values of L/D ratio, steel yield stress, and concrete strength. For these calculations the concrete modulus is assumed to be 57,000 * $(f_c')^{0.5}$, and the modulus of steel is assumed to be 29 x 10^6 psi. Tables 1.1 and 1.2 were developed in this manner. Table 1.1 is based on standard conditions as defined in ACI 209, and therefore, the following values for ν_u and $(\varepsilon_{sh})_u$ apply:

ν_u = 2.35 (normal range = 1.30 to 4.15)

$(\varepsilon_{sh})_u$ = 800 x 10^{-6} (normal range = 415 to 1070 x 10^{-6})
(800 x 10^{-6} used in lieu of 780 x 10^{-6} for consistency with Lin and Furlong report)

TABLE 1.1 Minimum % of Longitudinal Reinforcement
(for ν_u = 2.35 and $(\varepsilon_{sh})_u$ = 800 x 10^{-6})

		L/D							
f'c	**fy**	**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

For other than standard conditions, upper bound values for the normal range of the ultimate creep coefficient (ν_u) and shrinkage strain $(\varepsilon_{sh})_u$ are:

$$\nu_u = 4.15$$

$$(\varepsilon_{sh})_u = 1070 \times 10^{-6}$$

Table 1.2 presents the results for minimum reinforcement percentage when these more severe values for ultimate creep coefficient and shrinkage strain are applied in the appropriate equations.

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

		L/D							
f'c	**fy**	**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

In both Tables 1.1 and 1.2, the values are shown shaded when the percentage of required reinforcement is below 1.0 percent of the gross cross-sectional area of the column. It is clear when comparing Table 1.1 and Table 1.2 that increasing the ultimate creep coefficient and ultimate shrinkage strain to the upper-bound values (though still within the normal range) has a significant impact on the amount of longitudinal steel required to prevent passive yielding. The reinforcement percentage also increases significantly with concrete strength. This is due to the fact that the concrete modulus does not vary linearly with strength. The reinforcement percentage increases as the live load-to-dead load ratio decreases. This is expected because a higher percentage of dead load results in higher creep strains. It is important to note that these equations and tables apply only to concentrically loaded columns.

Table 1.1 indicates that if conditions exist such that the ultimate creep coefficient and shrinkage strains are at or below average values, and if little or no eccentricity is present in the column, then the minimum reinforcement ratio could be lowered for concrete strengths equal to or less than 8,000 psi and L/D ratios greater than or equal to 0.5.

Table 1.2 indicates that if the more severe upper-bound conditions for the creep coefficient and shrinkage strains exist, and little or no eccentricity is present in the column, then the minimum steel ratio could be lowered for concrete strengths equal to or less than 8,000 psi provided the L/D ratio is greater than or equal to 2.0.

Note that the case of over-design of a column is addressed in ACI 318-95[11] Section 10.8.4. This section essentially permits a maximum reduction of the minimum steel percentage from 1.0% to 0.50% for cases in which the column is over-designed by a factor of 2 or more. The section is quoted below:

10.8.4 - Limits of section

For a compression member with a larger cross-section than required by considerations of loading, a reduced effective area A_g not less than one-half the total area may be used to determine minimum reinforcement and design strength.

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

2.1 Introduction

This chapter summarizes the significant series of concrete column investigations which were carried out in the first half of this century. The summary begins with early work performed at the University of California. The most significant investigations, which were conducted during the 1930's at the University of Illinois and Lehigh University, are also reviewed here.

2.2 Davis and Davis[1] - (March 1931)

This study is one of the earliest reported investigations of creep and shrinkage of reinforced concrete columns. Part five of the study is the most applicable. The stated purpose of this portion of the study was to determine the effect of reinforcement on creep of concrete, and the effect of creep of concrete on stress in the reinforcement.

Davis and Davis tested a total of 8 reduced scale columns. All columns were 10 inches in diameter and 20 inches tall, and varied in reinforcement ratio and load condition. Two specimens were unreinforced, two were reinforced with 1.9 percent longitudinal steel in conjunction with 1.33 percent spiral reinforcement, and four specimens were control specimens with no applied load.

The columns were axially loaded in a condition of constant stress. This was achieved with steel plates and car springs. Prior to loading, the columns were stored for 50 days in 100% humidity and ten days in ambient conditions. Sixty days after casting, axial load producing a stress of 800 psi was applied to the four loaded specimens. The results presented in the report indicate a loading period of 18 months. The columns were loaded and stored in a controlled atmosphere in which temperature was held constant at 70 degrees Fahrenheit (plus or minus 1 degree F) and relative humidity was maintained at 50 percent (plus or minus 1 percent). No mention was made of load adjustment to compensate for creep and shrinkage of the columns.

One-half inch square internally threaded brass plugs were mounted in the forms prior to casting. Stainless steel inserts were screwed into the brass plugs after the concrete hardened. Three such inserts were installed around the circumference. In addition, three sets of gage holes were made in the longitudinal reinforcing bars. Strain measurements were made with a ten inch fulcrum type mechanical gage.

Significant conclusions were as follows:

1. For the unreinforced columns, combined creep and shrinkage after 18 months was approximately six times greater than the instantaneous deformation that occurred after load application.

2. For the reinforced columns, combined creep and shrinkage after 18 months was approximately four times the instantaneous deformation.

3. Stress in the longitudinal steel was reported as follows:

Stress due to instantaneous deformation	5,700 psi
Stress due to creep	11,400 psi
Stress due to shrinkage	13,200 psi
Total change in steel stress	30,300 psi

4. Assuming the load not carried by longitudinal steel was uniformly distributed over the full concrete cross-section, sustained stress in the concrete was reduced from 775 psi to 300 psi over the 18 month period.

5. The yield point of the steel should be a design consideration when columns are subjected primarily to dead load and conditions which cause significant shrinkage.

The conclusions from this investigation assumed that creep and shrinkage can be treated separately. Therefore, whatever change in length was not attributed to shrinkage was assumed to be due to creep under sustained load.

A literature summary included with this investigation referenced a study conducted by F.R. MacMillan(16). This study involved the instrumentation of columns in an actual building on the University of Minnesota campus. The results indicated a change in steel stress in the range of 36,000 to 45,000 psi.

2.3 Richart and Staehle[2] - Second Progress Report - University of Illinois (March 1931)

This study was part of the very significant concrete column investigations carried out in tandem at the University of Illinois and Lehigh University. Series 3 of this study concerned columns under sustained load. This second progress report dealt with a loading period of 20 weeks.

Richart and Staehle tested 108 reinforced concrete columns in this series. All columns had an outside diameter of 8-1/4 inches. The columns varied in reinforcement ratio and load condition. Of the 108 columns, 60 were loaded and 48 were without load. Forty-five unreinforced columns were added to the study. Of these 45 columns, six were subjected to sustained load. Longitudinal steel ratios of approximately 1.5, 4 and 6 percent, and spiral steel ratios of 1.24 and 2 percent were investigated. Longitudinal steel percentages were based on the core area of the column. The reinforcement details are as follows:

Type of longitudinal bars	Reinf. ratio	Yield stress
(4) 1/2-inch dia. round	1.57 percent	45,600 psi
(8) 1/2-inch square	3.98 percent	53,400 psi
(4) 5/8-inch dia. round and (4) 3/4-inch dia. round	5.98 percent	39,300 psi 51,100 psi

Type of spiral bars	Reinf. ratio	"Useful" limit	Ultimate
No. 5 rod at 1.35 inch pitch	1.24 percent	49,400 psi	79,500 psi
1/4 inch dia. at 1.19 inch pitch	2.0 percent	48,200 psi	74,200 psi

Nominal concrete strengths of 2000, 3500 and 5000 psi were investigated. The actual strength, modulus of elasticity, and computed modular ratios at 56 days were as follows:

Nominal strength	Actual strength	Modulus of Elasticity	Ratio of Moduli
2,000 psi	2,200 psi	2,830,000 psi	10.6
3,500 psi	3,730 psi	3,800,000 psi	7.9
5,000 psi	5,460 psi	4,290,000 psi	7.0

The modulus of elasticity was based on the slope of the stress-strain curve taken at 30 to 50 percent of the ultimate concrete stress.

The columns were axially loaded in a condition of constant stress. Two columns were loaded in tandem by placing them end to end. Railroad car springs were used to maintain the applied load. Five different types of springs were used. Load was applied by hand-tightening nuts until the appropriate spring displacement was reached. It was noted that no major eccentricities were noticed due to the loading. The applied load varied from 38,000 pounds to 130,400 pounds. One adjustment was made to the load at three months. The springs were checked for permanent deformation. None was noticed.

All specimens were made and cured in a high-humidity room and were stored in the room for 56 days. Then the specimens were loaded and maintained as follows:

1) Design load, lab air

2) No load, lab air

3) Design load, high-humidity room

4) No load, high-humidity room

The lab air varied in temperature from 70 to 90 degrees Fahrenheit and from 40 to 90 percent relative humidity. The high-humidity stored condition was a constant 70 degrees Fahrenheit and 100 percent relative humidity.

The spiral reinforcement was manufactured to a close tolerance and had an outside diameter of 8 inches. Steel forms were used, and were removed 24 hours after casting. The columns were then placed in the high-humidity room or wrapped with wet burlap. The gage points were drilled immediately after form removal.

Initial strains were read at day one. At 56 days, strains were again measured. Strains were measured at 1, 3, 4, 7, 14 and 28 days after loading, and each 28 days thereafter. A 10-inch Whittemore (mechanical) gage was used. Several different individuals were employed to read the gages.

Six inch diameter by 12 inch tall concrete cylinders were made at the time of casting. Two were tested at 56 days and two at one year.

Temperature was not accounted for in the results. It was noted that temperature change in a specimen would certainly occur at a slower rate than temperature change in the laboratory air. It was argued that accounting for such a change would only serve to complicate the results. Furthermore, a 10 degree change in temperature would lead to only a 2,000 psi change in stress of the longitudinal steel.

Significant conclusions after 20 weeks of loading were as follows:

1. Most creep and shrinkage occurred within the first five months.
2. The 56-day modulus of elasticity for the three different concrete strengths was as follows:

Design strength	Modulus of Elasticity
2,000 psi	2,830,000 psi
3,500 psi	3,800,000 psi
5,000 psi	4,290,000 psi

3. Spiral reinforcement had little effect on longitudinal creep and shrinkage.
4. The greatest increase in steel stress (after 20 weeks of loading) was 14,800 psi. The change in stress was generally 6,000 to 14,000 psi for the columns stored in ambient conditions.
5. The lightly reinforced 1.5 percent columns demonstrated the highest change in steel stress. The most highly reinforced columns (6.0 percent), demonstrated the lowest change in steel stress. It was speculated that this was due to the fact that a given amount of creep decreases the concrete stress much more rapidly in columns with a large amount of longitudinal reinforcement than in those with a small amount.
6. No marked change in the appearance of the curves was noticed due to variation in the concrete strength.
7. The average increase in steel stress after the first five months was as follows:

Longitudinal Reinforcement	Increase in steel stress
6.0 percent	9,200 psi
4.0 percent	9,700 psi
1.5 percent	13,200 psi

8. The tests of plain concrete columns demonstrated that creep of the concrete diminished with time.

9. A quantity referred to as the "sustained modulus of elasticity" which includes the effects of shrinkage, creep and elastic deformation was introduced. The values after five months were three times as great as the initial values.

10. Steel stresses after five months exceeded one-half the yield stress of the steel for only one case. It was concluded that the steel yield point was not likely to be reached.

11. More action was observed for columns stored in ambient conditions as opposed to those stored in the high-humidity room.

12. It was noted that the modulus of elasticity for specimens stored in the high-humidity room showed significant increase with time. This was offered as a possible explanation for why specimens stored in ambient conditions exhibited more creep under load than the moist-stored specimens.

13. It was speculated that if the steel did yield, large deflections would ensue and the spiral steel would become more actively involved.

2.4 Slater and Lyse[3] - Second Progress Report - Lehigh University (March 1931)

This was the companion investigation to that carried out at the University of Illinois. Series 3 concerned columns under sustained load. This second progress report dealt with a loading period of 20 weeks.

A total of 108 columns were tested. All columns were of similar dimensions to those tested at the University of Illinois. Some of these columns were unloaded companion columns. The length of the columns was reported to be 60 inches.

The columns were stored in conditions similar to those in the Illinois investigation. All columns and control cylinders were initially stored in a high-humidity room for 56 days. Temperature in the high-humidity room was kept at a constant 70 degrees Fahrenheit. The humidity was very nearly 100 percent. Temperature in the laboratory air varied from 60 to 95 degrees Fahrenheit. No mention was made of the relative humidity.

Columns were loaded in a state of constant stress. Columns were loaded in pairs by stacking them end-to-end. Load was applied by means of an 800-kip vertical screw-type testing machine at 56 days. This is a different method than the manual tightening of nuts that was used at Illinois. A small excess load was applied to compensate for the elongation of the rods and bending of plates that was expected upon raising the loading head. The columns were then stored in laboratory air or moist cured according to schedule.

Load on the columns stored in ambient laboratory conditions was readjusted after three months. The entire loading rig was placed back in the loading machine and the initial loading procedure was again carried out. At this time, the average decrease in load was found to be seven percent. Four percent was attributed to the deformation of the columns, and three percent to permanent set of the springs. Columns stored in the high-humidity room did not require readjustment.

Nominal concrete strengths of 2000, 3500 and 5000 psi were investigated. The water content was kept constant at 39.0 gallons per cubic yard. The water-cement ratio was varied as follows:

Design strength	Water-cement ratio
2000 psi	0.864
3500 psi	0.686
5000 psi	0.531

Concrete cylinders were cast with each column in keeping with the Illinois investigation.

The modulus of elasticity was measured at 56 days with an extensometer. The modulus was defined as the slope of the tangent at a stress of 500 psi. The average strength and modulus results at 56 days were as follows:

Nominal strength	Actual strength	Modulus of Elasticity
2,000 psi	2,230 psi	3,300,000 psi
3,500 psi	3,580 psi	3,800,000 psi
5,000 psi	5,260 psi	4,400,000 psi

Each column was instrumented with 20 gage lines for strain in the steel and 20 gage lines for strain in the concrete. All readings were taken twice and then averaged. The steel strain was found to be very nearly equal to the concrete strain with the possible exception of two specimens which were off by approximately 50 millionths. When two opposing gages were averaged, it was found that the results showed good agreement for the entire column.

As in the Illinois tests, shrinkage results were commonly subtracted from the time-deflection curves, and the remaining deformation was attributed to creep.

Significant conclusions (after 20 weeks of loading) were as follows:

1. A large increase in strain was noticed from two to four weeks after load was applied. After this, the rate of increase became smaller as time progressed.
2. The increase in deformation was much smaller for the moist-stored columns as compared to the columns stored in ambient conditions.
3. The rate of increase was practically independent of the concrete strength. This was attributed to the higher load applied to the columns with higher concrete strength.
4. The rate of increase was found to be practically independent of the amount of spiral steel.
5. The rate of increase was greatest for columns with the smallest steel ratio, and smallest for those with the highest steel ratio.
6. For the first four weeks, shrinkage was approximately the same for the three different longitudinal reinforcement ratios. After four weeks, shrinkage was greatest for the

columns with the smallest steel ratio and least for those with the largest steel ratio. After 20 weeks, the average shrinkage results were as follows:

Longitudinal reinforcement	Strain due to shrinkage
1.5 percent longitudinal	300 millionths
4 percent longitudinal	240 millionths
6 percent longitudinal	190 millionths

7. The higher-strength concrete produced higher shrinkage. This was especially true after four weeks.
8. Loaded columns No. 47 and No. 48 exhibited a tensile stress in the concrete of 90 psi after 20 weeks. Apparently, all load was being carried by the longitudinal steel at this point. These specimens were reinforced with 6 percent longitudinal steel.
9. The highest steel stress recorded at the end of 20 weeks was 42,660 psi. This was approximately four times the initial stress. This value was associated with columns No. 45 and No. 46 which were reinforced with 1.5 percent longitudinal reinforcement. This stress was very near the steel yield stress of 49,500 psi.
10. For the unloaded columns, the concrete experienced significant tensile stresses. This tensile stress was highest for those columns with 6 percent longitudinal reinforcement. The average maximum concrete tensile stress for columns No. 107 and No. 108 was 450 psi. No cracks were noticed in the concrete.
11. Strength of a column depended on the sum of the steel and concrete strengths regardless of the modulus of elasticity of either.

The question was raised as to whether the capacity of a column will be reduced when longitudinal steel is stressed (by time-dependent deformations) beyond its yield point. Also, questions of the effects of eccentricity, redistribution of moment in indeterminate structures, and deflections under indefinite periods of loading were raised.

This investigation concluded with theoretical formulations by W. H. Glanville[17] for the prediction of creep, shrinkage and initial elastic deformations. Other early investigations were also discussed.

The F. R. MacMillan study was again mentioned. The MacMillan study was said to have begun two months after casting and to have been monitored for six years by the time this second progress report was produced. One column (column No. 19) exhibited a steel stress of 45,000 psi by this time. It is probable that the steel had reached its yield stress. No physical signs of distress were noticed in the column.

2. 5 Richart and Staehle[4] - Fourth Progress Report - University of Illinois (January 1932)

This report presented further results for the columns under sustained load discussed previously. The second progress report dealt with a loading period of 20 weeks. This fourth report dealt with a loading period of 52 weeks. The strength and deformations when tested to failure were also discussed.

Eighteen plain and 26 reinforced columns were not tested to failure, but rather were retained for further observation during a second year.

As a review, this series consisted of 108 reinforced concrete columns, of which 60 were placed under sustained loads and 48 duplicates were kept in like storage conditions under no load. In addition to the reinforced columns, 45 plain columns of similar dimensions and materials were fabricated. Two storage conditions were investigated as follows:

Storage	Temperature	Relative humidity
Ambient laboratory conditions	70 to 85 deg. F.	40 to 90 percent
Moist room	66 to 74 deg. F.	saturated

Compression in the loading springs was calibrated and adjusted at regular intervals to compensate for strain in the columns.

Cylinders stored in ambient conditions showed an average strength increase of 15 percent while the moist-stored cylinders showed an average strength increase of 30 percent from the age of two months to 14 months.

Stress changes were very small from five months to 20 months. In some cases, stresses actually decreased during the last seven months. It was noted that changes in temperature and humidity may have affected the results to a considerable extent. The five-month observations were made during the winter when humidity was low, and the one-year observations were made during the summer when humidity was high. Thus, any increase in creep may have been offset by expansion due to moisture. Some irregularities due to seasonal variations can be noticed in the results.

The greatest steel stress in any one column after loading for one year was 30,800 psi. This column was loaded in accordance with the New York City building code. The initial elastic stress in the steel was 11,100 psi. Companion columns loaded according to the ACI Code showed a maximum steel stress of 26,700 psi. In very few of the columns did the steel stress exceed 50 percent of yield stress. The maximum deformation was reported to be approximately three times the initial elastic deformation.

Regarding the effect of concrete strength and longitudinal reinforcement ratio, results were similar to those stated in the second progress report. Deformation versus time curves were observed to flatten as time progressed but exhibited enough increase to warrant retaining certain specimens for at least another year.

The unreinforced columns were loaded to 500, 875, and 1250 psi, or one-fourth the nominal 56-day concrete strengths. This was considerably less than the concrete stresses in the reinforced specimens when initially loaded, and slightly greater than concrete stresses in the reinforced specimens after one year of sustained loading. The greatest total deformation was found to be 0.0012 for the 3500 psi concrete. If the effects of shrinkage were deducted, the net creep was essentially the same for the 3,500 psi and 5,000 psi columns. Shrinkage for all three grades of concrete was nearly constant and equal to 0.0004 at eight or nine months.

Modulus information was collected at one year and two months for comparison with that taken at two months. The modulus information was based on the initial tangent up to one-fourth the ultimate strength. The modulus was found to vary in accordance with strength versus time. The results for one year and two months with the dry and moist curing conditions are as follows:

Nominal strength	Dry strength	Moist strength	Dry M.O.E.	Moist M.O.E.
2,000 psi	2,665 psi	3,020 psi	2,985,000 psi	4,080,000 psi
3,500 psi	4,510 psi	4,740 psi	3,980,000 psi	4,820,000 psi
5,000 psi	6,135 psi	6,580 psi	4,170,000 psi	5,195,000 psi

For comparison, the results for two months are shown below:

Nominal strength	Dry strength	Moist strength	Dry M.O.E.	Moist M.O.E.
2,000 psi	2,200 psi	2,200 psi	2,830,000 psi	2,830,000 psi
3,500 psi	3,730 psi	3,730 psi	3,800,000 psi	3,800,000 psi
5,000 psi	5,460 psi	5,460 psi	4,290,000 psi	4,290,000 psi

It was evident that the modulus showed significant increase for the moist-stored columns. It was concluded that the elastic deformation due to load may have decreased by as much as 25 percent in these columns.

Reference was made to the “sustained modulus of elasticity” which is defined as the ratio of stress to deformation from all causes. For dry-stored specimens, this sustained modulus was approximately 25 percent of the initial value after one year. For moist-stored specimens, this sustained modulus was approximately 80 percent of the initial value for the same period.

Load was removed from all columns prior to loading to failure. This permitted recovery of the large elastic strains in the steel and led to tension cracks spaced at about 10 to 12 inches in the concrete. It was noted that when the columns were loaded to the one-year sustained load, cracks closed and the steel and concrete strains corresponded closely with those measured prior to removal of load.

For the unloaded companion columns, no shrinkage cracks were observed. It was noted that the difference in strains between the plain and reinforced columns was in excess of 0.0002, which is an amount of strain generally considered to cause tensile cracking under rapidly applied load. It was concluded that a considerable amount of tensile creep occurred to inhibit the formation of cracks.

Strength of the columns which had been subjected to load for one year was compared to strength of the unloaded companion columns. No significant difference in strength was observed.

The ratio of column strength (dimensions 8-1/4 inch diameter by 60 inch long) to cylinder strength (dimensions 6 inch diameter by 12 inch long) was found to be 0.86 for air-stored columns and 0.71 for moist-stored columns.

It was noted that the ultimate strength values for similar columns at Lehigh University were somewhat lower in most cases due to the use of a spherical head at one end of the column, whereas strength tests performed at the University of Illinois were made using flat test heads on the columns.

Significant conclusions (after 52 weeks of loading) were as follows:

1. The largest stress noted in the longitudinal steel occurred in a column constructed with 3,500 psi concrete and 1.5 percent longitudinal reinforcement. The steel stress reached a value of 30,800 psi. This column was loaded in accordance with the New York City building code. A similar column loaded in accordance with the ACI Code reached a stress of 26,700 psi. These stresses were still well below the yield point of 45,600 psi for the steel in these columns.
2. The ultimate strength of columns under sustained load was the same as the ultimate strength of unloaded companion columns.
3. The increase in strength of control cylinders was about 15 percent for dry-stored and about 30 percent for moist-stored cylinders at one year and 2 months. The modulus of elasticity increased 30 percent over the same period for the moist-stored cylinders but there was no consistent increase for the dry-stored cylinders.
4. The ratio of the strength of 8-1/4 inch by 60 inch plain columns to that of 6 by 12 inch cylinders was 86 percent for one year dry-stored columns and 71 percent for one

year moist-stored columns. This relative strength was apparently obtained for the reinforced columns as well.

This progress report concluded with series 5 and series 6 which were ultimate strength tests of large diameter columns. These series had some applicability and will be briefly discussed.

The purpose of series 5 was to ensure that actual-size columns, as used in building construction, would behave similarly to the model-size columns used in the majority of the test program. Series 5 consisted of 20 columns of 12, 20, and 28-inch core diameters. No cover was used with these columns. Each column had a height of 7-1/2 times the core diameter. The nominal concrete strength was 2,000 or 3,500 psi and the longitudinal reinforcement ratio was 1.5 or 4 percent of the core area. The spiral was always 1 percent of the core volume. It was generally concluded that there seemed to be no variation in strength with the size of the column.

The purpose of series 6 was to investigate the effect of the concrete shell. Fourteen columns having core diameters of 8, 12, 20, and 28 inches were investigated. The height was again 7-1/2 times the diameter. The longitudinal steel ratio was 4 percent for all columns. The columns with shells were, in general, found to be as strong as those without. For the smaller 8 inch diameter columns, in which the strength of the shell was greater than the margin of strength produced by the spiral, the columns with shells were considerably stronger than those without.

2.6 Lyse and Kreidler[5] - Fourth Progress Report - Lehigh University (January 1932)

This report presented further results for the columns under sustained load at Lehigh which were discussed previously. In similar fashion to the Illinois test program, the second progress report dealt with a loading period of 20 weeks. This fourth report dealt with a loading period of 52 weeks. The strength and deformations for columns tested to failure were also discussed.

Twelve dry-stored columns were retained for further tests. Eight were under sustained load and four were companion columns. These columns were loaded for 52 additional weeks.

The strength of the dry-stored columns was found to be slightly higher at 60 weeks than at eight weeks. In contrast, the moist-stored concrete showed a significant strength increase of 14 percent after being stored for 60 weeks. The modulus increased similarly. The 60 week results with dry and moist curing conditions are as follows:

Nominal strength	Dry strength	Moist strength	Dry M.O.E.	Moist M.O.E.
2,000 psi	2,240 psi	2,530 psi	2,700,000 psi	4,000,000 psi
3,500 psi	3,590 psi	4,030 psi	4,100,000 psi	4,500,000 psi
5,000 psi	5,520 psi	6,110 psi	3,800,000 psi	4,900,000 psi

Load was adjusted at regular intervals to compensate for deformation of the columns. The permanent set in the springs was found to be four percent for both dry and moist-stored specimens.

After 52 weeks under load, the load was released in intervals and the strains measured. When the load was released, the dry-stored columns developed transverse cracks while the moist-stored columns did not.

At 52 weeks, still very little difference in creep was noticed for different spiral reinforcement percentages. In contradiction to the results of 20 weeks, the results at 52 weeks exhibited a slight increase in creep with increase in the concrete strength.

The higher-strength concrete experienced the greatest shrinkage. The higher the percentage of longitudinal steel, the less shrinkage occurred. The shrinkage was greatest in the dry-stored columns at 20 weeks when air temperature and humidity were at a minimum.

After 52 weeks, the average steel stresses for different concrete strengths were as follows:

Nominal strength	Steel stress
2,000 psi	30,000 psi
5,000 psi	37,000 psi

The rate of increase of stress in the steel was much higher for columns having 1.5 percent longitudinal steel than for columns having 6 percent. It was noted that no stress existed in the concrete after 52 weeks for the columns with 2,000 psi concrete and 6 percent longitudinal reinforcement. The longitudinal steel was assumed to carry all the load.

Every dry-stored column showed transverse cracking upon release of the load. The columns with the highest percentages of reinforcement had the largest crack widths. The moist-stored columns were carefully inspected visually and strain measurements were made. None of the moist-stored columns exhibited cracking.

When tested to failure, transverse cracks in the dry-stored columns did not close completely. The longitudinal steel buckled in the columns without spiral reinforcement. For columns having no spiral reinforcement, the strength of the concrete had little effect on the load carried by the concrete.

The average stress in the steel for dry-stored columns was found to increase under sustained load as follows:

Longitudinal reinforcement	Initial elastic	52 weeks
1.5 percent	6,000 psi	37,000 psi
6 percent	16,000 psi	30,000 psi

The average stress in the steel for moist-stored columns was found to be significantly less than for the dry-stored columns. It increased under sustained load as follows:

Longitudinal reinf.	Initial elastic	52 weeks
Averaged	12,000 psi	19,000 psi

When tested to failure, it was found that the strength of the loaded columns varied between 95 and 112 percent of the strength of the unloaded companion columns. It was concluded that sustained loading had no effect upon the strength of the columns. The strength of the column varied directly with the amount of longitudinal reinforcement and with the yield stress of the longitudinal steel. The rate of variation was nearly the same for the three different strengths of concrete used. The strength of the columns increased quite regularly with an increase in the percentage of spiral reinforcement.

Significant conclusions (after 52 weeks of loading) were as follows:

1. The deformation due to creep under sustained load was slightly greater for columns with higher-strength concrete.
2. The deformation due to creep was greatest for columns having no spiral reinforcement. No substantial difference could be found for columns with 1.2 and 2.0 percent spiral reinforcement.
3. The rate of creep was greatest for columns with the smallest percentages of longitudinal reinforcement.

4. For columns with the same percentage of longitudinal steel, shrinkage was greatest for those with the higher strength concrete.
5. For columns with the same strength concrete, the least amount of longitudinal steel resulted in the greatest shrinkage.
6. The stress-strain curve for any column showed no definite yield point.
7. None of the unloaded columns cracked during the storage period.
8. Stress in the steel of dry-stored columns subjected to ACI working loads for 52 weeks increased from 6,000 to 37,000 psi for columns having 1.5 percent longitudinal reinforcement, and from 16,000 to 30,000 psi for columns having 6 percent longitudinal reinforcement. For moist-stored columns the average stress in the steel increased from approximately 12,000 to approximately 19,000 psi.
9. Sustaining an applied working load for 52 weeks had no appreciable effect on ultimate strength of the columns.
10. Strength of columns having concrete of the same strength and the same total yield strength for longitudinal reinforcement increased with increase in spiral reinforcement.
11. Ultimate strength of a concrete column having no spiral reinforcement was considered to equal 75 percent of the cylinder strength times the net core area plus the yield strength of the steel. If a spiral was included, the yield strength of the spiral times its effectiveness ratio was to be added.

2.7 Lyse(6) - Fifth Progress Report - Lehigh University (June 1933)

The intent of this series of tests was to investigate the maximum load that a concrete column could sustain indefinitely. Therefore, all columns in this investigation were loaded to very high percentages of the calculated ultimate load. Twenty-eight columns were loaded from between 70 and 100 percent of the calculated ultimate load. All columns had an outside diameter of 8-1/4 inches and were 60 inches long. All columns had either 4 or 6 percent longitudinal reinforcement and 0, 1.2 or 2 percent spiral reinforcement. The nominal strength of the concrete was 3,500 psi in all cases. The columns were in most cases loaded at 56 days.

The material properties of the reinforcing steel were as follows:

Reinforcement	Yield stress	Ultimate stress
4 percent long.	44,000 psi	64,400 psi
6 percent long.	44,700 psi	70,000 psi
1.2 percent spiral	none	85,500 psi
2.0 percent spiral	none	74,700 psi

In each case, three identical columns were failed by a "fast" loading procedure to determine the ultimate strength of the column. Then companion columns were loaded to significant percentages of this ultimate load and observed until failure occurred.

The loading rigs were similar to those used in previous tests at Lehigh University. Helical springs were again used to maintain load, and initial load was again applied in an 800 kip testing machine. The distance between the outside of the column and the vertical rods was 1/2 inch. Load was measured and adjusted by measuring strain in the steel rods with a mechanical strain gage. Adjustments were made to the load by hand-tightening the nuts.

Column 7 (4 percent longitudinal, 0 percent spiral) was loaded at 56 days to 80 percent of ultimate load for 115 days, then it was removed from the loading rig. The longitudinal steel had been stressed well beyond yield but the column still failed 17 percent higher than the calculated ultimate load.

Column 6 (4 percent longitudinal, 0 percent spiral) was loaded at 56 days to 80 percent of ultimate for a period of 700 days. It was noted that for the first year a large increase in deformation was observed. After the first year, the deformation increased very slowly but did not stop entirely. Total deformation of the column was approximately four times the yield strain of the longitudinal reinforcement. It was noted that, with the exception of a few vertical cracks near the ends of the column (which developed shortly after the time of loading), no signs of distress were present. It was concluded that this column could carry 80 percent of ultimate load indefinitely.

Column 11 (4 percent longitudinal, 1.2 percent spiral) was loaded at 112 days to 95 percent of ultimate load. This column failed after 45 minutes.

Column 12 (4 percent longitudinal, 1.2 percent spiral) was loaded at 112 days to

90 percent of ultimate load. It sustained this load for 65 hours but deflected laterally so much that it rested against the vertical rods of the loading rig. It was removed and loaded to failure.

Column 13 (4 percent longitudinal, 1.2 percent spiral) was also loaded at 112 days to 90 percent of ultimate load. At the time of the report, the column had sustained the load for 500 days. The strain at 500 days was approximately ten times the yield strain of the longitudinal reinforcement. It was noted that column 13 did not appear to be in danger of failure. The cover had spalled off in several places.

Column 14 was stored as a control column for column 13. This column was left unloaded to determine temperature and shrinkage strains. These strains were found to correspond to a stress of 7,500 psi in the longitudinal reinforcement.

Column 18 (4 percent longitudinal, 1.2 percent spiral) was loaded at 56 days to 95 percent of ultimate load. The column sustained this load for one day but buckled so badly the test was discontinued.

Column 19 was similar except it was loaded to 90 percent of ultimate. This column also buckled after one day.

Column 20 (4 percent longitudinal, 1.2 percent spiral) was loaded at 56 days to 85 percent of ultimate. At the time of the report, it had sustained this load for more than 300 days. The strain in the longitudinal steel was approximately 7 times the yield strain. This column had also buckled and was resting on the vertical rods of the loading rig.

Column 21 (4 percent longitudinal, 1.2 percent spiral) was loaded at 56 days to 80 percent of ultimate. At the time of the report, it had sustained this load for more than 300 days. The strain in the longitudinal steel was approximately 5-1/2 times the yield strain. This column also buckled and was nearly resting on the vertical rods of the loading rig. The concrete outside the spiral had begun to spall.

Column 25 (6 percent longitudinal, 2.0 percent spiral) was loaded at 56 days to 90 percent of ultimate. At the time of the report, it had sustained this load for nearly 300 days. Strain in the longitudinal steel was approximately 7-1/2 times yield strain. This column had buckled and was resting on the vertical rods of the loading rig. Concrete outside the spiral had begun to spall.

Significant conclusions are as follows:

1. The longitudinal reinforcement will carry its full yield stress at strains far in excess of yield strain.
2. The strength of the column was not decreased by being strained far beyond the yield point of its steel before loading to failure.
3. A reinforced concrete column will probably carry 80 percent of ultimate load for an indefinite period of time.
4. A column having no spiral or a small amount of spiral reinforcement will carry 80 percent of ultimate load at less deformation and with fewer signs of distress than will a column having a larger amount of spiral reinforcement.

2.8 Richart(7) - Tentative Final Report of Committee 105 (February 1933)

This report summarized the majority of the work carried out at Illinois and Lehigh.

One formula was presented for the ultimate strength of reinforced concrete columns. It was noted that this equation applied to concrete strengths from 2,000 to 8,000 psi and for longitudinal reinforcement of 1.5, 4.0, and 6.0 percent and longitudinal steel yield stresses of 39,000 to 68,000 psi. Other formulas were given for the yield point of all columns and the ultimate strength of tied columns.

It was noted that in extreme cases the steel stresses had reached 30,000 to 42,000 psi after five months of load application. The average increase in steel stress was approximately 12,000 psi in the Illinois tests and about 20,000 psi in the Lehigh tests. From five months to one year the increase was only about 2,000 psi more, and from one year to two years another 2,000 psi was measured.

Design formulas were presented for the maximum permissible load on spirally reinforced columns, and for tied columns. A design formula for the spiral ratio was also given.

A minimum reinforcement ratio of 0.010 was set for spirally reinforced columns and a minimum reinforcement ratio of 0.005 was set tied columns. The minimum number of bars was set as four for both cases. Little or no justification was given for the two different minimum reinforcement ratios.

A minority recommendation for design formulas was given by Bertin and Lyse. They also suggested a minimum ratio of 0.010 for spirally reinforced and 0.005 for tied columns. Again no justification was given for the difference between spiral and tied columns.

2.9 Logeman, Mensch, DiStasio[8] - Discussion of Report of Committee 105 - (Sept.- Oct. 1933)

These discussions were primarily concerned with the split in the committee over the proposed design formulas. In particular, to what extent the spiral could be counted on for ultimate and working loads was discussed by several contributors. The concepts of elastic versus plastic design formulas and behavior was discussed.

DiStasio pointed out the lack of support given for a minimum limit on longitudinal reinforcement of 0.010 for spiral columns as opposed to 0.005 for tied columns. He also called for further testing of the effects of bending prior to the acceptance of formulas for design.

2.10 Richart[9] - Discussion of Report of Committee 105, Closure by Chairman, Committee 105 - (Nov.-Dec. 1933)

This discussion again focused on the division among the committee. The majority report allowed for a smaller contribution of the spiral reinforcement as opposed to the minority report. This was justified by the fact that the spiral did not contribute significantly until very large deformations had taken place.

No discussion was given to the recommended minimum longitudinal reinforcement ratios presented in previous reports.

2.11 Conclusions

In series 3 of the investigations carried out at Illinois and Lehigh in the 1930's a total of 261 column specimens were fabricated. Of these columns, 126 were loaded in a state of near constant stress for at least 52 weeks. Columns were maintained in both an approximately 100%-humidity environment and in ambient conditions in the laboratories.

The columns were loaded in accordance with either the New York City Building Code or the ACI Code. Nominal concrete strengths of 2,000, 3,500 and 5,000 psi were investigated. Longitudinal reinforcing steel varied from 45,600 psi to 51,100 psi yield strength. Reinforcement ratios investigated were 1.5, 4 and 6 percent of the cross-sectional core area of the column.

Regarding columns loaded in accordance with the ACI code, after 52 weeks the largest stress in the longitudinal reinforcement at Illinois was 26,700 psi and that at Lehigh was 37,000 psi. These values were well below the 45,600 psi yield stress of the steel. The specimens at Lehigh had an initial elastic steel stress of 6,000 psi. These stresses were recorded in dry-stored specimens with 1.5 percent longitudinal reinforcement.

It was noted that for columns having 6 percent longitudinal reinforcement the stress increased from an initial stress of 16,000 psi to 30,000 psi after 52 weeks. This increase was reported in the Lehigh results.

When the columns were unloaded and then re-loaded to failure, it was found that the sustained period of loading had no effect on ultimate strength of the columns.

To lend some insight into the behavior of columns when the longitudinal steel is stressed beyond the yield point, a separate series of tests was carried out at Lehigh. These tests were discussed in the fifth progress report from Lehigh. In these tests, columns were loaded from 80 to 100 percent of their ultimate load and held until failure. Some of these specimens exhibited strains as high as ten times the steel yield strain without failing. Stability was a problem however and several of the specimens buckled to such an extent that they were resting on vertical bars of the loading rig. One of the conclusions of this series was that a column will probably carry 80 percent of its ultimate load indefinitely.

As an outcome of these investigations, a minimum reinforcement limit of 0.010 for spiral and 0.005 for tied columns was recommended. It is assumed that the intent was to prevent passive yielding of the longitudinal steel. Little justification for the recommended ratios was given.

Chapter 3. EXPERIMENTAL PROGRAM

3.1 Introduction

This chapter describes the experimental program currently underway in an investigation designed to re-evaluate the minimum 1% longitudinal reinforcement ratio required for columns. A total of 38 concrete columns have been fabricated and are being maintained under various levels of axial load or no load. Twenty-four of the 38 columns are being subjected to near-constant axial load. The remaining 14 are unloaded control specimens. Of the loaded columns, four are loaded with the axial load having an eccentricity of approximately 10% of the column diameter. All columns have a nominal 8-inch outside diameter and are 4 feet 0 inches long. All columns (with the exception of four unloaded control specimens) are spirally reinforced. Material control tests have also been conducted and are continuing as appropriate.

The variables investigated are concrete strength and percentage of longitudinal reinforcement. All columns are stored in an environment where the relative humidity has been generally kept between 30 to 60 percent, and the temperature has been allowed to vary from approximately 50 to 110 degrees Fahrenheit. It is intended that the columns will be maintained and observed for a period of approximately two years from the time of casting.

3.2 Column Details

Details of the longitudinal reinforcement and column cross-sections are shown in Figures 3.1 and 3.2. The average measured outside circumference at mid-height of the columns was 25.25 inches prior to loading. This resulted in an average measured diameter of 8.04 inches and an average "as-built" cross-sectional area of 50.77 square inches.

The longitudinal reinforcing steel consisted of either 0, 4, 6 or 8 No. 2 deformed bars. The No. 2 deformed bars have a measured cross-sectional area of 0.046 square

inches. The resulting longitudinal reinforcement ratios were 0, 0.0036, 0.0054, and 0.0072. All columns (with the exception of four unloaded control specimens) were confined with a number 9 annealed wire spiral. The wire was hand fed around a spinning 6-inch diameter steel mandrel. A pitch of approximately 2 inches (plus or minus 1/4 inch) was achieved in this way. The spiral relaxed upon removal from the mandrel. The final outside diameter of the spiral was approximately 7-1/2 inches, which led to a clear cover over the spiral of approximately 3/8 inch. All spirals were stored outside and moistened for a minimum of ten days to achieve a thin layer of corrosion.

Because some end cracking was observed for columns in Groups 1 and 2, all columns in Groups 2 and 4 were wrapped with fiber-reinforced plastic. This was done to preclude development and growth of any cracks in the end regions during the lengthy period that column specimens are under load.

The nomenclature used to describe each specimen includes a single letter to describe the loading type, a digit to indicate the nominal compressive strength of the concrete, a five-digit number to represent the reinforcement ratio, and a single digit to indicate the number of the specimen if more than one of a particular combination was studied. For example, the specimen name C8-0.0036-1 represents a concentrically loaded specimen (other options are E for eccentrically applied load or U for no load), with a nominal concrete compressive strength of 8000 psi (4 is used for f_c' of 4000 psi), a reinforcement ratio of 0.0036 (other reinforcement ratios are 0.0000, 0.00NS {no spiral}, 0.0054, and 0.0072), and the first of more than one specimen with these properties.

Table 3.1 lists the nominal concrete strengths, reinforcement ratios, whether spiral reinforcement was used, and the number of each combination investigated.

TABLE 3.1 Matrix of Test Specimens

Design concrete strength, psi (& load type)	8 long. bars, and spiral, ρ = 0.0072	6 long. bars, and spiral, ρ = 0.0054	4 long. bars, and spiral, ρ = 0.0036	0 long. bars, and spiral, ρ = 0.0000	0 long. bars, no spiral, ρ = 0.0000
8,000 (concentric)	3 specimens	3 specimens	3 specimens		1 specimen
8,000 (eccentric)	1 specimen		1 specimen		
8,000 (no load)	1 specimen	1 specimen	1 specimen	2 specimens	2 specimens
4,000 (concentric)	3 specimens	3 specimens	3 specimens		1 specimen
4,000 (eccentric)	1 specimen		1 specimen		
4,000 (no load)	1 specimen	1 specimen	1 specimen	2 specimens	2 specimens

Table 3.2 lists actual concrete strengths, reinforcement ratios, column end conditions, load eccentricity, casting dates, age at loading, and group number associated with casting for all 38 specimens.

3.3 Reduced-Humidity Environment

Two reduced-humidity enclosures were built for the columns. The approximate size of each enclosure is 20 feet by 20 feet by 12 feet high. The enclosures were framed using wood studs, and are fully wrapped with a single ply of 6 mil thick clear vapor barrier stapled to the wood studs. The enclosures are free-standing inside a light-gage metal framed warehouse on the J.J. Pickle Research Campus at The University of Texas. One dehumidifier is operating constantly in each enclosure. A plan view of each enclosure and the specimens contained in each is shown in Figures 3.3 and 3.4.

During cold weather, small space heaters are placed within the enclosures to avoid freezing temperatures. Also, an attempt has been made to keep temperatures above 50 degrees Fahrenheit at all times to prevent large fluctuations between summer and winter readings. No attempt has been made to further control the temperature.

The high and low temperatures and high and low relative humidities have been recorded on a regular basis. Readings have been generally taken at two or three days intervals. The readings for each of the four groups of specimens are shown in Figures 3.5 through 3.8.

3.4 Materials

a) Concrete

Two different design concrete strengths were tested. The nominal design strengths of the mixes were 4,000 and 8,000 psi at 28-days.

Concrete was ordered from a local ready-mix plant and delivered to the site generally within 1 hour after the concrete was batched. Two different mix designs were used. The specimens were cast in four different groups. No control could be exerted over moisture content of the coarse and fine aggregate, nor could moisture content of the aggregate be accurately estimated. The projected strength was therefore determined through test mixes with different slumps.

The coarse aggregate was rounded river gravel of 3/8-inch maximum size. A super-plasticizing admixture was used for the 8,000 psi mix, and was added to the mix at the batch plant. No super-plasticizer was used for the 4,000 psi mix. A retarder was used for both mixes.

The mix proportions (as reported by the ready-mix provider) for each of the four groups are shown in Table 3.3.

TABLE 3.3 Concrete Mix Proportions

Group	Batched Quantity, cu. yd.	Sand, lbs.	Cement Type II, lbs.	Water, lbs.	Water added, lbs.	3/8" Rock, lbs.	Ret-arder, oz.	Super-plast-icizer, oz.	Slump, inches
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	5280	58	-	6.5

It is noted that the mix for Group 2 did not appear to be identical to that of Group 1; the aggregate in the mix for Group 2 appeared to be larger and more plentiful.

b) Reinforcing Steel

The longitudinal reinforcing steel used in all cases was deformed with a nominal size of 6 mm (approximately 2/8 in.) and will hereafter be referred to as No. 2 bar. The actual cross-sectional area of a bar was determined by cutting a 44 inch long piece into 11 approximately equal segments and measuring the amount of water displaced in a graduated cylinder. This method was verified by weighing the same bar segments. The cross-sectional area of the bar determined in this manner was 0.046 square inches.

c) Fiberglass Reinforced Plastic

Some longitudinal cracking was noted near the tops and bottoms of the loaded specimens in Group 1 and 3. In order to prevent any such cracks from forming and propagating in the specimens of Groups 2 and 4, it was decided to confine the top and bottom of these specimens with a fiberglass wrap.

The wrap consisted of five layers of woven-roving, with properties similar to E-glass, and a thin resin matrix. The fiberglass wrap was applied by hand prior to loading. It was applied to all specimens in Group 2 and Group 4 including the unloaded control specimens. The height of the wrap is approximately 6 inches.

3.5 Manufacture of Test Specimens

a) Columns

i.) Formwork

Cardboard tubes (EZ Pour) with a nominal inside diameter of 8 inches were used to form all columns. The insides of the tubes were pre-coated with wax for ease of removal. The tubes were saw-cut to 4 foot lengths with a band saw. Care was taken to ensure that the ends were cut as square as possible.

The reinforcing cages were pre-assembled outside the forms and tied with plastic ties. The cages were then placed into the cardboard tubes. Three-sixteenth inch diameter steel rods were used as horizontal spacers at the top and bottom of the cages to maintain the desired concrete cover.

ii.) Casting

The columns were cast in four separate groups. The first through fourth groups consisted of 11, 11, 8, and 8 columns, respectively. Each group included six loaded columns. The remaining columns of each group were control specimens for determining shrinkage and temperature effects. Groups 1 and 2 had a 28-day design strength of 8,000 psi. Groups 3 and 4 had a 28-day design strength of 4,000 psi. The four groups were cast in the following order:

Group	Design Strength	Date
1	8,000 psi	February 7, 1996
3	4,000 psi	April 4, 1996
2	8,000 psi	May 15, 1996
4	4,000 psi	May 15, 1996 (same day as Group 2)

All columns were cast in the vertical position. A wooden platform was fabricated with twelve 9 inch by 9 inch by 1-1/2 inch deep forms to create level pedestals for casting the column specimens. Hydro-stone was cast into the forms, and a 6 mil sheet of vapor barrier was secured on top of the hydro-stone to provide a smooth surface for forming the bottom of the columns. The cardboard tubes (with the reinforcing cages inside) were secured in place with small steel clip angles and screws.

Concrete was placed in the cardboard tubes with a long-handled scoop. The tubes were held in place during this process. A small diameter mechanical vibrator was used to consolidate the concrete.

All columns were cast within the reduced-humidity enclosures. The approximate time required to cast each group of columns was one hour.

iii.) Curing

Immediately after casting, the tops of all columns were covered with a 6 mil vapor barrier and 1 inch thick steel plate to minimize moisture loss through the top end of the column. The vapor barrier and steel plate were left in place for three days. On the third day, the steel plate and vapor barrier were removed from the top of each column, then an approximately 3/8-inch thick hydro-stone leveling cap was cast on the top of each column. The cardboard tube was left in place for five days. On the fifth day, the columns were moved away from the wooden platform, and the cardboard tubes were stripped by hand. The columns were stored on end on the concrete floor until the mechanical strain gage (Demec) points were set. These points were generally installed between the seventh and tenth day after casting. Demec points were installed with the columns in a horizontal position. The columns were then again stored standing on end until loading.

iv.) Application of Fiberglass Reinforced Plastic

The fiber reinforced plastic end-wrap was applied by a representative of Ershigs, Inc. of Gatesville, Texas. The woven roving was first cut to an appropriate size. It was then wetted with resin and applied to the top and bottom ends of all specimens in Groups 3 and 4. The wrap was hand-rolled around the specimen approximately five times. The wrap was allowed to partially dry and then the ends were trimmed by hand with a mat knife.

b) Cylinders

A minimum of twelve 6 x 12 inch cylinders and eighteen 4 x 8 inch cylinders were cast with each group of specimens. The cylinders were compacted by hand in accordance with ASTM standards. Due to space restrictions, the cylinders were cast outside the reduced-humidity enclosures and capped with plastic caps. The cylinders were brought inside the enclosures on the third day after casting, and were maintained in the enclosures thereafter. The plastic caps were removed on the same day that steel plates were removed from the tops of columns. Plastic cylinder molds were removed on the same day cardboard tubes were removed from the column specimens.

3.5 Testing Appurtenances

a) Columns

i.) Testing Frames

Special test frames were developed for the column creep tests. A total of 24 test frames were built. Photographs of typical test frames are shown in Figures 3.9 and 3.10. A schematic of a frame is shown in Figure 3.11.

Two and one-half inch square by 3/16 inch thick steel tubes were used for the legs of all column test frames. Axial load was maintained on each specimen with four or eight triple-coil springs. Two-inch thick steel plates were used to sandwich the springs. The springs were loaded in a 600-kip capacity test machine and found to sustain a load of 20 kips at a deflection of approximately 1-1/2 inch. The load-deformation curve for the springs was found to be approximately linear. Therefore, eight springs were used in the loading frames for the 162 kip applied load while four springs were used in the loading frames for the 81.2 kip applied load. Due to this arrangement, the deformation of the springs under load was approximately 1-1/2 inches. This deformation was monitored with a metal scale attached to the side of each loading frame. The metal scale has an accuracy of 1/64-inch.

Four 5/8-inch diameter Dywidag bars were used for the 4,000 psi specimens and four 1-inch diameter Dywidag bars were used for the 8,000 psi specimens (see Figure 3.11). The Dywidag bars extend approximately 9 inches below the lower steel plate. Load was applied by sliding a 3-inch thick steel plate and hydraulic ram underneath the testing frame, then attaching the plate to the frame by means of Dywidag coupling sleeves and approximately 1-foot long Dywidag extensions connected to the lowest steel plate. With this arrangement, several columns could be loaded by one person in a single day.

In all cases, a 3-inch thick steel plate was used for the top-most plate. Over-sized holes were drilled in all plates to accommodate the Dywidag bars.

All column specimens in Groups 1 and 3 used a pinned end condition about one axis of bending at both ends of the column. This was achieved with 1-1/2 inch steel plates which were machined to accept a 1-1/4 inch diameter steel pin. Due to the difficulty in maintaining a perfectly plumb arrangement of the two pins combined with the inherent imperfections in the columns, noticeable eccentricities in the loading system were apparent when the earliest column specimens were loaded. As a result, the concentrically loaded columns for Group 4 were loaded without the pin supports. One-quarter inch thick Neoprene pads were used top and bottom to distribute the load applied at the ends of the columns and to make the setups more stable. The two eccentrically loaded specimens in this group were loaded with the pin-pin condition and no Neoprene pads.

The Neoprene pads were tried on the concentrically loaded 8,000 psi specimens of Group 2, but due to the higher load the columns tended to "walk" significantly. For this reason, the Neoprene pads were not used for any of the column specimens of Group 2. Again, eccentrically loaded specimens in this group used the pin-pin end condition. Wherever the pin-pin end condition was used, lateral restraint of the top steel plate was provided. This was accomplished with steel angles and wood struts spanning to the sides of the reduced-humidity enclosure (see Figure 3.10).

ii.) Strain Measurements

Various strain measurements were made for all of the columns. Both mechanical and electrical strain measurements were made.

a) Mechanical Gages

Each of the loaded concrete columns was instrumented with four lines of mechanical Demec gages. The points on the lines were 400 mm apart (approximately 16 inches). The Demec points were centered longitudinally along the column (shown in Figure 3.12). The gage lines were situated at 20 degrees from the North-South axis as is shown in Figure 3.13.

The Demec points used were 1-inch long metal H.I.T. anchors manufactured by Hilti. Heads of the anchors were drilled slightly off-center with a small bit to accommodate the attachment points of the Demec gage. Holes to accommodate the H.I.T. anchors were drilled in the concrete columns from 7 to 10 days after casting. The H.I.T. anchors were then set in place using a specially designed brass tamper and a two-part epoxy. The shaft of one H.I.T. anchor was then rotated to the proper location to accept the zero bar of the Demec gage.

The initial readings from the mechanical Demec gages were taken approximately three days after they were set. Measurements from the Demec gages were collected approximately every other day for the first six weeks after casting and approximately once a week thereafter.

b) Electrical Gages

Several of the longitudinal reinforcing bars were instrumented with electrical resistance strain gages. The columns were typically instrumented with four or six electrical strain gages on the longitudinal reinforcing bars. The gages were staggered 4 inches above and below mid-height of the columns to minimize reduction of the cross-sectional area (see Fig. 3.14). The numbering scheme used to identify strain gages is shown in Figure 3.15. All electrical strain gages had a resistance of 350 ohms.

In addition, each column was instrumented with a “floating” electrical strain gage. This device consisted of a 3/16-inch diameter hot-rolled A36 steel rod approximately 10 inches long with threaded ends. An electrical strain gage was attached to the center of the 3/16 inch diameter rod, then shrink-wrap tubing was placed over the rod before washers were locked onto each end with nuts. The resulting assembly was effectively an 8 in. strain gage. The floating gage was held in place with 3/16-inch diameter horizontal steel rods tied to the gage and reinforcement cage. These floating gages were located 8 inches from the bottom of each column and in the center of the cross-section. The location of these electrical strain gage assemblies is shown in Figures 3.14 and 3.15.

Electrical strain gages were zeroed prior to casting the concrete. Readings were taken approximately 10 minutes after the concrete was placed. The gages were then re-zeroed approximately 20 minutes after the concrete was placed. Electrical strain gage measurements were made approximately every three days after casting for a period of four weeks. After four weeks, measurements were generally made weekly.

iii.) Testing Procedure

All loaded columns (with the exception of SPECIMEN E8-0.0036 which was subjected to load equal to $0.30*A_g*f_c'$) were subjected to axial load equivalent to $0.40*A_g*f_c'$. SPECIMEN E8-0.0036 was not loaded as highly due to noticeable cracking on the compression side of the column. This cracking was the result of drilling for the Demec instrumentation points and existed prior to application of load. The resulting applied loads were 81.2 kips for the 4,000 psi specimens and 162 kips for the 8,000 psi specimens. The four eccentrically loaded columns all had eccentricities of 0.80 inches (equivalent to 10% of the nominal column diameter).

Load was initially applied to the specimens with a 300 kip-capacity hydraulic ram. After the load was applied with the ram it was locked in place by hand-tightening the Dywidag nuts beneath the plate under the column specimen (see Fig. 3.11). Pressure in the ram was monitored with a dial gage accurate to 200 psi. A small additional load was initially applied to account for anticipated seating of the nuts on the Dywidag bars. This seating loss was less than 2 percent of the applied load in all cases. No attempt was made to alter the applied load during testing to compensate for temperature effects.

Load was applied to the columns from 14 to 28 days after casting. The exact day in which load was applied to each specimen is shown in Table 3.2.

Change in load (as indicated by the spring relaxation) was monitored every few days. The load could be monitored both with the metal scale attached to the loading frame and also with deformation of the column. When the metal scales indicated 3/64 inch relaxation of the springs (approximately equal to a 3 percent reduction in load) or strain in the column indicated a similar change in load, the initial load was re-established using the hydraulic ram. Again, a slight overload was imposed to account for the anticipated seating of the Dywidag nuts. This re-loading procedure was carried out approximately 60 days after initial loading for all columns.

c) Concrete Cylinders

Small cylinders and large cylinders were tested at 7, 14, 21, 28, 42, and 56 days. At least two cylinders were tested and the results averaged on each testing day. For the 28-day tests, three cylinders were used and the results averaged. At least two large and three small cylinders are being maintained with each group of specimens to obtain strength and modulus information when creep tests are discontinued. On each test day, ultimate strength and modulus of elasticity information was recorded. Strength and modulus tests were conducted in accordance with ASTM C39-61(5) and ASTM C 469-94, respectively.

d) Reinforcing Steel

The yield and ultimate stress of the No. 2 bars was determined through four tensile tests conducted in a 60-kip capacity Tinius-Olson universal test machine.

Chapter 4. PRELIMINARY EXPERIMENTAL TEST RESULTS

4.1 Introduction

The preliminary test results for individual column specimens are shown in Figures 4.1 through 4.73. For purposes of comparison, preliminary test results for all concentrically loaded column specimens are shown with results for the unloaded specimens in Figures 4.74 through 4.79.

The specimens are currently being maintained in the reduced-humidity enclosures described in Chapter 3 and data are being gathered on a regular basis. For the purposes of this thesis, only data collected before July 24, 1996, will be considered. The ages of the different groups of specimens as of July 24, 1996, is as follows:

Group	Casting date	Last reading considered	Age of specimens
1	Feb. 7, 1996	July 24, 1996	167 days
2	May 15, 1996	July 24, 1996	70 days
3	April 4, 1996	July 24, 1996	142 days
4	May 15, 1996	July 24, 1996	70 days

Because temperature changes result in measurable strains in the specimens, strain data for all column specimens have been adjusted for temperature effects. A temperature coefficient of 6.5 micro-strain per degree Fahrenheit was used. The largest temperature differential for the period considered was 39 degrees Fahrenheit. This results in a maximum temperature-induced strain of 246 micro-strain.

The restraint offered by the coil-springs used to maintain load on the column specimens is not significant when compared to the temperature-induced strain. For the worst-case scenario of the stiffest spring group and the most flexible concrete column, the calculated difference between restrained expansion (by springs) and unrestrained expansion for a temperature differential of 40 degrees Fahrenheit was determined to be less than 10 micro-strain. This is less than would be imposed by a temperature differential of only 2 degrees Fahrenheit. Therefore, the restraint offered by the springs was neglected when adjusting strain data from loaded specimens for temperature effects.

In almost all cases, temperature was recorded at the time strain-measuring devices (whether mechanical or electrical) were read. In the event that no temperature reading was available, an average of the high and low temperature readings for the day in question was used. Note that the temperature readings used were ambient temperature readings, as opposed to internal readings in the concrete specimens.

For review, the nomenclature used for each column specimen is illustrated below. For example;

C8-0.0036-1

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

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

0.0036 = number of longitudinal reinforcing bars (0.0000, 0.00NS {no spiral}, 0.0036, 0.0054, or 0.0072)

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

4.2 Individual Column Specimens

The Demec (mechanical) gage data versus time for each column specimen is presented first and is followed by an average of all the Demec gage readings for each specimen. These two plots are shown on a single page. The Demec data for each column specimen are followed by plots of electrical strain gage data versus time for the same column specimen. The electrical gage data are presented for each gage and then an average of the electrical gage data is presented. Again, the two electrical gage plots for each individual specimen are presented on a single page.

Data for the concentrically-loaded specimens are presented first, the eccentrically loaded specimens are presented next, and the unloaded specimens are presented last. Results for the 8,000 psi and 4,000 psi specimens are presented separately. The most lightly-reinforced specimens are presented before the more highly-reinforced specimens.

Some discussion is offered with each set of specimens when appropriate to clarify the results or to point out and explain irregularities in the data.

a) Concentrically Loaded Specimens

i.) 8,000 psi specimens

Strain data for 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
0.0072	4.14 - 4.19

The following is a brief description of the data collected for each specimen.

Demec gage 1 of Specimen C8-0.0000 (Fig. 4.1) was set incorrectly and is unreadable. Gage 2 yielded larger strains than gages 3 and 4. Some eccentricity relative to the north-south axis likely exists in the loading set-up. Therefore gage 1 would be expected to yield strains similar to gage 2, and the average strain would therefore be higher. Only one electrical "floating" gage was cast in this specimen and it failed to operate properly.

Demec gage 1 of Specimen C8-0.0036-1 (Fig. 4.2) was set incorrectly and is unreadable. The three readable gages yielded data with little scatter. Of the electrical gages, only 4 and 5 provided plausible data (Fig. 4.3). The electrical gages measured significantly smaller strains than the mechanical gages (approximately 40% less). As will be demonstrated for practically all electrical strain gage data, this is typical.

Demec gage 2 of Specimen C8-0.0036-2 (Fig. 4.4) was set incorrectly and is unreadable. Some eccentricity relative to the north-south axis likely exists in the loading set-up. Therefore gage 1 would be expected to produce strains similar to gage 2, and the average strain would therefore be higher. With the exception of gage 2, the electrical gages measured significantly smaller strains than the mechanical gages (Fig. 4.5).

The Demec gage measurements for Specimen C8-0.0036-3 (Fig. 4.6) show significant scatter and indicate load being applied eccentrically by the loading apparatus about a skewed axis. The electrical gage data (Fig. 4.7) demonstrates similar scatter but lower strains when compared to the mechanical gage data.

Demec gage 2 of Specimen C8-0.0054-1 (Fig. 4.8) exceeded the range of the mechanical device used to read the gage points approximately 60 days after casting. Demec gage 4 provided significantly higher readings than the other three gages and went out-of-range approximately 100 days after casting. It is not possible for one gage (such as gage 4) to yield substantially higher strains when the other three show good agreement. The initial zero for gage 4 may have been read incorrectly. Therefore, gage 4 was not used in the averaged results.

Demec gages 1 and 3 of Specimen C8-0.0054-2 were set improperly and were therefore unreadable. The data from gages 2 and 4 (Fig. 4.10) tend to indicate a small eccentricity about the north-south axis. All five electrical gages provided plausible data (Fig. 4.11). The electrical gages again measured lower strains than the mechanical gages.

The Demec measurements for Specimen C8-0.0054-3 (Fig. 4.12) show relatively little scatter. All four Demec gages were readable. Only three of the seven electrical gages (Fig. 4.13) provided plausible data. All but one of the electrical gages again measured lower strains than the mechanical gages.

Demec gage 3 of Specimen C8-0.0072-1 was set improperly and was therefore unreadable. The remaining three Demec gages (Fig. 4.14) yield strain data with very little scatter. Three of the five electrical gages provided plausible data (Fig. 4.15) and the data also show little scatter. The electrical gages again yielded lower strains than the mechanical gages.

Demec gage 1 of Specimen C8-0.0072-2 went out-of-range and could no longer be read approximately 70 days after casting. This gage produced significantly higher readings than the other three gages (Fig. 4.16). As was mentioned previously, it is not possible for one gage (such as gage 1) to measure substantially higher strains when the other three gages show good agreement. Therefore, gage 1 data was not used in the averaged results. Three of the five electrical gages provided plausible data (Fig. 4.17) and this data also showed little scatter. The electrical gages again yielded lower strains than the mechanical gages.

The Demec gage readings for Specimen C8-0.0072-3 (Fig. 4.18) yielded strains with significant scatter, indicative of eccentricity in the loading apparatus. The electrical gage data (Fig. 4.19) demonstrate similar scatter and slightly lower strains when compared to the mechanical gage data. In this case, the mechanical and electrical readings showed relatively good agreement. Only two of the seven electrical gages provided plausible data.

ii.) 4,000 psi specimens

Strain data for 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

The following is a brief description of the data collected for each specimen.

Demec gage 2 of Specimen C4-0.0000 was set incorrectly and is unreadable. Data from the remaining three gages (Fig. 4.20) show good agreement. Only one electrical “floating” gage was cast into this specimen. This gage measured smaller strains than the mechanical gages (Fig. 4.21).

Strain data from Demec gages of Specimen C4-0.0036-1 (Fig. 4.22) show little scatter. Of the electrical strain gages, three of the five provided plausible data (Fig. 4.23). Electrical gages 3 and 4 measured strains similar to those measured by the mechanical gages.

Strain data from Demec gages of Specimen C4-0.0036-2 (Fig. 4.24) show little scatter. Three of the five electrical gages provided plausible data. With the exception of gage 2, the electrical gages yield slightly smaller strains than the mechanical gages (Fig. 4.25). Electrical gage 2 provided strain readings very similar to those from the mechanical gages. The three readable electrical gages provided strains with relatively little scatter.

The Demec strain gage data for Specimen C4-0.0036-3 indicate some separation between gages 1 and 2 and gages 3 and 4. This indicates eccentricity about the north-south axis in the load applied by the testing apparatus. The electrical gage data indicate a large amount of scatter. Electrical strain gage data collected by strain gage 4 (Fig. 4.27) is similar to the mechanical gage data.

Strain data measured by Demec gages of Specimen C4-0.0054-1 show some scatter. The pattern suggests eccentricity of the applied load about both axes (Fig. 4.28). Three of the five electrical gages provided plausible data. The electrical strain gage data (Fig. 4.29) exhibited similar scatter, and were typically somewhat smaller than strains measured with mechanical gages.

With the exception of strains measured by Demec gage 2 of Specimen C4-0.0054-2, the Demec gage data exhibited little scatter (Fig. 4.30). Three of the five electrical gages provided plausible data until day 60 (Fig. 4.31). At day 60, gages 2 and 3 ceased to provide data. Strains from the remaining electrical gage (gage 4) were lower than strains measured by the mechanical gages.

Demec strain readings from Specimen C4-0.0054-3 (Fig. 4.32) show significant scatter, indicative of eccentricity in the applied load. Only two of the seven electrical strain gages provided plausible data (Fig. 4.33). Electrical gage 2 measured strains similar to those measured with mechanical gages.

The Demec readings for Specimen C4-0.0072-1 (Fig. 4.34) indicate significant eccentricity about the north-south axis. Three of the five electrical gages provided plausible data, but with significant scatter (Fig. 4.35). The electrical gages yielded lower strains than the mechanical gages.

Demec gage 1 of Specimen C4-0.0072-2 went out-of-range and could no longer be read approximately 45 days after casting. This gage provided strain readings similar to gage 2 prior to going out-of-range (Fig. 4.36). The remaining three gages provided data with little scatter. Two of the five electrical gages provided plausible data (Fig. 4.37). Electrical gage 4 measured strains similar to those measured by the mechanical gages.

Demec gage 4 of Specimen C4-0.0072-3 was placed incorrectly and was therefore unreadable. The three remaining Demec gages measured strains with some scatter, which is indicative of eccentricity in the applied axial load (Fig. 4.38). The electrical gages provided little or no meaningful data (Fig. 4.39).

iii.) General discussion

The Demec gages appear to provide reliable data. When eccentricity of applied load has been apparent in the Demec strain readings, the loading apparatus has been visually inspected and, in most cases, the eccentricity has been visually confirmed.

The electrical gage data is not nearly as reliable as the Demec strain gage data. In some cases, the strain versus time data measured with electrical gages exhibited a significant “dip” in the strain readings prior to loading. This may be due to thermal expansion of the specimens prior to loading. However, not all of the gages measured this same “dip” in strain response. The gages which measured this behavior often became unstable at a later date.

In most cases, the electrical gages measured lower strains than the Demec gages. This may be because the electrical gages are not perfectly aligned with the axis of the reinforcing bars. A very large number of electrical strain gages were applied and some error in the installation process was to be expected. The Demec gages by contrast have a much larger gage length, and therefore, were more easily set in vertical position. The reinforcing bars also may not have been positioned vertically. Both of these factors would tend to produce strain readings which are lower than if the longitudinal bars and strain gages were aligned vertically. However, the most plausible explanation for reduced strain readings by electrical resistance gages is deterioration of the protective sealant over the gage and the connections between the strain gage and electrical leads.

The electrical strain gages of Groups 2 and 4 provided noticeably poorer results when compared to the strain readings made with electrical gages of Groups 1 and 3. This may be attributed to two separate problems. The first is the possibility of poorer workmanship with respect to water-proofing of the gages at the time of application. The second is the possibility that the gages were contaminated in some way when they were allowed to sit for several months after having been applied to the reinforcing bars prior to casting 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. It is possible that humidity adversely affected the gages prior to casting.

b) Eccentrically Loaded Specimens

i.) 8,000 psi specimens

Strain measurements for the 8,000 psi **eccentrically loaded** specimens are presented in the following figures:

Reinf. ratio	Figures
0.0036	4.40 - 4.41
0.0072	4.42 - 4.43

The following is a brief description of the data collected for each specimen.

The Demec gage readings for Specimen E8-0.0036 reflect the intentionally-imposed eccentricity of 0.80 inches about the north-south axis (Fig. 4.40). The strain readings exhibit relatively little scatter. Gages on the east face (gages 1 and 2) measured very small compressive strains. Three of the five electrical gages provided plausible data (Fig. 4.41). Gages 2 and 4 were mounted on longitudinal bars nearest the east and west face, respectively. As would be expected, gage 2 measured slight tensile strains, and gage 4 measured compressive strains. Gage 4 ceased to operate approximately 25 days after casting. It is important to note that this specimen was loaded to 0.30*f_c'*(gross area) as opposed to the other eccentrically-loaded specimens which were loaded to 0.40*f_c'*(gross area). As mentioned in Chapter 3, this was done because drilling for the H.I.T. anchors used for Demec points had caused noticeable damage to the compressive face of the specimen. It was feared that rapid deterioration or failure might occur if the specimen was fully loaded.

The Demec strain readings for Specimen E8-0.0072 also clearly reflect the imposed eccentricity of 0.80 inches about the north-south axis (Fig. 4.42). The strain readings exhibited little scatter. Gages on the east face (gages 1 and 2) measured very small tensile strains. Six of the seven electrical gages provided meaningful data (Fig. 4.43). These electrical gages measured strains that qualitatively reflected what would be expected as a result of the eccentric loading. Gages 2 and 5 were mounted on the longitudinal bars nearest the east and west face, respectively. As would be expected, gage 2 measured slight tensile strains and gage 5 measured compressive strains. These strains are similar to those obtained from the Demec gage data.

ii.) 4,000 psi specimens

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

Reinf. ratio	Figures
0.0036	4.44 - 4.45
0.0072	4.46 - 4.47

The following is a brief description of the data collected for each specimen.

The Demec strain readings for Specimen E4-0.0036 illustrate the effect of the intentionally-imposed eccentricity of 0.80 inches about the north-south axis (Fig. 4.44). The strain readings exhibit little scatter. Gages on the east face measured very small tensile strains. Only one of the five electrical gages (the central “floating” gage) provided plausible data (Fig. 4.45). This gage measured strains that are between the Demec readings on the east and west faces, as would be expected due to the central location of this “floating” electrical gage.

The Demec strain readings for Specimen E4-0.0072 illustrate the effect of the imposed eccentricity of 0.80 inches about the north-south axis (Fig. 4.46). The gage readings exhibit some scatter that is indicative of a slight unintentional eccentricity about the east-west axis. Gage measurements on the east face show very small compressive strains. As for Specimen E4-0.0036, only one of the five electrical gages (the central “floating” gage) provided plausible data (Fig. 4.47). Once again, this gage indicated strains between the values measured by the Demec gages on the east and west face, as would be expected due to the central location of this “floating” electrical gage.

iii.) General discussion

The Demec gages provided what appears to be reliable data. In all four eccentrically-loaded specimens, the intentional eccentricity is clearly noted in the strain data.

In cases where the electrical gages provided useful data, these readings were consistent with Demec readings and what would be expected due to the eccentric-loading condition.

c) Unloaded Specimens

i.) 8,000 psi specimens

Strain data for the 8,000 psi **unloaded** specimens are presented in the following figures:

Reinf. ratio	Figures
0.00NS	4.48 - 4.50
0.0000	4.51 - 4.54
0.0036	4.55 - 4.56
0.0054	4.57 - 4.58
0.0072	4.59 - 4.60

The following is a brief description of the data collected for each specimen.

The Demec gages for Specimen U8-0.00NS-1 measured strains with little scatter (Fig. 4.48). Only Demec gages 1 and 4 were installed in the unloaded specimens. Only one electrical gage (the central "floating" type) was cast into this specimen. This gage provided data that is reasonably compatible with data measured by the mechanical gages (Fig. 4.49).

The Demec gages for Specimen U8-0.00NS-2 measured unusual scatter in strain data from day 18 to day 30 after casting (Fig. 4.50). This scatter is unexplained. Only the floating electrical gage was cast into this specimen, and provided no reliable data.

The Demec gage readings for Specimen U8-0.0000-1 measured strains with little scatter (Fig. 4.51). Electrical gage 1, which is of the "floating" type, provided strain data which is similar to that from the Demec gages. Electrical gage 2 was placed on the spiral reinforcement. This gage indicated slight compressive strains. This is consistent with shrinkage of the concrete (Fig. 4.52).

The Demec strain gage readings for Specimen U8-0.0000-2 exhibit significant scatter (Fig. 4.53). Electrical gage 1, the "floating" type, indicated strains in excess of those data provided by the mechanical gages (Fig. 4.54). Electrical gage 2, which was placed on the spiral reinforcement, provided no meaningful data.

The Demec strain gage readings for Specimen U8-0.0036 exhibit some scatter (Fig. 4.55). There is a slight “dip” in the Demec reading for gage 1 at 58 days after casting. This is likely due to an error in reading the instrument. Four of the five electrical gages provided plausible data (Fig. 4.56). Electrical gages 1 and 5 measured slightly smaller strains that those recorded by the Demec gages.

The Demec strain gage readings for Specimen U8-0.0054 exhibit significant scatter from day 21 to day 110 (Fig. 4.57). After day 110, the readings are fairly stable. All five electrical gages provided meaningful data (Fig. 4.58). The average of these electrical-gage strain readings provided strains similar to those recorded by the Demec gages.

The Demec strain gage readings for Specimen U8-0.0072 exhibited relatively little scatter (Fig. 4.59). Four of the five electrical gages provided meaningful data, and the average of these electrical-gage readings was similar to strains recorded by the Demec gages (Fig. 4.60).

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ii.) 4,000 psi specimens

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

Reinf. ratio	Figures
0.00NS	4.61 - 4.63
0.0000	4.64 - 4.67
0.0036	4.68 - 4.69
0.0054	4.70 - 4.71
0.0072	4.72 - 4.73

The following is a brief description of the data collected for each specimen.

The Demec strain gages for Specimen U4-0.00NS-1 measured strains with reasonably little scatter (Fig. 4.61). Only one electrical gage, the central “floating” type was cast into this specimen, and it provided no reliable data.

Only Demec gage 4 was readable for Specimen U4-0.00NS-2 (Fig. 4.62). Only one electrical gage ,the “floating” type, was cast into this specimen. This gage measured slightly higher strains than the readable Demec gage (Fig. 4.63).

The Demec strain gage readings for Specimen U4-0.0000-1 exhibited some scatter (Fig. 4.64). Electrical gage 1, which is the “floating” type, provided strain data similar to strains measured with the Demec gages (Fig. 4.65). Electrical gage 2 was placed on the spiral reinforcement. This gage indicated slight tensile strains. This contradicts what is expected to occur due to shrinkage in the specimen.

The Demec strain gage readings for Specimen U4-0.0000-2 exhibited relatively little scatter (Fig. 4.66). Electrical gage 1, which is the “floating” type, provided unreliable data (Fig. 4.67). Electrical gage 2 was placed on the spiral reinforcement and indicated slight tensile strains. Again, this was not expected.

The Demec strain gage readings for Specimen U4-0.0036 exhibited little scatter (Fig. 4.68). There is a significant “dip” in the Demec readings for gage 1 at 58 days after casting. This is likely due to an error in reading the instrument. Two of the five electrical gages provided plausible data (Fig. 4.69). Electrical gage 4 measured strains similar to those recorded by the Demec gages. Electrical gage 1 is of the "floating" type and it measured somewhat smaller strains.

The Demec gage readings for Specimen U4-0.0054 exhibit very little scatter (Fig. 4.70). Three of the five electrical gages provided reliable data (Fig. 4.71). The average of these electrical gages produced strains similar to those recorded by the Demec gages.

The Demec strain gage readings for Specimen U4-0.0072 exhibited very little scatter (Fig. 4.72). Two of the five electrical gages provided plausible data (Fig. 4.73). Electrical gage 1, which is the "floating" type, provided readings similar to those of the Demec gages. Electrical gage 3 provided readings that were substantially smaller than those from gage 1 and also exhibited erratic measurements during the first 40 days after casting.

iii.) General discussion

The Demec gages provided what appear to be reliable data. Considering the small changes in strain which are being recorded, relatively little scatter is apparent in the results.

In the cases where the electrical gages provided useful data, these readings were generally in agreement with the strains recorded by the mechanical gages. Because the strain gages have been recording data since casting was performed, and measurements with the mechanical gages generally began approximately 15 days after casting, measured strains for the electrical gages were typically higher. When the electrical strain data from the first 15 days is subtracted, the electrical and mechanical gages show reasonable correlation.

Again, the electrical gages were significantly more reliable in Groups 1 and 3 than they were in Groups 2 and 4. The possible reasons for this have been discussed previously.

3. Comparison of Concentrically Loaded and Unloaded Specimens

For purposes of comparison, the preliminary results for the concentrically loaded and unloaded specimens are shown in Figures 4.74 through 4.79. The unloaded specimens without spiral reinforcement are not presented here because they have no loaded companion specimens. The data are presented as follows:

Design Concrete Strength	Group	Figure
8,000 psi	1 & 2	4.74
8,000 psi	1	4.75
8,000 psi	2	4.76
4,000 psi	3 & 4	4.77
4,000 psi	3	4.78
4,000 psi	4	4.79

The data have been plotted with the loading day of each specimen adjusted to correspond with day 0. This was done for both the loaded and unloaded specimens. The specimens of Groups 1 and 3 were cast prior to those of Groups 2 and 4, and therefore, significantly more data is available for Groups 1 and 3. The plots for Groups 2 and 4 are shown with dashed lines. As was mentioned previously, the data for all specimens have been adjusted for temperature effects.

The data plotted for each specimen are the averaged Demec data. These data appear to be substantially more reliable than the electrical strain gage data.

a) 8,000 psi Specimens

Figure 4.74 shows the preliminary test results for all of the 8,000 psi specimens. Most specimens in Group 1 were loaded for approximately 150 days, and most specimens of Group 2 were loaded for approximately 50 days. Much less scatter is apparent for the Group 1 results than for the Group 2 results. Differences in concrete used for the Group 2 specimens is manifested in the higher strains observed for every specimen of Group 2 compared to comparable specimens in Group 1.

Figure 4.75 presents the results for Group 1 specimens only. With the exception of Specimen C8-0.0072-2, the more lightly reinforced specimens exhibit slightly higher strains as expected. This is true for both the loaded and unloaded specimens.

Figure 4.76 presents the results for Group 2 specimens only. A significant amount of scatter in the results is noted. When examining the data, it is apparent that the initial elastic strain for the unreinforced specimen (C8-0.0000) is smaller than for the reinforced specimens in Group 2. This suggests that either the load was incorrectly applied or that an insufficient number of Demec gage readings are available for a reliable average. It is likely that the latter is largely responsible because Demec gage 1 was not set properly, and therefore, was not readable. It is likely that this gage would have increased the elastic strain average because it is located on the same side of the specimen as Demec gage 2. As evident in Figure 4.1, Demec gage 2 measured significantly higher strains than Demec gages 3 and 4. This is likely due to unintended eccentricity about the north-south axis in the loading frame.

In regard to the three reinforced concentrically loaded specimens of Group 2 (C8-0.0036-3, C8-0.0054-3 and C8-0.0072-3), the results are also not entirely as expected. The total strain for the specimen with a reinforcement ratio of 0.0054 is larger than that for the specimen with a reinforcement ratio of 0.0036. The specimen with a reinforcement ratio of 0.0072 experienced lower total strain than the more lightly reinforced specimens, as expected.

The unloaded specimen for Group 2 exhibited very little shrinkage strain compared to strains measured in the companion specimen in Group 1. This is possibly due to the higher humidity in the enclosure for the Group 2 specimens as compared to Group 1.

General information regarding the concentrically loaded 8,000 psi specimens is presented in Table 4.1. The table presents total strains for 40, 80 and 120 days after loading. When data were not available for the day indicated, linear interpolation was used between the data from adjacent dates on which data were recorded.

b) 4,000 psi Specimens

Figure 4.77 presents the preliminary results for all the 4,000 psi specimens. Most specimens in Group 3 were loaded for approximately 120 days, and most specimens of Group 4 were loaded for approximately 50 days. A reasonable scatter is evident in the results. Concrete material differences for Group 4 are manifested in the slightly higher strains for the Group 4 specimens than for Group 3.

Figure 4.78 presents the results for Group 3 specimens only. A limited amount of scatter is apparent in the measured strain data. With the exception of Specimen C4-0.0072-1 and U4-0.0054, the more lightly reinforced specimens exhibited slightly higher strains as expected.

Figure 4.79 presents the results for Group 4 specimens only. A limited amount of scatter is noticed in the strain data. The amount appears comparable to that observed for the Group 3 specimens at the same stage of response. The strain data for Specimens C4-0.0000, C4-0.0036-3 and C4-0.0054-3 are tightly grouped. The specimen with the highest reinforcement ratio (C4-0.0072-3) experienced slightly higher strains than the other specimens. This may be due to over-application of load or an inadequate number of Demec readings (one Demec gage was unreadable).

General information regarding the concentrically loaded 4,000 psi specimens is presented in Table 4.2. As for Table 4.1, results for 40, 80 and 120 days after loading are presented. As for Table 4.1, linear interpolation was used where necessary.

c) General Discussion

The differences in the concrete strength and resulting differences in concrete modulii for the different groups of specimens are noticeable in the test results.

Shrinkage strains are higher for the 4,000 psi specimens than for the 8,000 psi specimens.

Elastic strains are lower for the 4,000 psi specimens than for the 8,000 psi specimens. This is due to the magnitude of load applied, which is a function of the design compressive strength of the specimen.

Groups 1, 3 and 4 exhibited relatively little scatter in the strain results compared to Group 2. This may be due to a number of reasons, which include an inadequate number of readable Demec gages for a reliable average, incorrect application of the load, and incorrect Demec readings.

It was initially feared that certain specimens of Groups 2 and 4 may have been mislabeled. This was found to be unlikely due to the following reasons: specimens with reinforcement ratios of 0.0000 and 0.0036 have one and five embedded electrical gages, respectively. The specimens with reinforcement ratios of 0.0054 and 0.0072 both have seven embedded electrical gages. Because wires attached to the gages are exposed, it is possible to confuse the specimens with reinforcement ratios of 0.0054 and 0.0072, but it is not possible to confuse these specimens with those that have reinforcement ratios of 0.0000 or 0.0036. Furthermore, the specimens with ratios of 0.0000 and 0.0036 have a unique number of gages (and lead wires) and therefore cannot be confused with each other.

Groups 2 and 4 have been loaded for a brief period of time (approximately 50 days). As more data are collected, the general trends may shed more light on what now appear to be anomalies in the data.

4.4 Concrete Cylinders

The average strengths of concrete cylinders are reported in Table 4.3 for 14, 28 and 56 days after casting.

TABLE 4.3 Strength Data for Cylinders Cast With Column Specimens

		Small Cylinders, (4 inch x 6 inch)			Large Cylinders, (6 inch x 12 inch)		
Group	Design Strength 28 days, psi	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	-	6,524	6,918	-
3	4,000	5,060	5,502	5,486	4,896	5,389	5,661
4	4,000	4,117	4,443	-	4,222	4,461	-

The average modulii of elasticity for each of the concrete mixes at 14, 28 and 56 days after casting are listed in Table 4.4. Modulus of elasticity was determined with an extensometer.

TABLE 4.4 Modulus Data for Cylinders Cast With Column Specimens

		Small Cylinders, (4 inch x 6 inch)			Large Cylinders, (6 inch x 12 inch)		
Group	Predicted M.O.E. 28 days, ksi	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	5,434	5,088	5,202	-	5,401	5,417
2	5,098	4,963	4,659	-	4,888	5,103	-
3	3,605	4,044	4,246	3,045	4,239	4,257	4,161
4	3,605	3,593	3,531	-	3,918	3,676	-

These concrete strength and modulus data are presented graphically in Figures 4.80 through 4.83.

4.5 Reinforcing Steel

The stress-strain relationship for the 6 mm diameter (referred to as No. 2) bars used for longitudinal reinforcement in the column specimens is shown in Fig. 4.84. Four eighteen-inch long segments of the reinforcing steel were tensile tested in a 60-kip capacity Tinius-Olsen universal test machine to establish the yield and ultimate stresses of the steel. The yield stress was determined by observing a plot of the load versus deformation curve and recording the load at the departure from linearity. This load was then divided by the cross-sectional area of the bars (0.046 square inches).

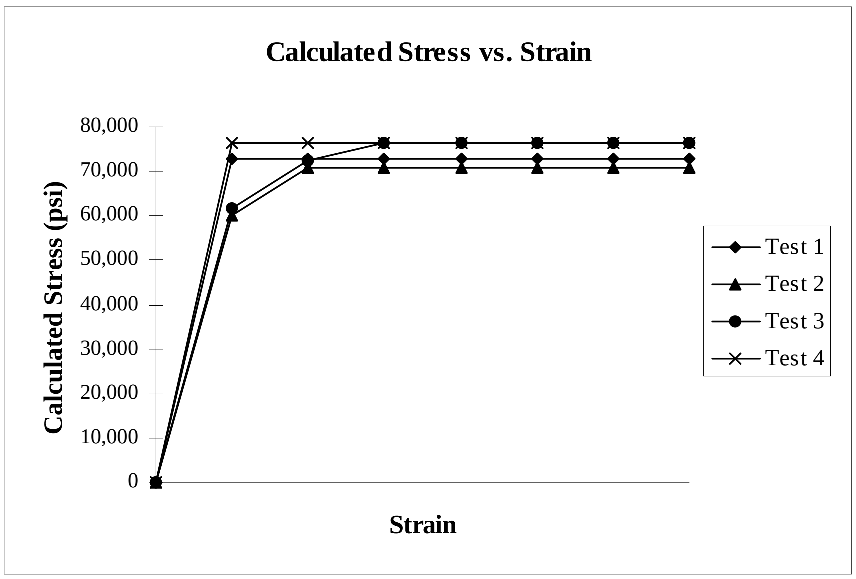

FIGURE 4.84 Stress-Strain Relationship for 6 mm Diameter (No. 2) Reinforcing Steel

The calculated yield and ultimate stress for each test was as follows:

Test number	Yield Stress (psi)	Ultimate Stress (psi)
1	73,000	73,000
2	60,000	71,000
3	62,000	76,000
4	76,000	76,000

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

5.1 Introduction

This chapter summarizes ACI 209R-86[12] entitled "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures". This report was first published in 1982 and was re-approved in 1986. The method described is applied to the test specimens, then the experimental test results are compared with those predicted by the ACI 209 procedure. Where the ACI equations are presented they are followed by the equation number given in ACI-209 (e.g. ACI 2-8).

5.2 Summary of ACI 209R-86 Procedure

ACI 209 presents equations for predicting creep, shrinkage, and temperature effects that are applicable to both moist and steam-cured concrete. This thesis considers only moist-cured concrete, and therefore, the equations for steam-cured concrete are not given. All equations presented in ACI-209 refer to plain concrete.

The recommended creep and shrinkage equations for standard conditions are as follows:

The basic equation for the prediction of creep is:

$$\nu_t = \frac{t^{0.60}}{(10+t)^{0.60}} \nu_u \qquad \text{(ACI 2-8)}$$

where

ν_t = creep coefficient for time after loading

t = time in days after loading.

ν_u = ultimate (with time) creep coefficient
(normal range = 1.30 to 4.15)

The basic equation for the prediction of shrinkage is:

$$(\varepsilon_{sh})_t = \frac{t}{(35+t)} (\varepsilon_{sh})_u \qquad \text{(ACI 2-9)}$$

where

$(\varepsilon_{sh})_t$ = shrinkage after 7 days

t = time after the end of initial moist curing.

$(\varepsilon_{sh})_u$ = ultimate (with time) shrinkage strain
(normal range = 415 to 1070 micro-strain)

The values of ν_u and $(\varepsilon_{sh})_u$ must be modified by correction factors for conditions that are other than standard. The average values suggested for ν_u and $(\varepsilon_{sh})_u$ are:

ν_u = $2.35\gamma_c$ and

$(\varepsilon_{sh})_u$ = $780\gamma_{sh} \times 10^{-6}$

where γ_c and γ_{sh} represent the product of the applicable correction factors for other than standard conditions. The correction factors apply to loading age, differential shrinkage, period of initial moist curing, ambient relative humidity, average thickness of member or volume-to-surface area ratio of member, ambient temperature, slump, fine aggregate percentage, cement content, and air content. These correction factors are discussed individually below.

Loading Age:

For moist-cured concrete with a loading age of later than 7 days the creep correction factor is:

$$\text{Creep } \gamma_{la} = 1.25(t_{la})^{-0.118} \qquad \text{(ACI 2-11)}$$

where t_{la} is the age at loading in days.

Differential Shrinkage:

Differential shrinkage is computed by subtracting the shrinkage estimated for the period from 7 days to the starting date of the time interval in question from the shrinkage estimated for the period from 7 days to the ending date of the time period in question.

For example, the shrinkage strain between 28 days and 1 year would be estimated as the 7-day to 1-year shrinkage minus the 7-day to 28-day shrinkage.

Initial Moist Curing:

For shrinkage of concrete moist-cured for a period of time other than 7 days, shrinkage factors γ_{cp} are given in Table 5.1. Linear interpolation may be used between the values given.

TABLE 5.1 ACI 209R-86 Shrinkage Factors

Moist curing duration, days	Shrinkage γ_{cp}
1	1.2
3	1.1
7	1.0
14	0.93
28	0.86
90	0.75

Ambient Relative Humidity:

For ambient relative humidity greater than 40 percent, the following creep and shrinkage correction factors apply:

Creep $\gamma_\lambda = 1.27 - 0.0067\lambda$, for $\lambda > 40$ (ACI 2-14)

Shrinkage $\gamma_\lambda = 1.40 - 0.010\lambda$, for $40 \leq \lambda \leq 80$ (ACI 2-15)

$= 3.00 - 0.030\ \lambda$, for $80 > \lambda \leq 100$ (ACI 2-16)

where λ is ambient relative humidity in percent.

If ambient relative humidity is less than 40 percent, then γ_λ shall be greater than 1.0. The average relative humidity in the enclosures for all four groups of specimens was between 37 and 39 percent. Therefore, for the purposes of this thesis, a γ_λ value of 1.0 was used for all cases.

<u>Average thickness (if volume-surface area ratio method not used):</u>

During the first year after loading:

Creep $\gamma_h = 1.14 - 0.023\ h$, (ACI 2-17)

For ultimate values:

Creep $\gamma_h = 1.10 - 0.017\ h$, (ACI 2-18)

During the first year of drying:

Shrinkage $\gamma_h = 1.23 - 0.038\ h$, (ACI 2-19)

For ultimate values:

Shrinkage $\gamma_h = 1.17 - 0.029\ h$, (ACI 2-20)

where h is the average thickness of the part or member under consideration.

Volume-to-Surface Area Ratio Method (if average thickness method not used):

Creep $\gamma_h = (2/3) * [1+1.13\exp(-0.54\ v/s)]$ (ACI 2-21)

Shrinkage $\gamma_{vs} = 1.2\exp(-0.12\ v/s)$ (ACI 2-22)

where v/s is the volume-to-surface area ratio of the member in inches.

For either method, γ_{sh} should not be taken less than 0.2.

Temperature other than 70 degrees Fahrenheit:

Temperature is the second major factor affecting creep and shrinkage. Humidity is generally considered to be more important due to the small range of operating temperatures for most structures.

At 122 degrees Fahrenheit, creep strain is approximately two to three times the creep strain at 68-75 degrees Fahrenheit. From 122 to 212 degrees Fahrenheit creep continues to increase with temperature, reaching four to six times that experienced at room temperatures.

For the purpose of this thesis, it was assumed that creep strain at 122 degrees Fahrenheit is 2.5 times the creep strain at 70 degrees Fahrenheit. Linear interpolation was used between these values. The maximum and minimum temperature values, as recorded for each group of specimens, were averaged, and the average temperature value for the entire loading period was determined. This average temperature value was used to determine the creep correction factor. It is noted that the temperature values used were ambient, as opposed to being recorded in the concrete specimens.

Slump:

Creep $\gamma_s = 0.82 + 0.067s$ (ACI 2-23)

Shrinkage $\gamma_s = 0.89 + 0.041s$ (ACI 2-24)

where s is the observed slump in inches.

Fine Aggregate Percentage:

Creep $\gamma_\Psi = 0.880 + 0024\psi$ (ACI 2-25)

For $\psi \leq 50\%$

Shrinkage $\gamma_\Psi = 0.30 + 0.014\psi$ (ACI 2-26)

For $\psi > 50\%$

Shrinkage $\gamma_\Psi = 0.90 + 0.002\psi$ (ACI 2-27)

where ψ is the ratio of the fine aggregate to total aggregate by weight expressed as a percentage.

Cement Content:

Shrinkage $\gamma_c = 0.75 + 0.00036c$ (ACI 2-28)

where c is the cement content in pounds per cubic yard.

Air Content:

Creep $\gamma_\alpha = 0.46 + 0.09\alpha$, (ACI 2-29)

but not less than 1.0

Shrinkage $\gamma_\alpha = 0.95 + 0.008\alpha$ (ACI 2-30)

where α is the air content in percent.

These correction factors were determined for each loaded column specimen and its companion unloaded specimen. These correction factors and the data from which they were derived are presented in Table 5.2 for the 8,000 psi specimens and in Table 5.3 for the 4,000 psi specimens. The sum of these factors and the resulting corrected values for ultimate creep (ν_u) and shrinkage $\{(\varepsilon_{sh})_u\}$ are also presented in these tables.

5.3 Comparison of Predicted and Experimental Results

The equations for creep and shrinkage discussed above were applied to the specimens, and the predicted results are plotted with the experimental results. For clarity, only one specimen per group for a particular reinforcement ratio (0.0036, 0.0054 or 0.0072) is presented and compared with the predicted results. Results are shown for each of the four groups of specimens because the environmental conditions and concrete composition varied significantly from group to group.

The predicted initial strains due to axial load were computed using the transformed section. The 28-day modulus for the concrete was obtained from the compressive-strength data for the average of three 6 x 12 inch cylinders. The value used for the 28-day modulus of concrete was 57,000 * $(f_c')^{0.5}$, and the modulus used for the reinforcing steel was 29,000 ksi. The initial strain calculation based on the transformed section is as follows:

$$\varepsilon_{initial} = \frac{P}{[A_g(1-\rho_g) + n\rho_g A_g] \cdot E_{ci}} \qquad \text{(Eqn. 5-1)}$$

where

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

P = applied axial load

A_g = gross cross-sectional area of concrete column

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

A_{st} = total area of longitudinal reinforcing steel

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

E_{st} = modulus of steel reinforcement

ρ_g = A_{st}/A_g

The effective modulus was used in conjunction with the transformed section to predict the strains due to creep and load. In this approach, the effective concrete modulus is simply substituted for the initial concrete modulus when computing strains. The calculation for the effective modulus is as follows:

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

where

$E_{eff.}$ = Effective modulus of concrete at time after loading considered

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

ν_t = creep coefficient at time t

t = time after load (in days)

therefore

$$\varepsilon_{initial} + (\varepsilon_{creep})_t = \frac{P}{[A_g(1-\rho_g)+A_g\rho_g n_{eff.}]\cdot E_{eff.}} \qquad \text{(Eqn. 5-2)}$$

where all values are as before except:

$(\varepsilon_{creep})_t$ = strain in reinforced specimen due to creep at time after loading considered

$n_{eff.}$ = modular ratio at time after loading considered (E_{st} / $E_{eff.}$)

Shrinkage strains were obtained by applying the resisting force in the longitudinal steel to the transformed area of the concrete column specimen. The resisting force due to the steel as the concrete attempts to shrink is computed as:

$$P_{resisting} = (\varepsilon_{sh})_t\, E_{st}\, A_g\, \rho_g \qquad \text{(Eqn. 5-3)}$$

where all terms are as before except:

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

Applying this resisting force to the transformed column section results in the following equation for strain due to shrinkage:

$$(\varepsilon_{shrinkage})_t = (\varepsilon_{sh})_t - \left\{\frac{(\varepsilon_{sh})_t E_{st} A_g \rho_g}{[A_g(1-\rho_g) + A_g \rho_g n_{eff.}] \cdot E_{eff.}}\right\} \qquad \text{(Eqn. 5-4)}$$

Total strain is obtained by summing the initial, creep, and shrinkage strains.

$$(\varepsilon_{total})_t = [\varepsilon_{initial} + (\varepsilon_{creep})_t] + (\varepsilon_{shrinkage})_t \qquad \text{(Eqn. 5-5)}$$

The results are presented graphically in the following figures:

Design Concrete Strength	Group	Figure
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 topmost curves indicate total strains (initial + creep + shrinkage) for the reinforced concrete specimens. Shrinkage strains for the specimens are shown in the lower curves. The values predicted using ACI-209 are shown as dashed lines. The curves are plotted beginning at the time of load application, which was generally between 14 and 28 days after casting. Shrinkage which occurred between the end of curing (5 days) and time of loading (14 to 28 days) is not shown in these curves. The purpose of these strain versus time curves is to compare ACI 209 predicted response with experimental results. Initial shrinkage strains were relatively insignificant. Differential shrinkage from time of load application to time desired was obtained as discussed earlier.

By substituting unity for the time-dependent portion of the ACI equations for ν_t (ACI 2-8) and $(\varepsilon_{sh})_t$ (ACI 2-9), the equations for ultimate creep and shrinkage values for the plain concrete are obtained.

$$\nu_t = [\ 1.0\ ]\ \nu_u \qquad \text{(modified ACI 2-8)}$$

$$(\varepsilon_{sh})_t = [\ 1.0\ ]\ (\varepsilon_{sh})_u \qquad \text{(modified ACI 2-9)}$$

where all terms are as previously defined. For purposes of review:

ν_u = ultimate (with time) creep coefficient
(normal range = 1.30 to 4.15)

$(\varepsilon_{sh})_u$ = ultimate (with time) shrinkage strain
(normal range = 415 to 1070 micro-strain)

ν_u = $2.35\gamma_c$

$(\varepsilon_{sh})_u$ = $780\gamma_{sh} \times 10^{-6}$ inch/inch

γ_c = product of applicable creep correction factors for other than standard conditions

γ_{sh} = product of applicable shrinkage correction factors for other than standard conditions

By substituting ν_u and $(\varepsilon_{sh})_u$ for ν_t and $(\varepsilon_{sh})_t$ in the appropriate equations for predicting creep and shrinkage strains of the reinforced specimens, it is possible to estimate the ultimate creep and shrinkage strains. The resulting expressions are:

$$\varepsilon_{initial} + (\varepsilon_{creep})_{ult.} = \frac{P}{[A_g(1-\rho_g) + A_g\rho_g n_{e-ult.}]\cdot E_{e-ult.}} \quad \text{(Eqn. 5-6)}$$

$$(\varepsilon_{shrinkage})_{ult.} = (\varepsilon_{sh})_u - \frac{[(\varepsilon_{sh})_u E_{st} A_g \rho_g]}{[A_g(1-\rho_g) + A_g\rho_g n_{eff.}]\cdot E_{eff.}} \quad \text{(Eqn. 5-7)}$$

where all terms are as before except:

$E_{e\text{-}ult.}$ = ultimate effective modulus = $E_{ci} / (1 + \nu_u)$

$n_{e\text{-}ult.}$ = ultimate modular ratio = $E_{st} / E_{e\text{-}ult.}$

$(\varepsilon_{creep})_{ult.}$ = ultimate creep strain in reinforced concrete specimen

$(\varepsilon_{shrinkage})_{ult.}$ = ultimate shrinkage strain in reinforced concrete specimen

The total strain in the reinforced concrete specimen can then be expressed as:

$$(\varepsilon_{total})_{ult.} = \varepsilon_{initial} + (\varepsilon_{creep})_{ult.} + (\varepsilon_{shrinkage})_{ult.} \quad \text{(Eqn. 5-8)}$$

Values of the predicted initial and ultimate strains for each type of column specimen considered in this study are presented in Tables 5.6 and 5.7. For the unloaded specimens, initial and creep strains are equal to zero. Therefore, the predicted ultimate strain for the unloaded reinforced specimens is simply $(\varepsilon_{shrinkage})_{ult.}$.

5.4 General Discussion

a) 8,000 psi Specimens

The total strains obtained experimentally for select Group 1 specimens (Figure 5.1) are in reasonable agreement with the total strains predicted by ACI-209. The predicted shrinkage strains are only slightly higher than the highest strains obtained experimentally. The predicted total strains are initially slightly higher than the experimental strains and are slightly lower than the maximum experimental total strains at the end of the data collected thus far. A noticeable jump is apparent in the total strain results for Specimen C8-0.0036-1. This jump corresponds with the day on which the Demec gage was dropped. Subsequent readings for all specimens were adjusted for a new zero reading, and only minor differences in readings were noted for most specimens. However, note that the response of Specimen C8-0.0036-1 would be closer to the predicted response if the measured response after the jump was shifted downward by the amount of the jump. Furthermore, load in the specimens in this group was adjusted upward at approximately 62 days after initial loading. This is evident as a slight “upturn” in strain response at that time. Additionally, the experimental results were adjusted for the average ambient temperature recorded on the day the gages were read. The ambient temperature was likely higher than the actual temperature in the concrete specimen, which likely resulted in slight under-estimation of the total strains.

The total strains obtained experimentally for Group 2 (Figure 5.2) exhibit significant scatter, but are in general agreement with the total strains predicted by ACI-209. The predicted shrinkage strains are only slightly higher than those obtained experimentally from 0 to 20 days after loading. Beyond 20 days, however, the shrinkage strains for Specimen U8-0.0000-2 (which is the only unloaded, spirally-reinforced specimen in this group) are well below the predicted values. It is speculated that this may be due to the higher average relative humidity during this period. The reduction in shrinkage strain appears to be reflected in the total strain results for the loaded specimens. Load in these specimens was adjusted at approximately 44 days after initial loading. The effect of reloading is noticeable to varying degrees in the results.

b) 4,000 psi Specimens

The total strains obtained experimentally for Group 3 (Figure 5.3) exhibit reasonable agreement with the total strains predicted by ACI-209. The predicted shrinkage strains are only slightly higher than those obtained experimentally. The specimens in this group were generally reloaded at 59 days after initial loading. The effects of reloading are evident in the measured strain responses as a significant increase in strains for all loaded specimens. A positive temperature differential of 14 degrees Fahrenheit exists between the readings taken at 41 days and those taken at 59 days after initial loading. As mentioned earlier, the adjustment made for temperature differentials likely resulted in a slight under-estimation of total strains.

Total strains obtained experimentally for Group 4 (Figure 5.4) are less than the total strains predicted by ACI-209. Predicted shrinkage strains are higher than those obtained experimentally from 0 to 20 days after initial loading. Beyond 20 days the shrinkage strains for Specimen U4-0.0000-2 (which is the only unloaded and spirally-reinforced specimen in this group) fall well below the predicted values. Again, it is speculated this may be due to the increase in average relative humidity during this time period. The reduction in shrinkage strain appears to be reflected in the total strain results for the loaded specimens. It is significant that the unloaded specimen in this group demonstrated a very similar strain history to the unloaded specimen of Group 2. These groups (Groups 2 and 4) were cast on the same date (May 15, 1996). Specimens in this group were generally reloaded 37 days after initial loading. Effects of reloading are discernible in some instances.

Chapter 6. SUMMARY AND CONCLUSIONS

6.1 Introduction

The objective of this investigation is to study the long-term response of concrete columns reinforced with less than the code-required minimum reinforcement percentage of 1.0% of the gross concrete area. Columns in this study have been loaded for less than nine months, and therefore, the results presented and any accompanying discussion should be viewed as preliminary. An experimental investigation has been underway and the preliminary results of this investigation were compared to responses predicted by the analytical method reported by ACI Committee 209R-86[12].

6.2 Experimental Investigation

A total of 38 concrete columns were cast and are currently being investigated. Each column has a nominal cross-sectional diameter of 8 inches and is 4 feet 0 inches long. Of the 38 columns, 24 are under sustained load equal to $0.40*f_c'*A_g$ (with one exception). The load is being maintained with heavy coil springs. All columns were loaded between 14 and 28 days after casting. Ambient humidity has been reduced as much as practical. The following variables are being investigated:

1. Concrete Strength

   Concrete design strengths (at 28 days) 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 equal to 0.10*column diameter.

6.3 Comparison of Experimental Results with Predicted Analytical Results

In Chapter 5, results obtained experimentally were plotted with the results predicted by the ACI 209 analytical procedure. This was done for the majority of the specimens. The loading age, average amibient humidity, average ambient temperature, concrete strength and other important parameters were considered for each specimen, and appropriate factors were used in the ACI 209 procedure. In most cases, good agreement was found between the results predicted by ACI 209 and the experimental results.

The effect of temperature is apparent in the experimental and analytical results. Specimens which were cast in the summer months (Groups 2 and 4) exhibit higher rates of creep than specimens which were cast in the winter and spring months (Groups 1 and 3). Temperature effects are considered in the ACI 209 procedure. ACI 209 notes that creep strains at 122 degrees Fahrenheit are roughly two to three times those at 68-75 degrees Fahrenheit. However, no creep coefficient is directly suggested for temperatures exceeding 70 degrees Fahrenheit. Because temperature had an apparent impact in the experimental results, it was accounted for in the application of the ACI 209 procedure. For purposes of this thesis, it was assumed that creep strains are 2.5 times as great at 122 degrees Fahrenheit as at 70 degrees Fahrenheit, and that linear interpolation can be used between these temperature values. As mentioned earlier, good correlation between the experimental and analytical results was observed.

6.4 Predictions of Future Behavior

As mentioned in Chapter 1, values of the ultimate creep coefficient (ν_u) and ultimate shrinkage strain [$(\varepsilon_{sh})_u$] have upper-bounds of 4.35 and 1070, respectively, for other than standard conditions (as defined in ACI 209). These upper-bound values for creep and shrinkage are applied to an 8,000 psi specimen with a reinforcement percentage of 0.72%. A loading age of 14 days after casting is assumed. In Figure 6.1, the predicted total strains for this specimen over a loading period of two years are shown together with the predicted total strains for Specimens C8-0.0072-1 and C8-0.0072-3. This procedure is carried out similarly for a 4,000 psi specimen with reinforcement percentage of 0.72%. The results for this case are shown in Figure 6.2.

When examining Figures 6.1 and 6.2, it is apparent that the specimens with higher-strength concrete are predicted to experience larger total strains than the lower-strength specimens. As discussed previously, this is due to the loading algorithm. It is also apparent that temperature has a significant impact on creep (and therefore total) strains. This can be seen by comparing C8-0.0072-1 and C4-0.0072-1 to C8-0.0072-3 and C4-0.0072-3. The C8-0.0072-3 and C4-0.0072-3 specimens were cast in the summer months and indicate much higher total strains are expected than for their companion specimens (C8-0.0072-1 and C4-0.0072-1) which were cast in the winter and spring. The lower concrete strengths of the C8-0.0072-3 and C4-0.0072-3 specimens is contributing to this behavior as well. The predicted total strains of the C8-0.0072-3 and C4-0.0072-3 specimens are very close to those predicted for the upper-bound values mentioned in ACI 209. The values used for ultimate creep coefficient and ultimate shrinkage and the season when cast are shown below.

Specimen	Season when cast	ν_u	$(\varepsilon_{sh})_u$
ACI-C8-0.0072 (upper-bound)	N/A	4.35	1070
ACI-C8-0.0072-1	Winter	2.83	829
ACI-C8-0.0072-3	Summer	3.74	801
ACI-C4-0.0072 (upper-bound)	N/A	4.35	1070
ACI-C4-0.0072-1	Spring	2.91	936
ACI-C4-0.0072-3	Summer	4.16	920

It is noted that the predicted behavior is based on the assumption that the average temperature will remain fairly constant over time. This will most likely not be the case due to seasonal variations. It is also noted that shrinkage strains prior to loading are (unconservatively) neglected.

When comparing the predicted total strains with the steel yield strain, it is projected that both 8 ksi specimens will reach the yield strain of the reinforcing steel prior to two years of loading. One of the 4 ksi specimens (cast in the summer) is projected to reach the steel yield strain prior to two years of loading.

6.5 Conclusions

The experimental investigation is not yet complete. However, some preliminary conclusions can be drawn based on data gathered to this point.

1) The enviromental conditions in which the columns are cast and maintained under load have a very significant effect on long-term deformations; temperature and humidity are particularly important.

2) The procedure described by ACI 209 predicts the behavior of the experimental columns with what appears to be acceptable accuracy.

3) Temperature was accounted for when applying the ACI 209 procedure. The inclusion of temperature effects was necessary to achieve good predictions of specimen behavior.

4) Decreasing live-to-dead load ratios will increase the percentage of longitudinal steel required to prevent passive yielding.

5) Increasing concrete strengths will increase the percentage of longitudinal steel required to prevent passive yielding (due to the loading algorithm as previouly mentioned).

6) If the environment is such that average values (as suggested by ACI 209) for “standard conditions” can be used for the ultimate creep coefficient and ultimate shrinkage [ν_u = 2.35 and $(\varepsilon_{sh})_u$ = 780], then the minimum steel percentage could be

reduced for a significant number of L/D ratios and concrete strengths. This is shown in Table 1.1 of Chapter 1, and is reproduced here for convenience as Table 6.1 for convenience. The shaded portions of the table indicate combinations of concrete strengths and live-to-dead load ratios in which the minimum reinforcement percentage could be reduced below 1.0%.

TABLE 6.1 Minimum % of Longitudinal Reinforcement
(for ν_u = 2.35 and $(\varepsilon_{sh})_u = 800 \times 10^{-6}$)

		L/D							
f'c	**fy**	**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

7) If the environment is such that upper-bound values (as suggested by ACI 209) for "other than standard conditions" must be used for the ultimate creep coefficient and ultimate shrinkage [ν_u = 4.15 and $(\varepsilon_{sh})_u$ = 1070], then the number of cases in which the required percentage of reinforcing steel is below 1.0% is significantly reduced. This is shown in Table 1.2 of Chapter 1 and is reproduced here for convenience as Table 6.2. The shaded portions of the table indicate instances in which the minimum reinforcement percentage could be reduced below 1.0%.

TABLE 6.2 Minimum % of Longitudinal Reinforcement (for ν_u = 4.15 and $(\varepsilon_{sh})_u$ = 1070 x 10^{-6})

		L/D							
f'c	**fy**	**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

8) Due to conclusions 6 and 7, it appears that a straight-forward reduction in percentage of longitudinal reinforcing steel is not warranted. Rather, an equation (or other method like a step-function) could be developed which incorporates the ultimate creep coefficient, ultimate shrinkage, L/D ratio, and concrete strength.

9) As of July 24, 1996 (the last reading considered for this thesis), the compression steel in all four eccentrically-loaded specimens had either reached or very nearly reached strains corresponding to nominal yield of the reinforcement. At the time this thesis was completed, strains in the compression steel were beyond actual yield strain. The columns have not failed, but significant curvature deformations are obvious.

6.6 Further Research

1) A function (or other method) for minimum reinforcing steel could be developed. This equation would most likely be based on the expected ultimate creep coefficient, ultimate shrinkage, L/D ratio, and concrete strength.

2) A method other than ACI 209 for predicting creep and shrinkage effects should also be applied to the specimens to evaluate the accuracy of the method. Ideally, the alternate method would include temperature effects directly and would account for changes in humidity and temperature over time.

3) The analytical portion of this thesis has focused almost exclusively on the concentrically-loaded specimens. The eccentrically-loaded specimens could be investigated further through additional experimental work and/or analytical methods.

BIBLIOGRAPHY

1. Raymond E. Davis and Harmer E. Davis, “Flow of Concrete Under the Action of Sustained Loads”, Journal of the American Concrete Institute, Vol. 2 - No. 7, Proceedings Vol. 27, p. 837, March 1931.

2. F. E. Richart and G. C. Staehle, “Second Progress Report on Column Tests at the University of Illinois”, Journal of the ACI, Vol. 2 - No. 7, Proceedings Vol. 27, p. 761, March 1931.

3. W. A. Slater and Inge Lyse, “Second Progress Report on Column Tests Made at Lehigh University”, Journal of the ACI, Vol. 2 - No. 7, Proceedings Vol. 27, p. 791, March 1931.

4. F. E. Richart and G. C. Staehle, “Fourth Progress Report on Column Tests at the University of Illinois”, Journal of the ACI, Vol. 3 - No. 5, Proceedings Vol. 28, p. 279, January 1932.

5. Inge Lyse and C. L. Kreidler, “Fourth Progress Report on Column Tests Made at Lehigh University”, Journal of the ACI, Vol. 3 - No. 5, Proceedings Vol. 28, p. 317, January 1932.

6. Inge Lyse, “Fifth Progress Report on Column Tests Made at Lehigh University”, Journal of the ACI, Vol. 4 - No. 9, Proceedings Vol. 29, p. 433, June 1933.

7. F. E. Richart (Chairman), R. L. Bertin and Inge Lyse, “ ‘Reinforced Concrete Column Investigation’ - Tentative Final Report of Committee 105, and Minority Recommendations for Design Formula of Reinforced Concrete Columns”, Journal of the ACI, Vol. 4 - No. 6, Proceedings Vol. 29, February 1933.

8. R. T. Logeman, Walter H. Wheeler, L. J. Mensch and J. Di Stasio, “Discussion of Report of Committee 105: ‘Reinforced Concrete Column Investigation’- Tentative Final Report of Committee 105 and Minority Recomendations for Design Formulas of Reinforced Concrete Columns”, Journal of the ACI, Proceedings Vol. 30, p. 78, Sept. - Oct. 1933.

9. F. E. Richart (Chairman), “Discussion of Report of Committee 105: ‘Reinforced Concrete Column Investigation’ - Closure by Chairman, Committee 105”, Journal of the ACI, Proceedings Vol. 30, p. 153, Nov. - Dec. 1933.

10. ACI 318-35, Committee 318, “Building Code Requirements for Reinforced Concrete and Commentary”, (ACI 318-35), ACI, Detroit, Michigan, 1935.

11. ACI Committe 318, “Building Code Requirements for Reinforced Concrete and Commentary”, (ACI 318-95), ACI, Detroit, Michigan, 1995.

12. ACI 209-R86, “Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures”, ACI, Detroit, Michigan, 1986.

13. S. Mindess and J. F. Young, “Concrete”, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1981.

14. R. Green, “Behavior of Unrestrained Reinforced concrete columns Under Sustained Load”, Dissertation, The University of Texas at Austin, January 1966.

15. C. H. Lin and R. W. Furlong, “Longitudinal Steel Limits for Concrete Columns”, ACI Structural Journal, Vol. 92 - No. 3, p. 282, May - June 1995.

16. F. R. McMillan, “A Study of Column Test Data”, Journal of the American Concrete Institute, Proceedings Vol. 17, p. 150, 1921.

17. W. H. Glanville, “Shrinkage Stresses” and “The Creep or Flow of Concrete Under Load”, Technical Papers No. 11 and 12, Building Research Technical Papers, Dept. of Industrial Research, England.