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16. Abstract

Existing minimum longitudinal reinforcement requirements for columns were developed to prevent passive yielding of reinforcement resulting from creep deformations in the concrete. Tests used to support this limit were conducted decades ago when steel yield strengths for reinforcing bars were approximately half of what is common today. The design of a very large percentage of columns in TxDOT bridges is typically controlled by minimum eccentricity of loads, implying a substantial reduction in column steel and resulting economic savings might be possible with present-day materials.

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Evaluation of Minimum Longitudinal Reinforcement Requirements for Reinforced Concrete Columns

by

Paul H. Ziehl, Jeffrey E. Cloyd, and Michael E. Kreger

Research Report 1473-S

Research Project 0-1473

Conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

in cooperation with the

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration

by the

Center for Transportation Research
Bureau of Engineering Research
The University of Texas at Austin

October 1998

DISCLAIMERS

IMPLEMENTATION

This research study was conducted with the intent of reducing the required minimum longitudinal reinforcement ratio for columns subjected to axial load in combination with minimal bending moment. However, although simple concrete creep models suggest that longitudinal reinforcement ratios reduced below the current 1 percent requirement will prevent passive yielding of longitudinal reinforcement, results of the long-term creep tests reported herein do not support this hypothesis. Therefore, the current AASHTO requirement for minimum longitudinal reinforcement in columns should remain at 1 percent of gross-section area.

ACKNOWLEDGEMENTS

The research study reported herein involved the long-term monitoring of reinforced concrete columns under sustained axial load. The report is based on portions of theses published at The University of Texas by Paul H. Ziehl and Jeffrey E. Cloyd. In addition to these co-authors of the report, the authors are indebted to a number of other graduate students and undergraduate students who assisted with fabrication of testing apparatuses and specimens, and loading of the specimens. The overall project was greatly enhanced by the technical assistance provided during the planning stages of the testing program by Mr. David W. McDonnold (TxDOT Project Director) of the TxDOT Design Division.

SUMMARY

Existing minimum longitudinal reinforcement requirements for columns were developed to prevent passive yielding of reinforcement resulting from creep deformations in the concrete. Tests used to support this limit were conducted decades ago when steel yield strengths for reinforcing bars were approximately half of what is common today. The design of a very large percentage of columns in TxDOT bridges is typically controlled by minimum eccentricity of loads, implying a substantial reduction in column steel and resulting economic savings might be possible with present-day materials.

Twenty-four reinforced concrete column specimens were cast and subjected to a sustained axial load of $0.4f_c'A_g$. Long-term axial deformations of the column specimens were monitored using electronic and mechanical strain gages. An additional 14 unloaded specimens were cast to monitor temperature and shrinkage-related deformations. All specimens were housed in reduced-humidity enclosures. Test variables included nominal concrete compressive strengths of 4000 and 8000 psi, longitudinal reinforcement ratios of 0.36, 0.54, and 0.72 percent, and eccentricity of axial load equal to zero or 0.10 times the column diameter. Plots of measured strain versus time are presented for all specimens, and experimental results are compared with an analytical model reported by ACI Committee 209. Final recommendations for column longitudinal reinforcement are presented.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Since the 1930's a minimum reinforcement ratio of 1 percent (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 percent 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 strength was in the range of 39 to 54 ksi. The 1 percent 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 possible that 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 percent. However, before such a change is made, it is prudent to verify this limit with a comprehensive experimental study.

Because a substantial percentage of all bridge piers require less than the minimum 1 percent 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 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 placing fewer longitudinal bars and the modest added benefit of reduced congestion in piers.

1.2 OBJECTIVE AND SCOPE OF THIS INVESTIGATION

The objective of this investigation was to determine the behavior of reinforced concrete columns which are reinforced with less than the current code-required minimum longitudinal reinforcement ratio of 1.0 percent (AASHTO Standard Specifications for Highway Bridges[18] Section 8.18.1.2 and ACI 318-95[11] Section 10.9.1). It was hoped that this lower limit of 1.0 percent could be reduced for certain cases. The applicable code/specification section are quoted below:

AASHTO 8.18.1.2–The minimum area of longitudinal reinforcement shall not be less than 0.01 times the gross area, A_g, of the section.

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

1.2.1 EXPERIMENTAL

A total of 38 conventionally-reinforced concrete columns were cast and tested. Each column had a nominal cross-sectional diameter of 8 inches and was 4 feet long. Of the 38 columns, 24 were subjected to axial load. The applied axial load was $0.40*f_c'*A_g$ for all but one of the axially-loaded specimens. This load corresponds with the maximum service load which can be derived from the AASHTO Bridge Specification [18] (Section 8.16.4.2.1) and ACI 318-95[12](Section 10.3.5.) for required strength of a tied column (using the approximation A_g*f_c' equal to the strength of the column). The load was 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 were made using mechanical Demec gages and electrical strain gages. Ambient humidity in the enclosures containing the test specimens was 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 percent, 0.54 percent, and 0.72 percent.
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.

1.2.2 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 *ACI Structural Journal* paper 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 report. 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.

The following is a brief summary of the rationale developed in the paper "Longitudinal Steel Limits for Concrete Columns" for preventing passive yielding of the reinforcing steel. ACI 318 load factors, which were used by Lin and Furlong in their assessment of reinforcement limits, are used here also.

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 impose a lower limit on the steel ratio.

Figure 1.1 is of use in understanding the relationships developed.

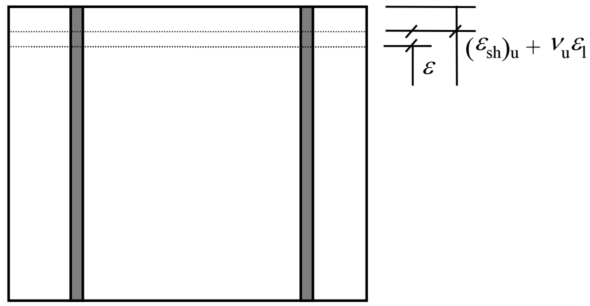


Figure 1.1 Schematic of Reinforced Concrete Column

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

$$1.4D + 1.7L = \varphi P_o \qquad \textbf{\textit{Equation 1.1}}$$

becomes

$$1.4D + 1.7XD = \varphi P_o \qquad \textbf{\textit{Equation 1.2}}$$

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 and AASHTO Bridge Specification requirements for spiral columns])
P_o	=	design axial load strength of column for zero eccentricity
φP_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,

$$D = \frac{\varphi P_o}{(1.4 + 1.7X)} \qquad \textbf{\textit{Equation 1.3}}$$

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 elasticity for concrete at time of loading:

$$\varepsilon_1 = \frac{D}{\left[A_g E_{ci}\left(1+\rho_g(n-1)\right)\right]} \qquad \textit{Equation 1.4}$$

where all terms are as defined previously and

E_{ci} = concrete modulus 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 column.

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

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:

$$f_s = E_s\left[(\varepsilon_{sh})_u + \nu_u\varepsilon_1 - \varepsilon\right] + nE_c\varepsilon_1(1+X) \qquad \textit{Equation 1.6}$$

where

f_s = total stress in longitudinal reinforcing steel

The maximum value of f_s allowed can be expressed as:

$$f_s = Rf_y \qquad \textit{Equation 1.7}$$

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_1 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 v_u and $(\varepsilon_{sh})_u$ apply:

v_u = 2.35 (normal range = 1.30 to 4.15)

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

Table 1.1 Minimum percent of Longitudinal Reinforcement (for $\nu_u = 2.35$ 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

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

v_u = 4.15

$(\varepsilon_{sh})_u$ = 1070×10^{-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 percent of Longitudinal Reinforcement (for $v_u = 4.15$ 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 AASHTO Bridge Specification [18] Section 8.18.1.2 and ACI 318-95[(11)] Section 10.8.4. The ACI section essentially permits a maximum reduction of the minimum steel percentage from 1.0 percent to 0.50 percent for cases in which the column is over-designed by a factor of 2 or more. The AASHTO section permits an even greater reduction.

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 eight 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 percent 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 Fahrenheit, 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 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 8¼ 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 fabricated 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 Fahrenheit 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) were 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 companion specimens were kept in like storage conditions under no load. In addition to the reinforced columns, 45 unreinforced 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 8¼ 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 8¼ 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 7½ 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 7½ 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 ps	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 reinf.	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 8¼ 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 ½ 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 5½ 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 7½ 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 carried 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 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 0.01 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 0.01 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 0.01 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 Committee Report 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 percent-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 against 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 0.01 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

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

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

3.2 COLUMN DETAILS

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

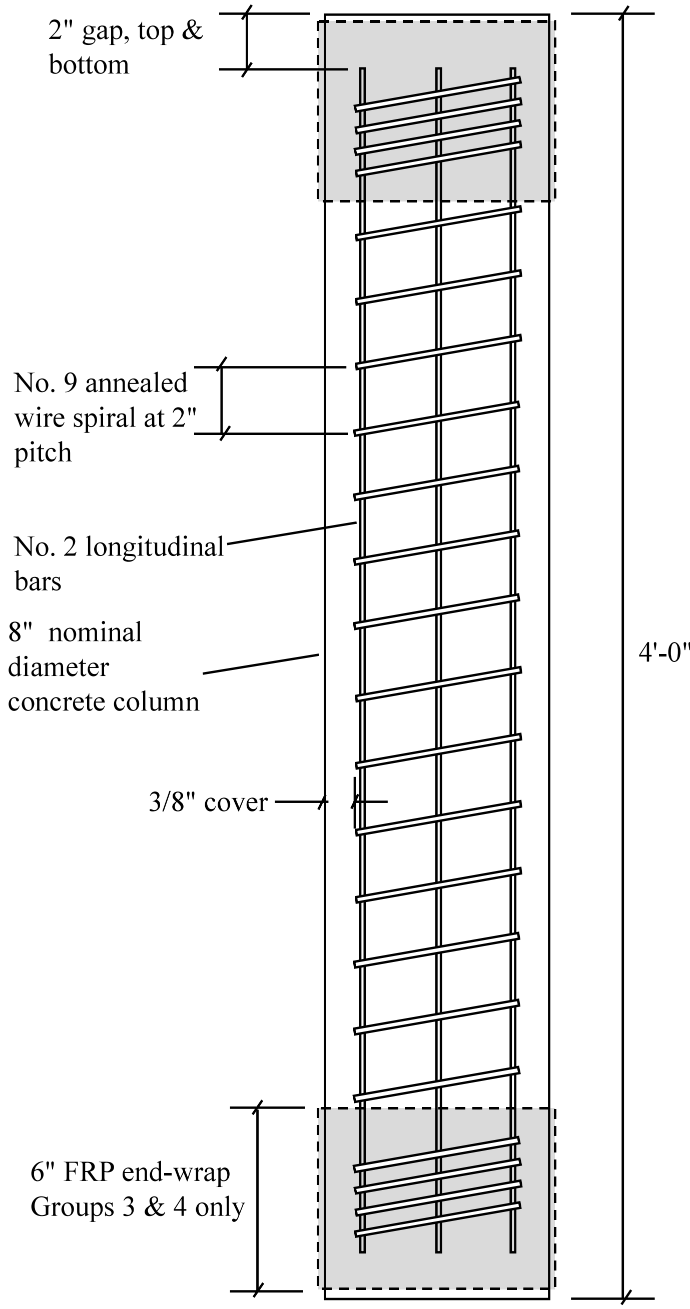


Figure 3.1 Typical Column Elevation

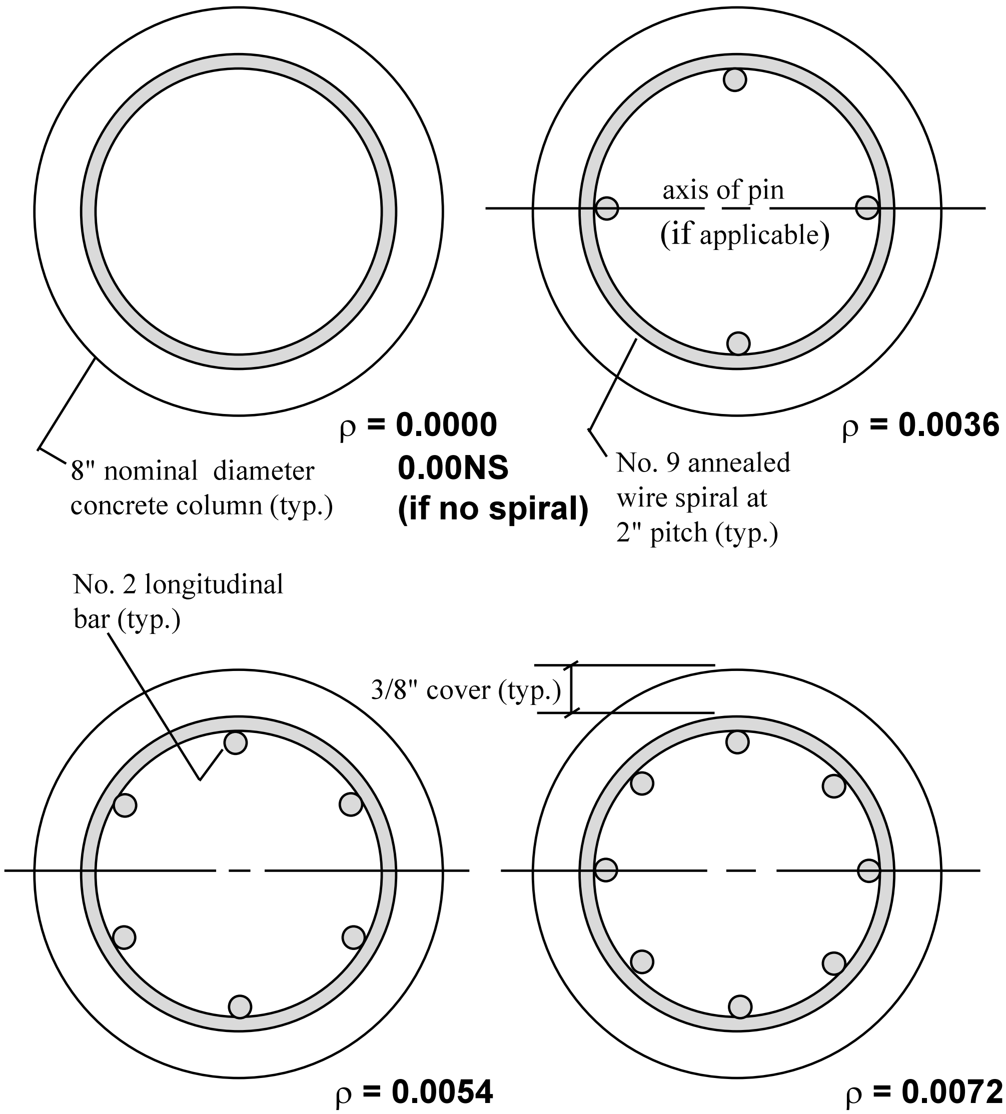


Figure 3.2 Column Specimen Cross Sections

Number 2 deformed bars were used for longitudinal reinforcement in the columns. Depending on the reinforcement ratio desired, 0, 4, 6, or 8 bars were used in the specimens. Because the Number 2 bars have a 0.046 square inch cross-sectional area, the resulting longitudinal reinforcement ratios for the columns were 0.0000, 0.0036, 0.0054, and 0.0072.

Spiral reinforcement was made with number 9 annealed wire. By hand feeding the wire around a 6-inch diameter spinning tube, a spiral with 7½ inch diameter and 2 inch pitch was created. The spirals were stored outside in a moist environment to allow a thin layer of corrosion to form.

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

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

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

Table 3.1 Number of Specimens

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

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

Table 3.2 Concrete Column Details

Specimen Number	Design Concrete Strength, psi	Concrete Strength At 28 Days, psi	Long. Reinf. Ratio	Spiral Reinf. Ratio	Column Load, kips	Loading Eccentricity, inches	End Condition	Casting Date	Age At Loading, days	Group #
C8-00	8.000	6.920	0.0000	0.0027	162.5	None	flat-flat	5/15/96	21	2
C8-36-1	8.000	9.180	0.0036	0.0027	162.5	None	pin-pin	2/7/96	20	1
C8-36-2	8.000	9.180	0.0036	0.0027	162.5	None	pin-pin	2/7/96	17	1
C8-36-3	8.000	6.920	0.0036	0.0027	162.5	None	flat-flat	5/15/96	22	2
C8-54-1	8.000	9.180	0.0054	0.0027	162.5	None	pin-pin	2/7/96	19	1
C8-54-2	8.000	9.180	0.0054	0.0027	162.5	None	pin-pin	2/7/96	19	1
C8-54-3	8.000	6.920	0.0054	0.0027	162.5	None	flat-flat	5/15/96	22	2
C8-72-1	8.000	9.180	0.0072	0.0027	162.5	None	pin-pin	2/7/96	19	1
C8-72-2	8.000	9.180	0.0072	0.0027	162.5	None	pin-pin	2/7/96	19	1
C8-72-3	8.000	6.920	0.0072	0.0027	162.5	None	flat-flat	5/15/96	20	2
C4-00	4.000	4.460	0.0000	0.0027	81.3	None	flat-flat	5/15/96	16	4
C4-36-1	4.000	5.390	0.0036	0.0027	81.3	None	pin-pin	4/4/96	23	3
C4-36-2	4.000	5.390	0.0036	0.0027	81.3	None	pin-pin	4/4/96	22	3
C4-36-3	4.000	4.460	0.0036	0.0027	81.3	None	flat-flat	5/15/96	15	4
C4-54-1	4.000	5.390	0.0054	0.0027	81.3	None	pin-pin	4/4/96	25	3
C4-54-2	4.000	5.390	0.0054	0.0027	81.3	None	pin-pin	4/4/96	24	3
C4-54-3	4.000	4.460	0.0054	0.0027	81.3	None	flat-flat	5/15/96	15	4
C4-72-1	4.000	5.390	0.0072	0.0027	81.3	None	pin-pin	4/4/96	23	3
C4-72-2	4.000	5.390	0.0072	0.0027	81.3	None	pin-pin	4/4/96	21	3
C4-72-3	4.000	4.460	0.0072	0.0027	81.3	None	flat-flat	5/15/96	15	4
E8-36	8.000	6.920	0.0036	0.0027	121.9	0.80	pin-pin	5/15/96	22	2
E8-72	8.000	6.920	0.0072	0.0027	162.5	0.80	pin-pin	5/15/96	23	2
E4-36	4.000	4.460	0.0036	0.0027	81.3	0.80	pin-pin	5/15/96	27	4
E4-72	4.000	4.460	0.0072	0.0027	81.3	0.80	pin-pin	5/15/96	27	4
U8-NS-1	8.000	9.180	0.0000	0.0000	0	-	-	2/7/96	-	1
U8-NS-2	8.000	6.920	0.0000	0.0000	0	-	-	5/15/96	-	2
U8-00-1	8.000	9.180	0.0000	0.0027	0	-	-	2/7/96	-	1
U8-00-2	8.000	6.920	0.0000	0.0027	0	-	-	5/15/96	-	2
U8-36	8.000	9.180	0.0036	0.0027	0	-	-	2/7/96	-	1
U8-54	8.000	9.180	0.0054	0.0027	0	-	-	2/7/96	-	1
U8-72	8.000	9.180	0.0072	0.0027	0	-	-	2/7/96	-	1
U4-NS-1	4.000	5.390	0.0000	0.0000	0	-	-	4/4/96	-	3
U4-NS-2	4.000	4.460	0.0000	0.0000	0	-	-	5/15/96	-	4
U4-00-1	4.000	5.390	0.0000	0.0027	0	-	-	4/4/96	-	3
U4-00-2	4.000	4.460	0.0000	0.0027	0	-	-	5/15/96	-	4
U4-36	4.000	5.390	0.0036	0.0027	0	-	-	4/4/96	-	3
U4-54	4.000	5.390	0.0054	0.0027	0	-	-	4/4/96	-	3
U4-72	4.000	5.390	0.0072	0.0027	0	-	-	4/4/96	-	3

Note: All columns cast and maintained in a humidity-controlled enclosure.

3.3 REDUCED-HUMIDITY ENVIRONMENT

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

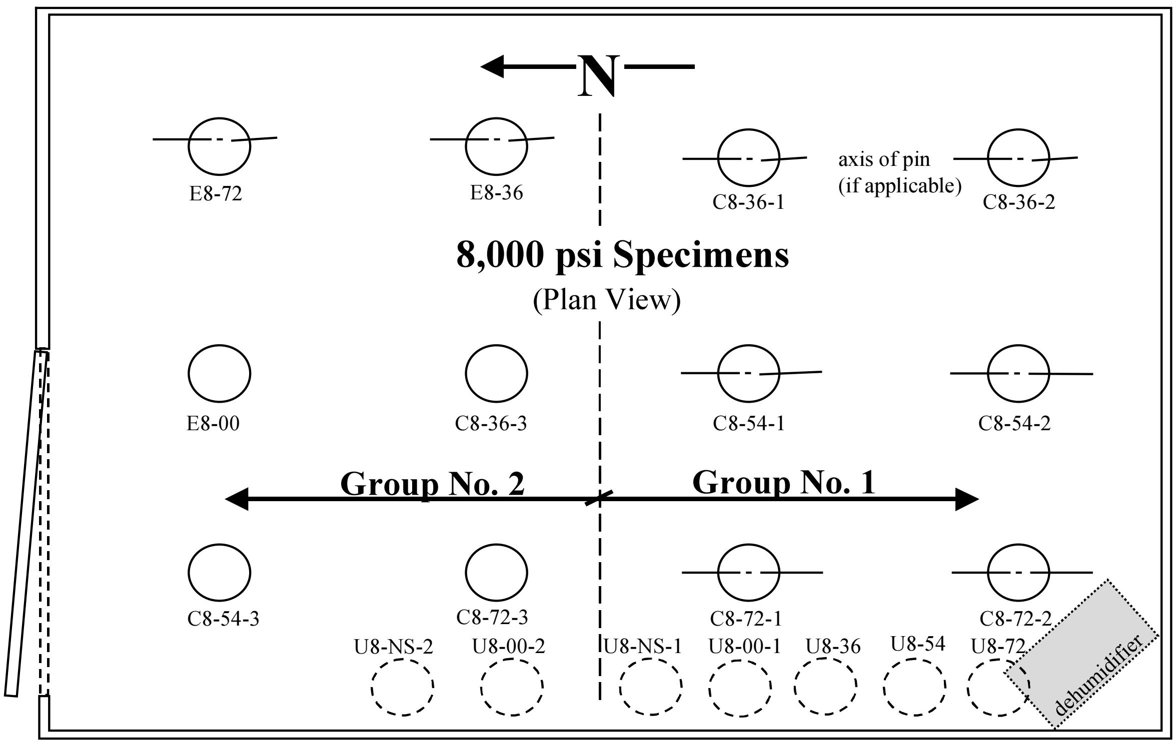


Figure 3.3 Enclosure No. 1–Locations of Test Specimens

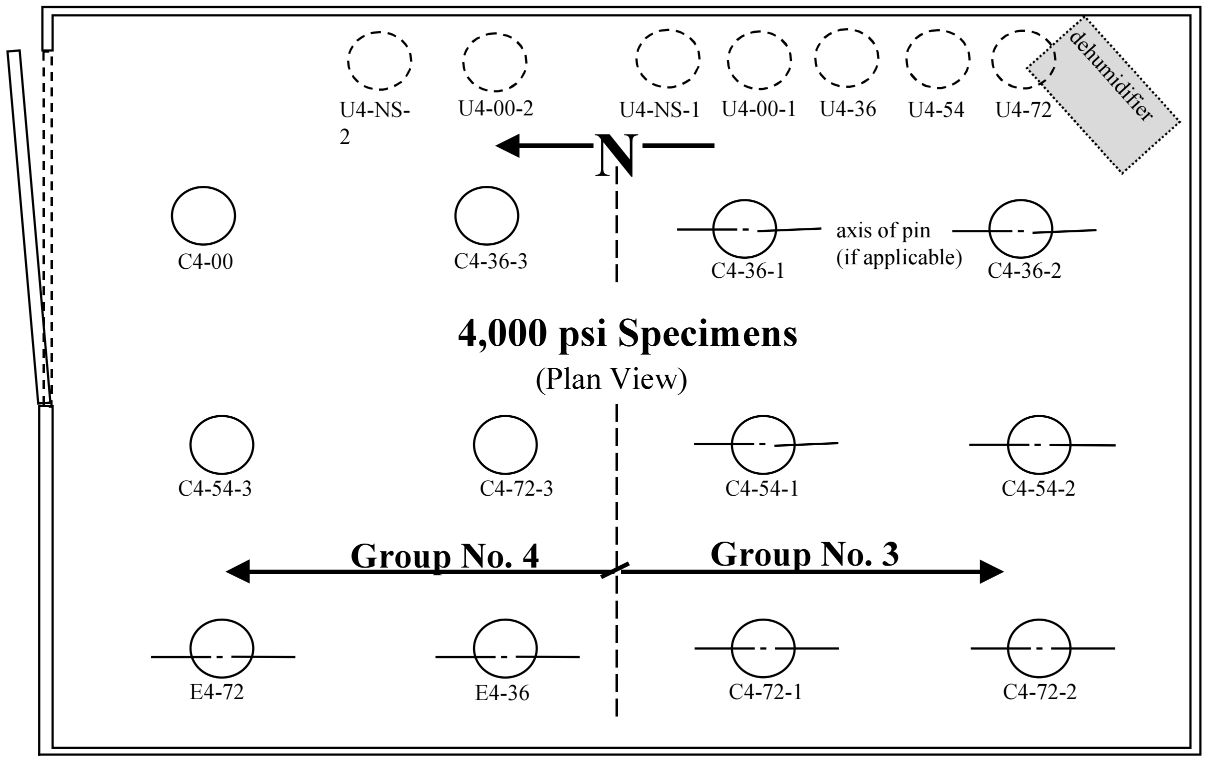
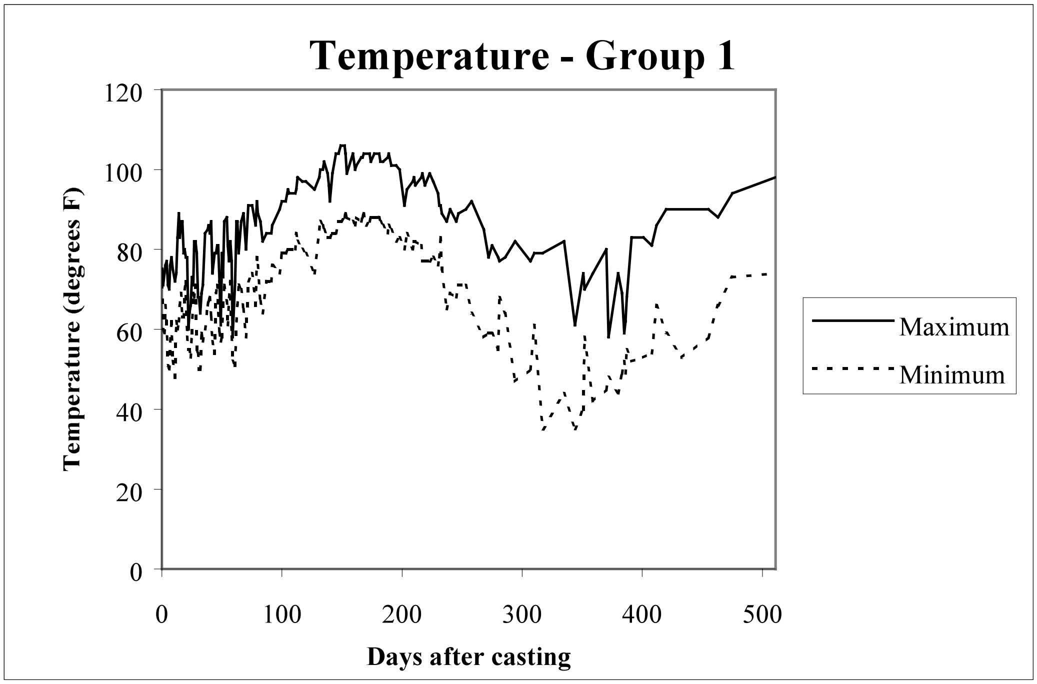


Figure 3.4 Enclosure No. 2–Locations of Test Specimens

On cold days, small space heaters were used to keep the temperature in the enclosures above 50 degrees Fahrenheit.

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



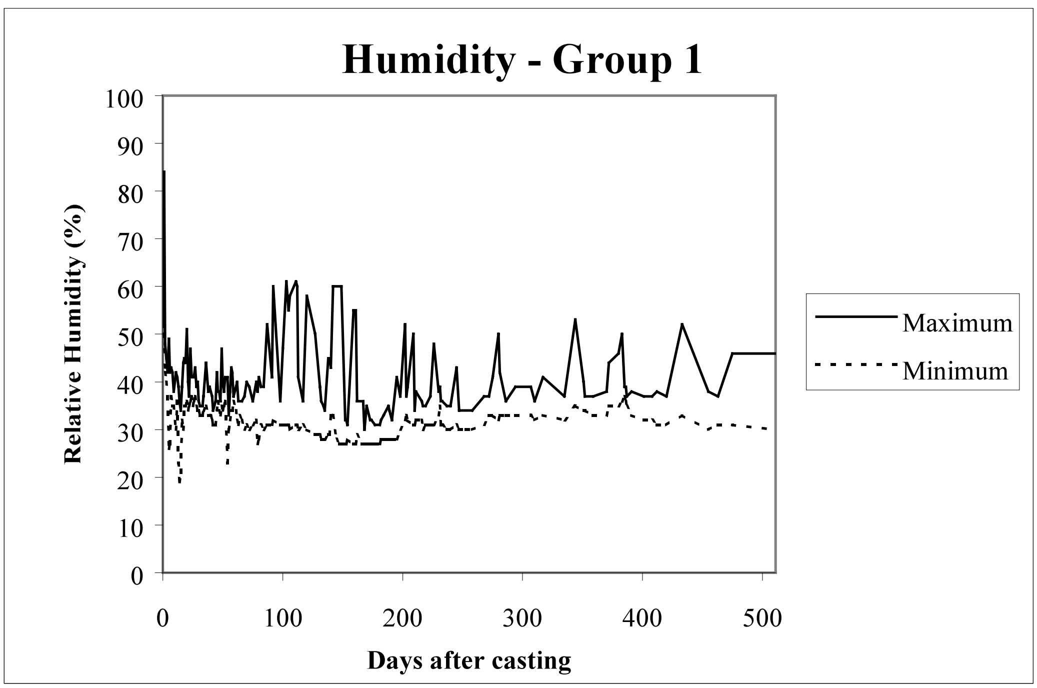
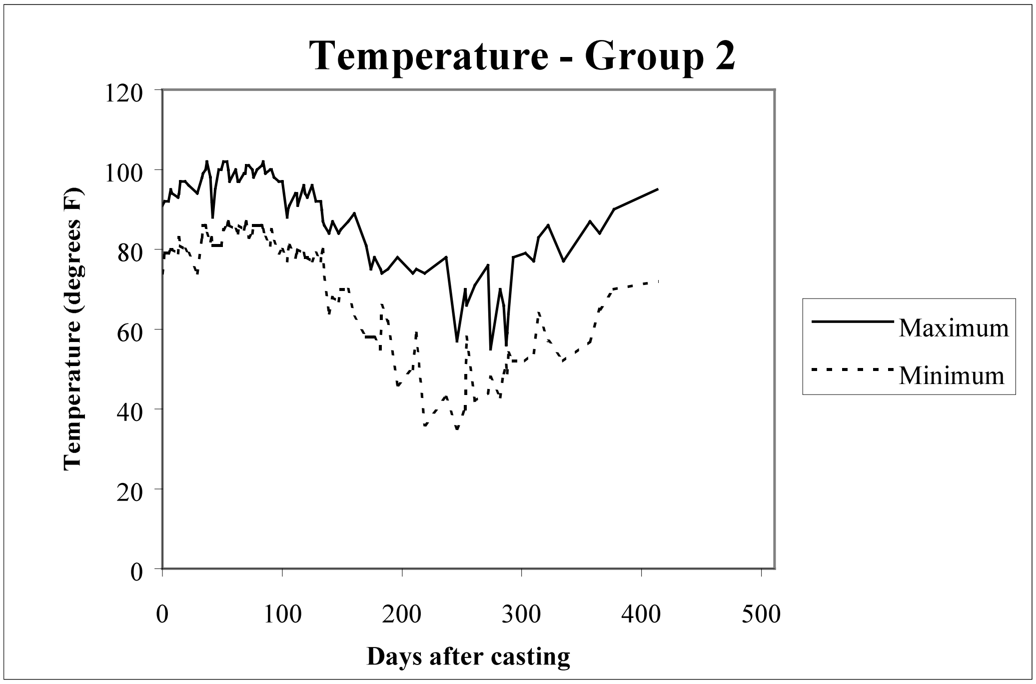


Figure 3.5 Temperature and Humidity History for Group 1 Specimens



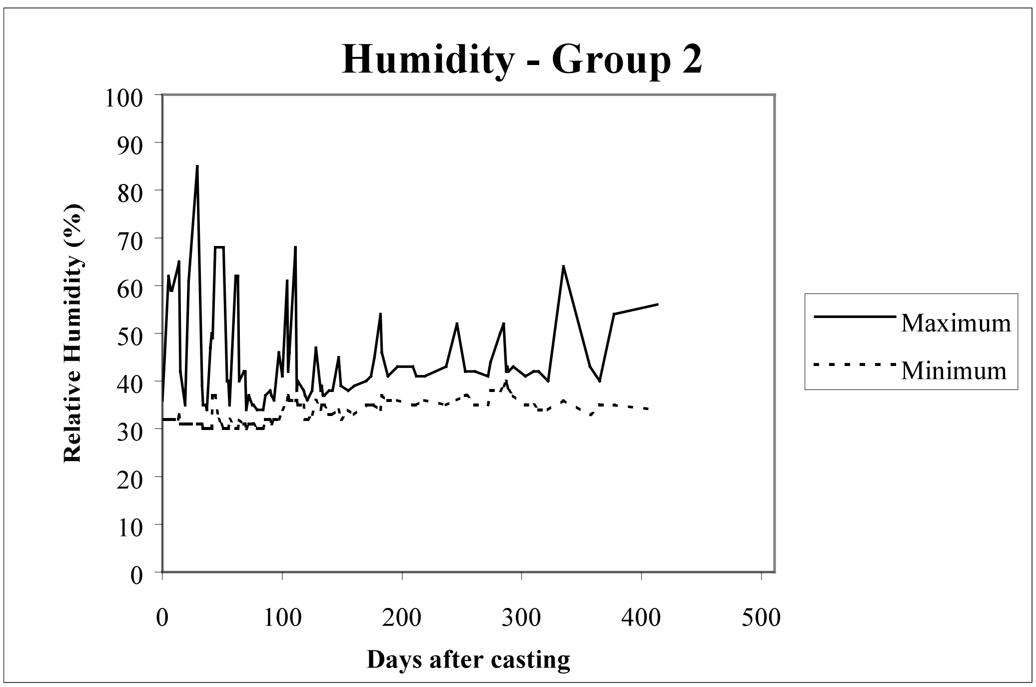
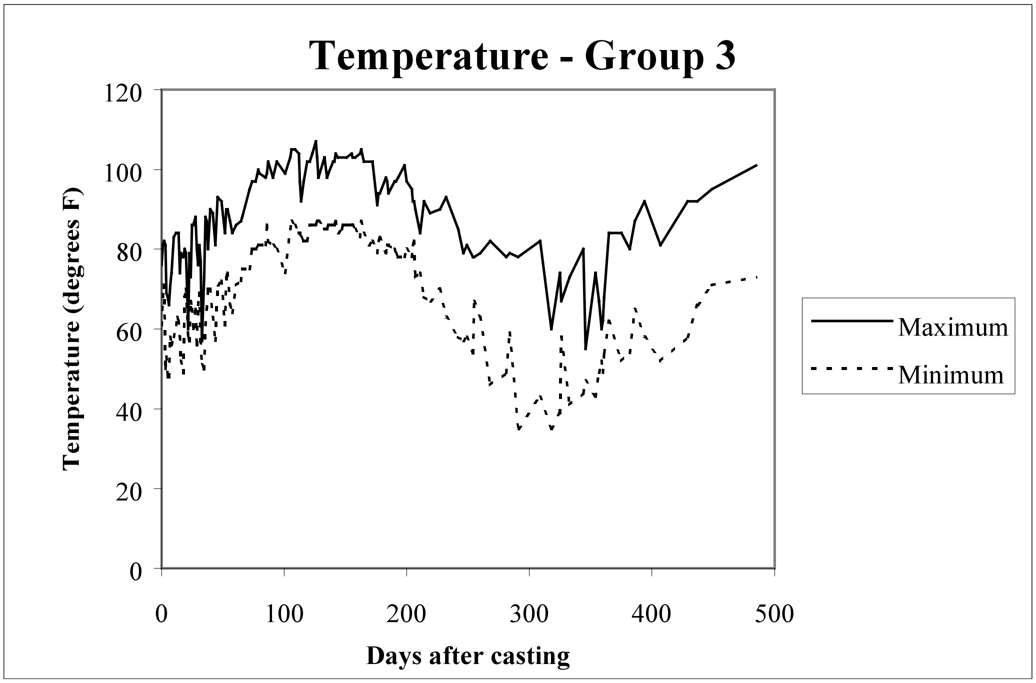


Figure 3.6 Temperature and Humidity History for Group 2 Specimens



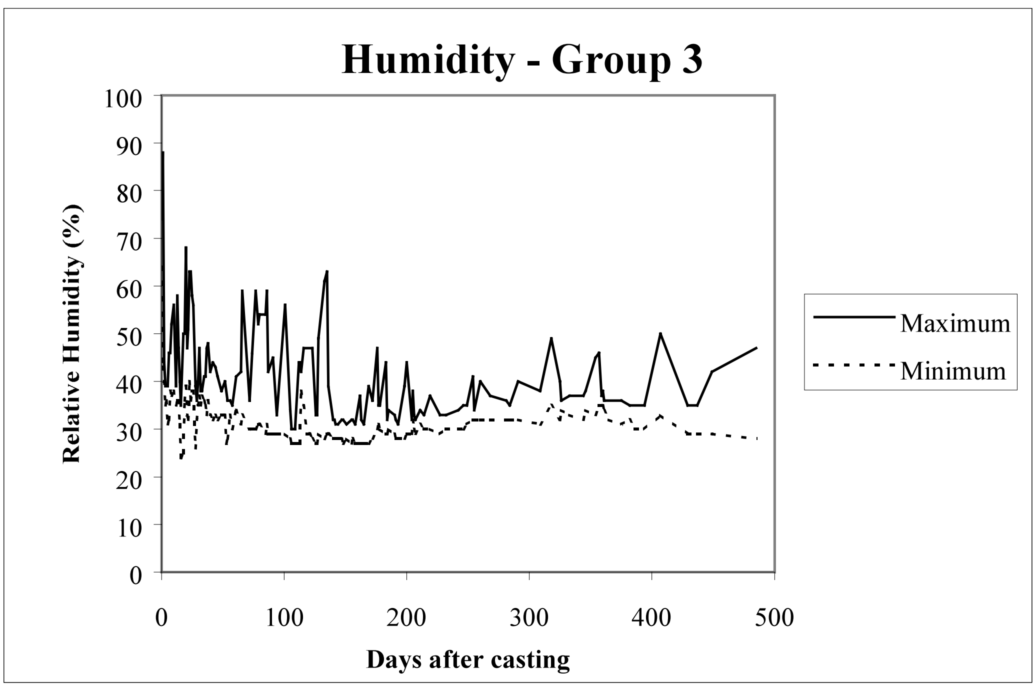
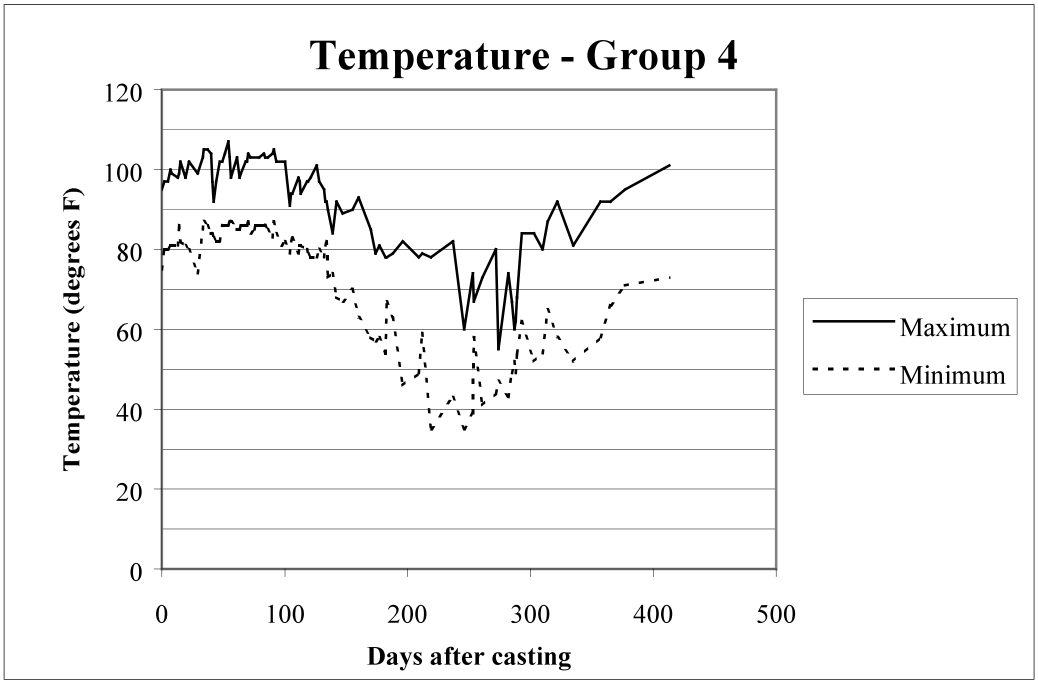


Figure 3.7 Temperature and Humidity History for Group 3 Specimens



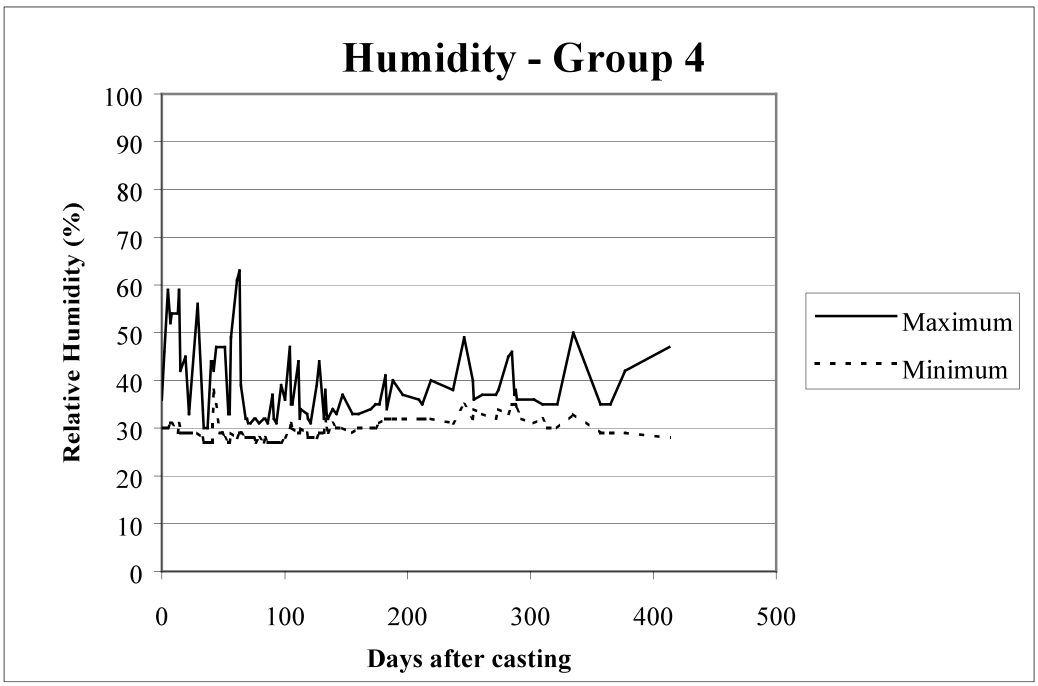


Figure 3.8 Temperature and Humidity History for Group 4 Specimens

3.4 Materials

3.4.1 *Concrete*

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

A local ready-mix plant was the source for the concrete. Concrete was brought to the lab four separate times, once for each group of specimens. The moisture content of the fine and coarse aggregates used in the concrete was not controlled and it was impossible to accurately estimate the values. To identify the proper mix proportions, test mixes with various slumps were used.

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

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

Table 3.3 Concrete Mix Proportions

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

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

3.4.2 *Reinforcing Steel*

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

3.4.3 *Fiberglass Reinforced Plastic*

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

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

3.5 MANUFACTURE OF TEST SPECIMENS

3.5.1 COLUMNS

3.5.1.1 Formwork

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

3.5.1.2 Casting

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

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

3.5.1.3 Curing

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

3.5.1.4 Application of Fiber-Reinforced Plastic

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

3.5.2 CYLINDERS

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

3.6 Testing Appurtenances

3.6.1 Columns

3.6.1.1 Test Frames

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

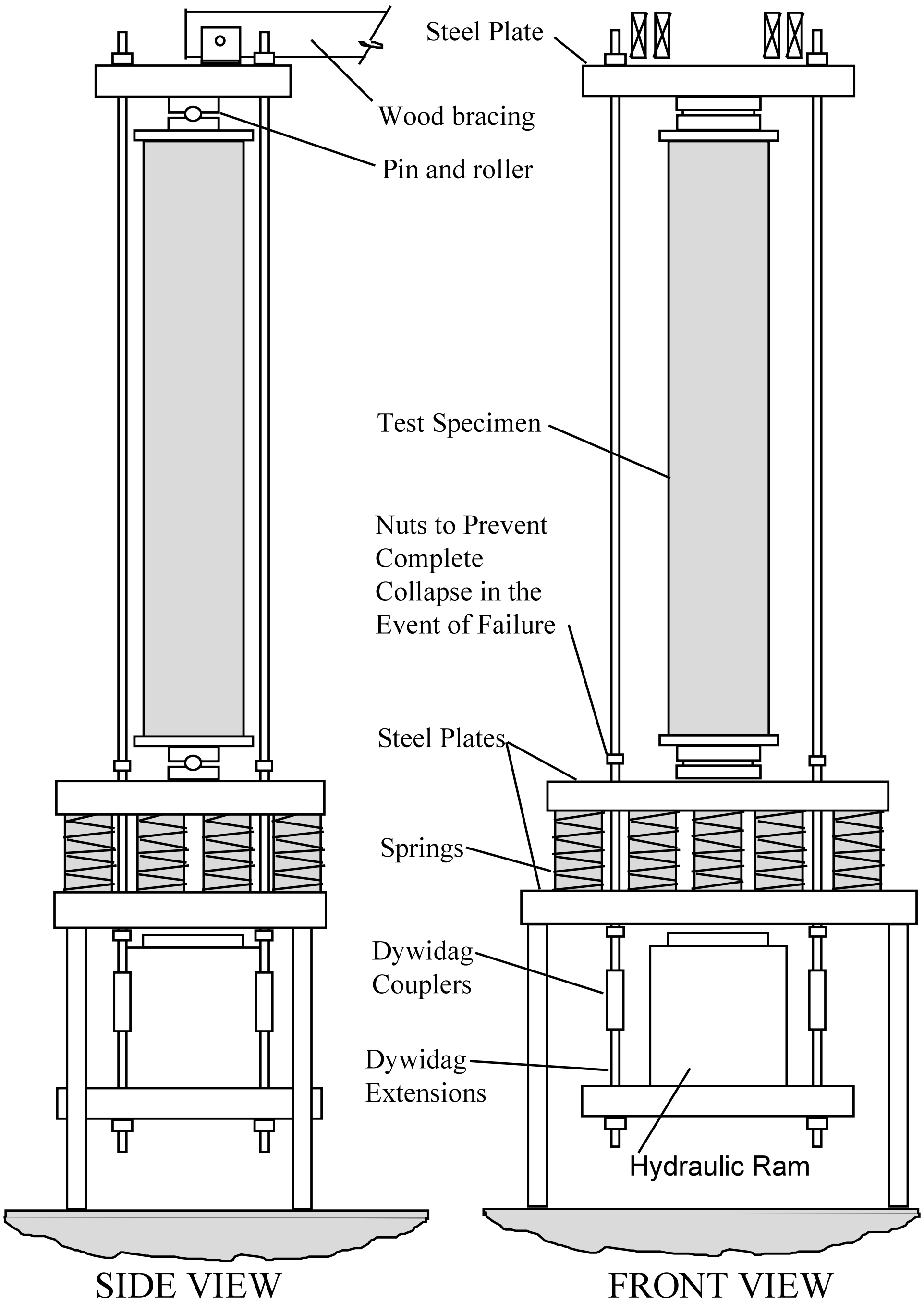


Figure 3.9 Schematic Side and Front Views of Test Setup

The legs of the testing frames were made from $\frac{3}{16}$ inch thick steel tubes. Load was maintained using triple-coil springs that deflected 1½ inches under 20 kips of sustained load. Eight springs were used to maintain the 162 kip load, and four springs were needed to maintain the 81 kip load. Deformations of the springs were monitored using a metal scale with an accuracy of $\frac{1}{64}$ inch.

Four Dywidag bars were used in all the setups. The 4,000 psi specimens used $\frac{5}{8}$ inch diameter bars and the 8,000 psi specimens used 1 inch diameter bars. The bars extended from approximately 3 inches above the 3 inch thick steel top plate down to 9 inches below the bottom plate. To apply load to each specimen, a hydraulic ram was placed on a 3 inch steel plate which was positioned beneath each frame and attached to the Dywidag bars using four coupling sleeves. Once load was applied to a specimen, nuts beneath the steel plate holding the triple coil springs were snug-tightened to maintain the spring deflections and thus, the load on the specimen. Using this setup, several specimens could be loaded each day.

Groups 1 and 3 had pinned-end conditions. Two 1½ inch steel plates were separated by a 1¼ inch diameter steel rod which fit into depressions in the steel plates. This arrangement was used on both the top and bottom of the specimens. Installation of these pins proved to be quite difficult because it was difficult to align the two pins with the imperfections in the columns and loading system. Because of this difficulty the concentrically-loaded specimens in Groups 2 and 4 were not loaded using these pins. Instead, some specimens had neoprene pads placed between the top and bottom plates and the specimens. This approach also contained inherent problems because the pads tended to "walk" on the surface of the bottom plate during loading, which resulted in eccentricity of the applied load. As a result, only two of the specimens had neoprene pads. The remainder were loaded without neoprene pads. The eccentrically-loaded specimens used the pins and had their top plates braced against lateral movement.

3.6.1.2 Strain Measurements

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

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

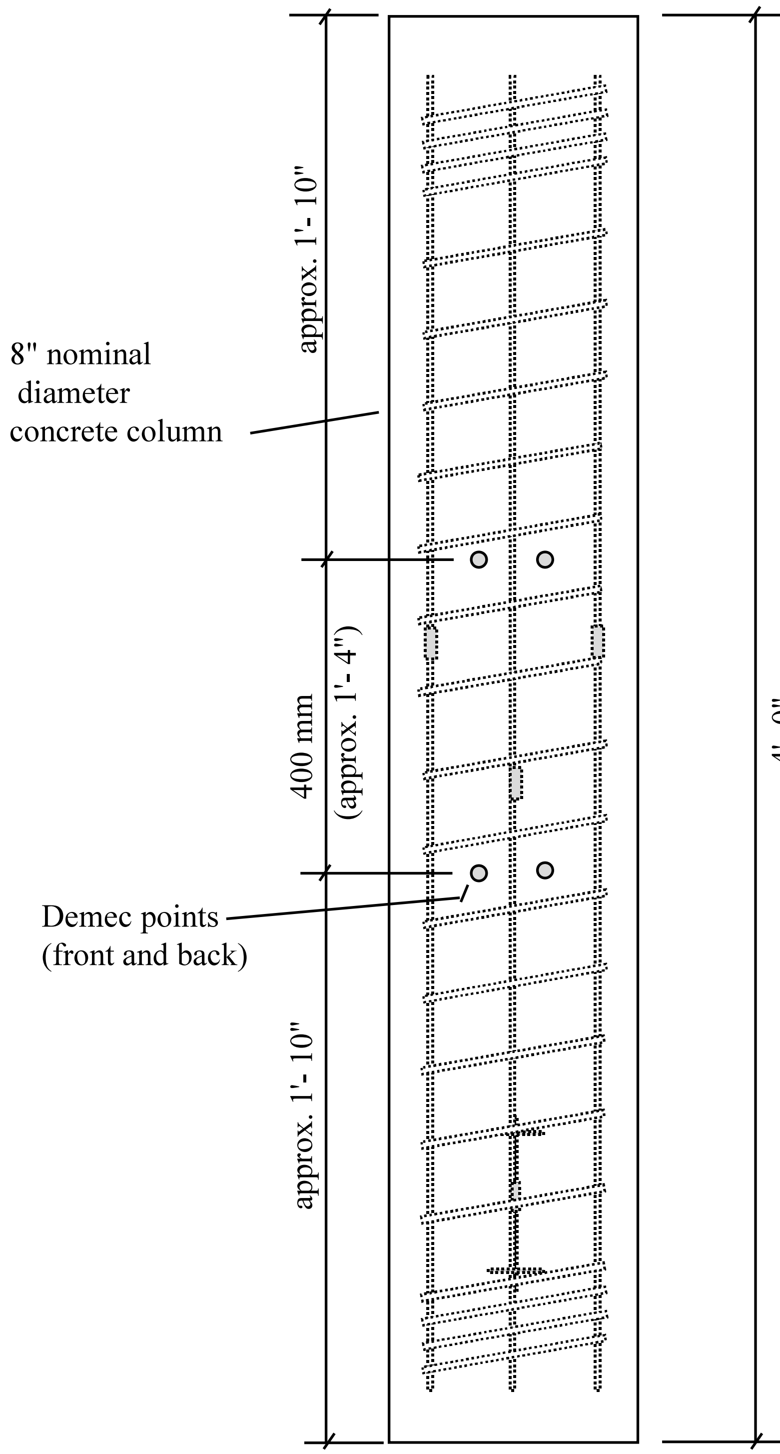


Figure 3.10 Vertical Location of Demec Points

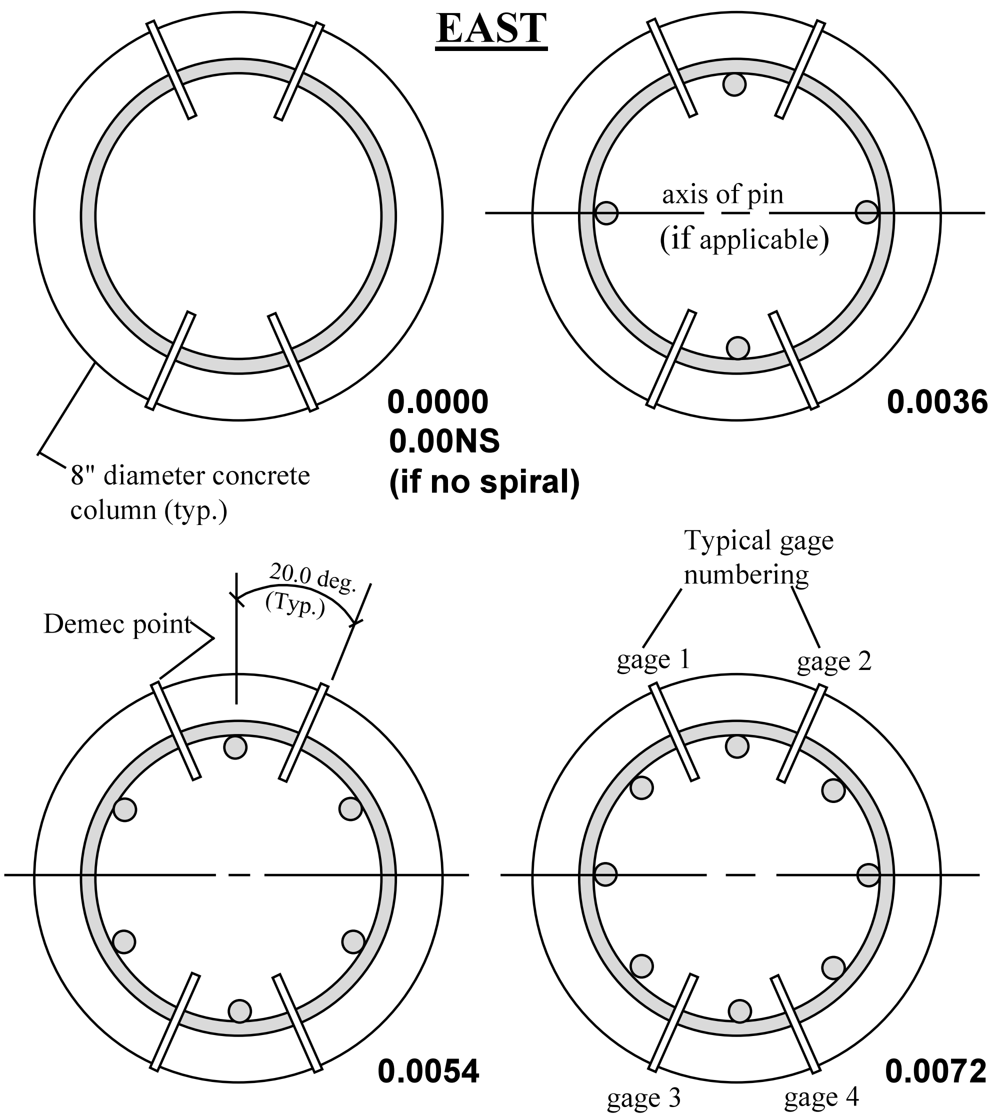


Figure 3.11 Plan Location of Demec Points

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

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

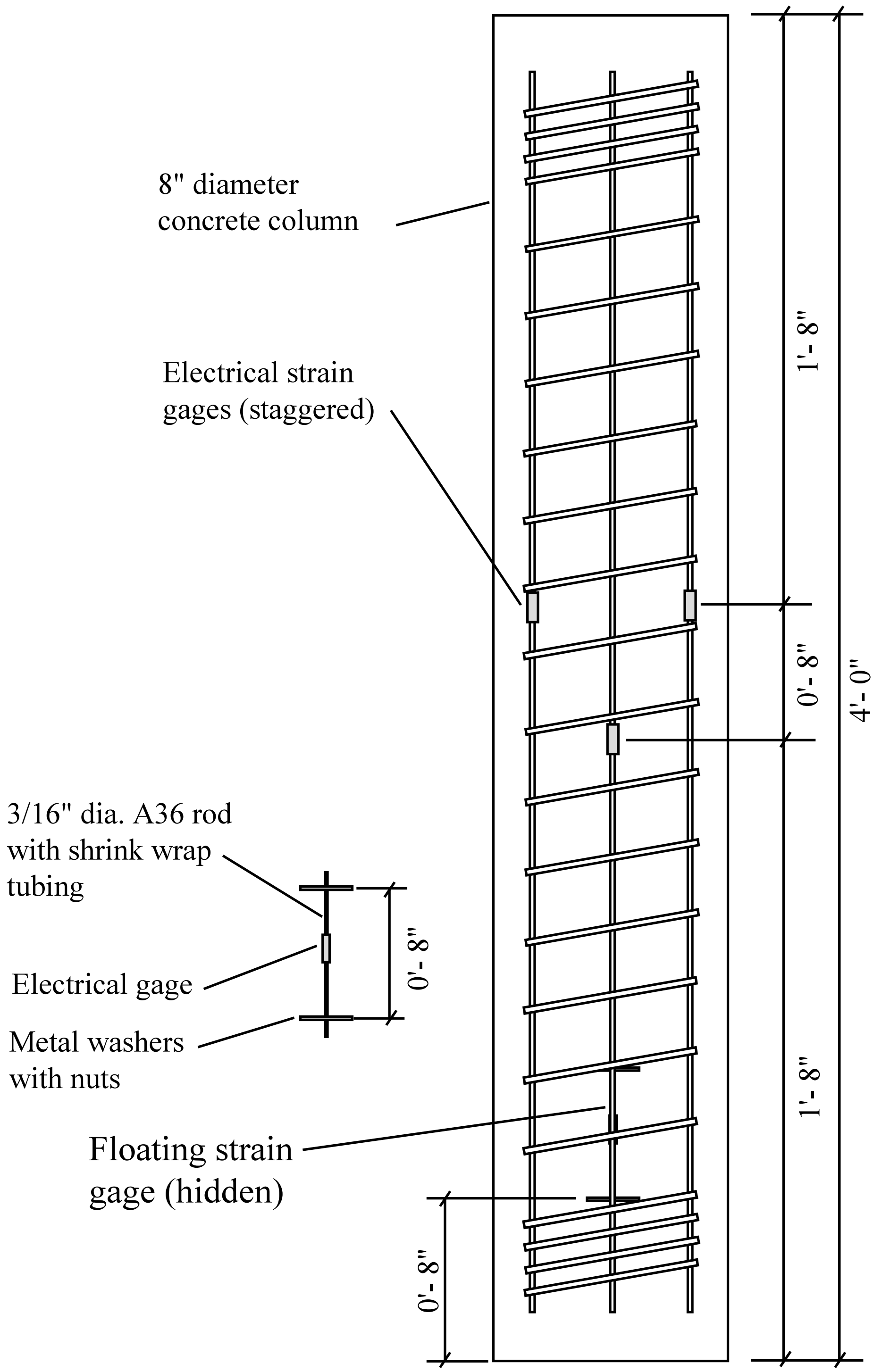


Figure 3.12 Vertical Location of Electrical Gages

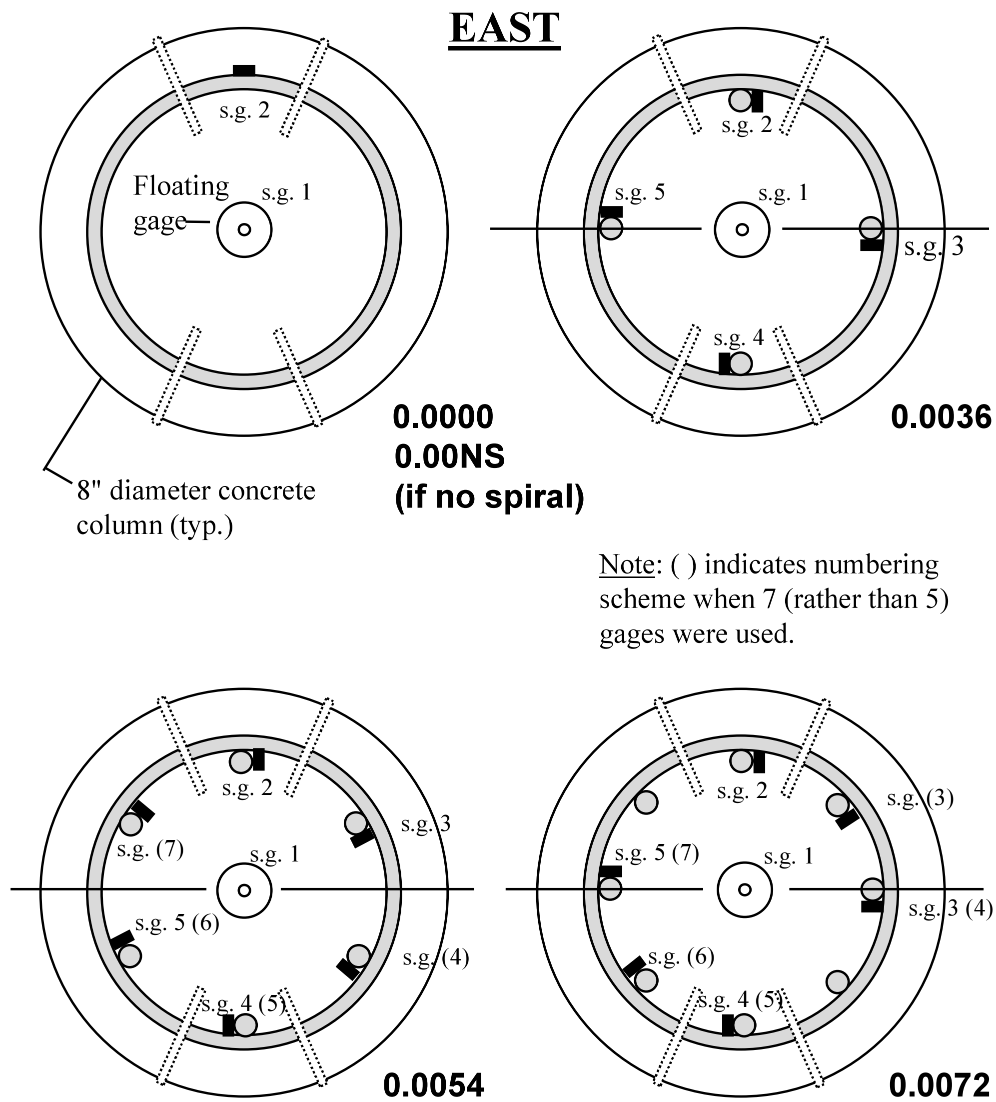


Figure 3.13 Plan Location of Electrical Gages

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

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

3.6.1.3 Testing Procedure

Each loaded specimen was subjected to $0.40{*}A_g{*}f_c'$ of axial load, where A_g is the gross cross-sectional area of the column and f_c' is the nominal compressive strength of the concrete. The exception to this loading was Specimen E8-36 which was loaded with $0.30{*}A_g{*}f_c'$. The load for Specimen E8-36 was reduced due to noticeable cracking on the compression side of the column. The resulting loads were 81.2 kips for the 4,000 psi specimens and 162 kips for the

8,000 psi specimens. The columns with eccentric load had an eccentricity of 0.80 inches which was equivalent to the code minimum of 10 percent of the nominal column diameter.

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

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

3.6.2 CONCRETE CYLINDERS

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

3.6.3 REINFORCING STEEL

Four tensile tests were conducted on the Number 2 bars using a 60-kip capacity Tinius-Olson universal test machine. The yield and ultimate strength of the Number 2 bars were determined from these tests.

CHAPTER 4

EXPERIMENTAL TEST RESULTS

4.1 INTRODUCTION

The test results for individual column specimens are shown in Figures 4.1 through 4.38. These results are based on data collected from the initiation of creep tests through July 2, 1997. To enable convenient comparisons, test results from individual concentrically-loaded specimens are grouped together in Figures 4.39 through 4.46.

This report considers data collected through July 2, 1997. The rate of time-dependent strain increase had dropped to such a low level that temperature-related changes in strain masked the time-dependent changes. The very low rate of strain increase is evident in many of the strain-vs.-time plots shown in Figurers 4.1 through 4.24.

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

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

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

Temperature inside each enclosure was typically measured at the time strain readings in the specimens were made. Temperature readings used were ambient temperature readings as opposed to temperature readings from inside the concrete specimens. In the event that no temperature reading was available, the researchers' judgement was used to provide a reasonable temperature for the day and time of the reading in question. This estimated temperature was based on readings taken prior to and after the missing temperature data and previous experience with temperatures in the enclosures.

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

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

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

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

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

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

4.2. Individual Column Specimens

The data are presented in two plots on two pages for each specimen. Demec (mechanical gage) data are presented with a plot of the running average of the Demec readings for that specimen. Data from the electrical gages, which tended to be inconsistent and more erratic with time, were deemed unreliable and are not presented here. These data are presented in the thesis by Cloyd[19].

The electrical gages of Groups 2 and 4 had a higher failure rate than the gages of Groups 1 and 3. This may be attributed to two separate problems: the possibility of poorer workmanship in water proofing the gages at the time of application, and the possibility that gages were somehow contaminated when allowed to sit for several months after having been applied to the reinforcing bars prior to casting in the concrete specimens. All strain gages were applied prior to January, 1996. Specimens in Groups 2 and 4 were not cast until May 15, 1996.

Specimen strain data are presented in three separate sets. Data for the concentrically-loaded specimens are presented first, followed by the eccentrically-loaded specimens, and then the unloaded specimens. Data for each of these groups are subdivided into 8 ksi and 4 ksi groups. Each of these groups is presented in order from specimens containing the least steel to specimens containing the most steel.

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

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

4.2.1 Concentrically Loaded Specimens

4.2.1.1 8,000 psi

Strain data from the 8,000 psi concentrically-loaded specimens are presented in the following figures:

Reinf. ratio	Figures
0.0000	4.1
0.0036	4.2–4.4
0.0054	4.5–4.7
0.0072	4.8–4.10

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

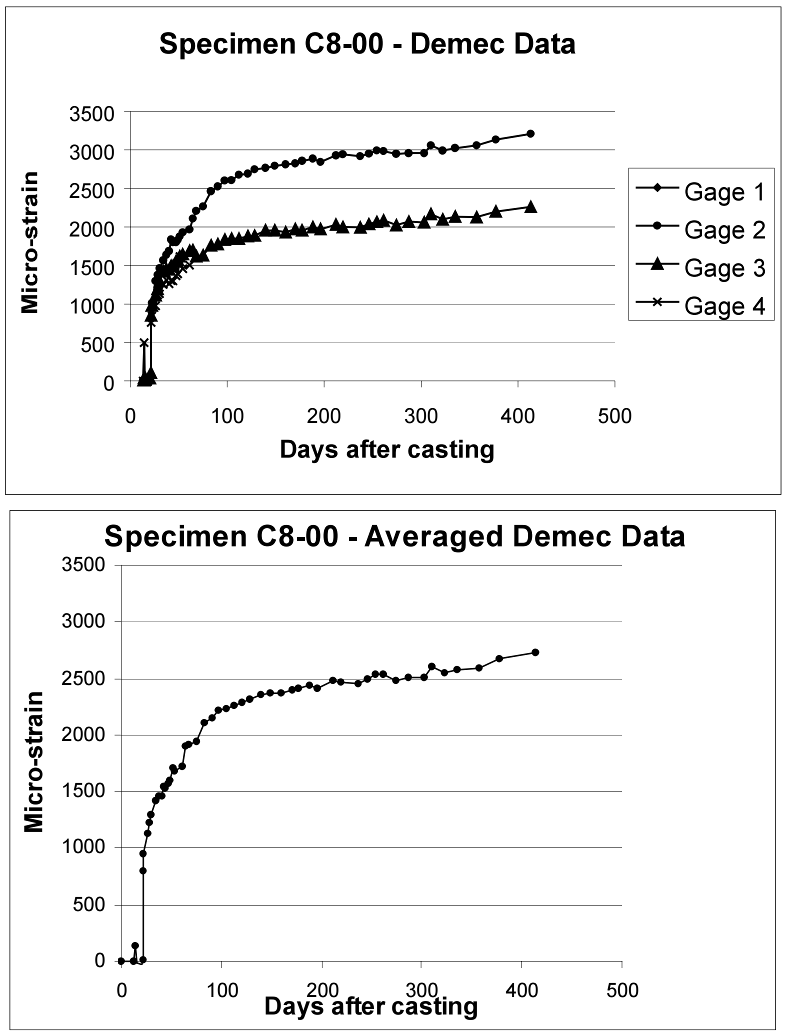
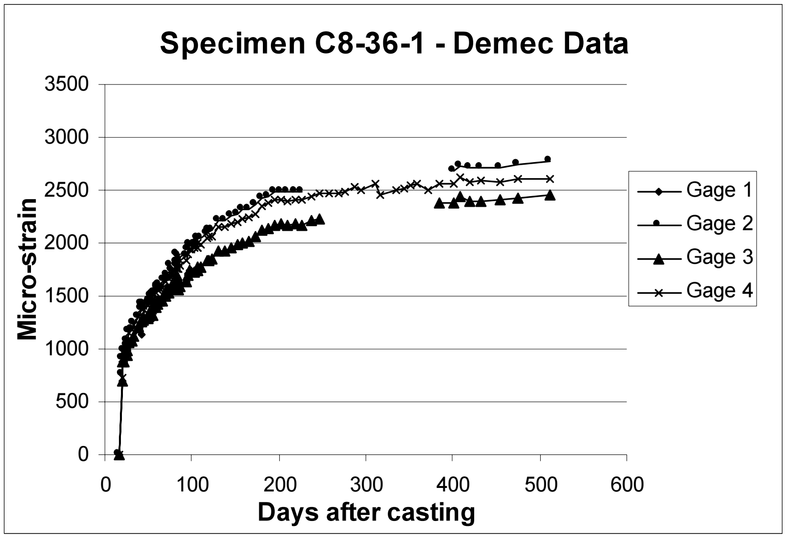


Figure 4.1 Mechanical Strain Gage Data for Specimen C8-00

Specimen C8-36-1 (Fig. 4.2) also had Demec gage 1 fail to produce data due to improper installation. The remaining gages produced data with little scatter. The Demec points for gages 2 and 3 experienced large deflections, and thus, some data is missing as previously explained. Specimen C8-36-2 (Fig. 4.3) had Demec gage 2 improperly installed and was unreadable. Gages 1, 3, and 4 produced reasonable data with some scatter likely due to unintentional eccentricity. Specimen C8-36-3's (Fig 4.4) Demec gages had significant scatter, most likely from eccentricity about a northeast-southwest axis. This explains gages 1 and 4 having equal strains while gage 3 produced significantly higher strains and gage 2 produced significantly lower strains.



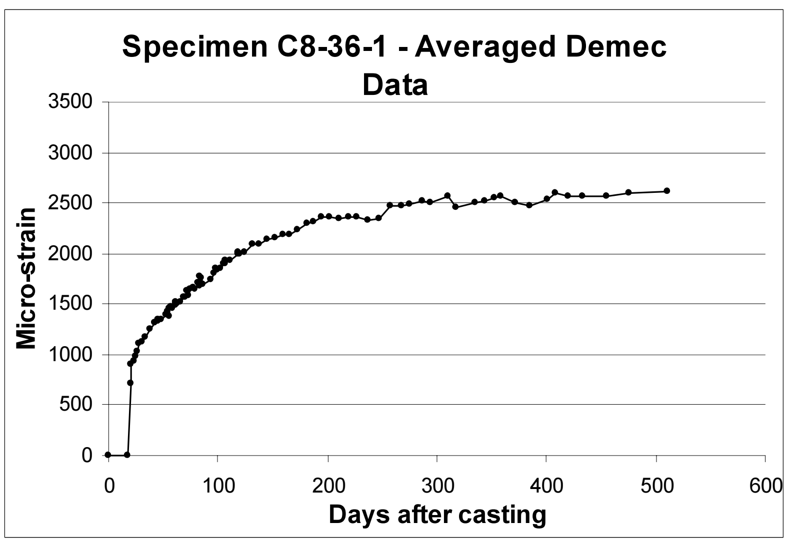
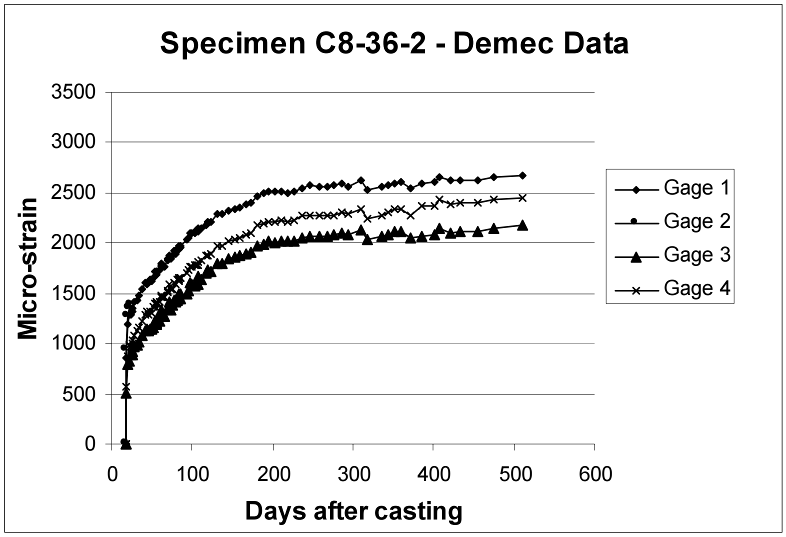


Figure 4.2 Mechanical Strain Gage Data for Specimen C8-36-1



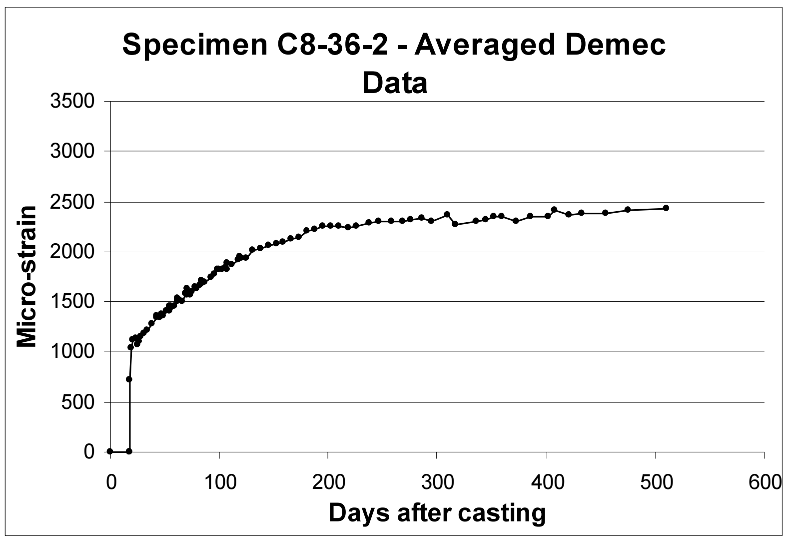
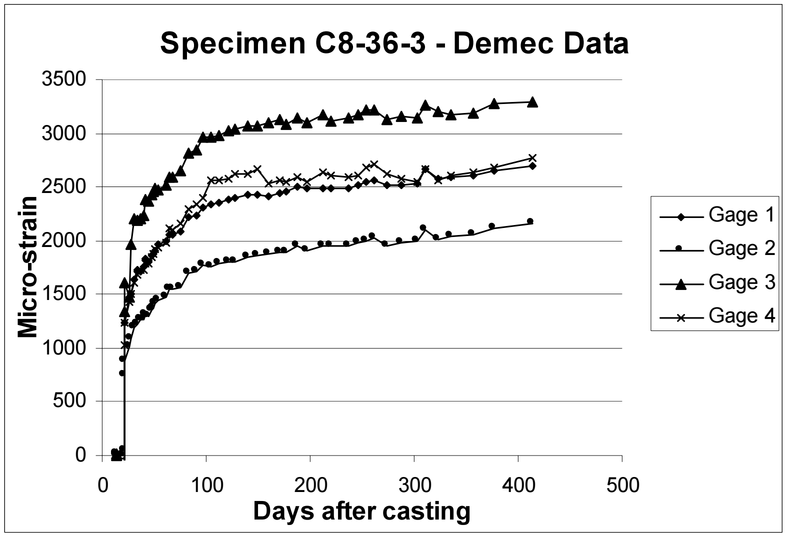


Figure 4.3 Mechanical Strain Gage Data for Specimen C8-36-2



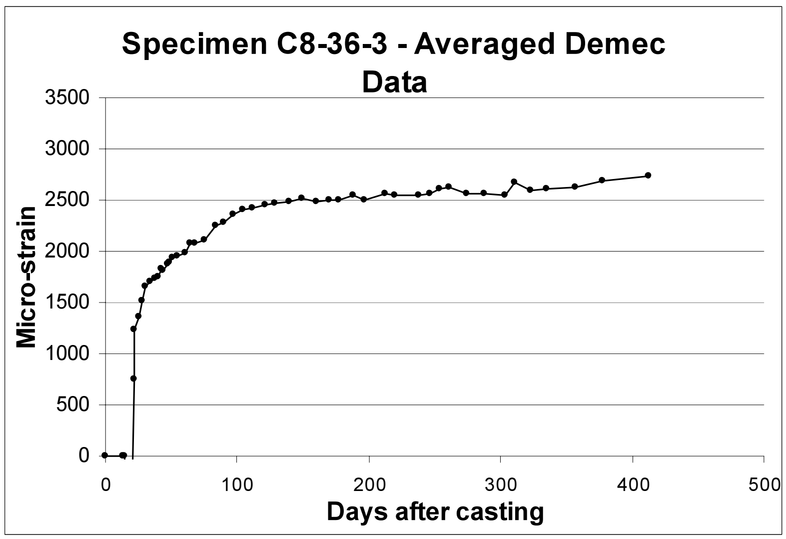
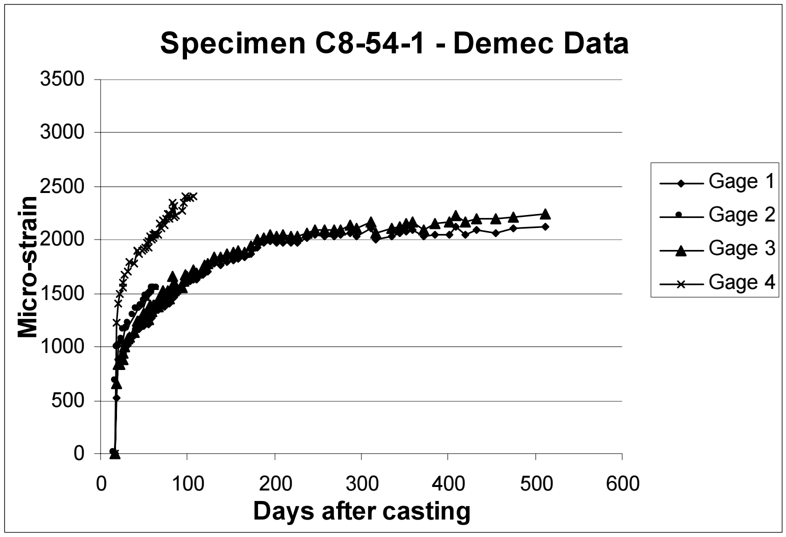


Figure 4.4 Mechanical Strain Gage Data for Specimen C8-36-3

Specimen C8-54-1 (Fig. 4.5) had two gages fail to provide data soon after loading. Gage 2 failed after 60 days and gage 4 failed after 100 days. Specimen C8-54-2 (Fig 4.6) had only two of its Demec gages installed properly. The remaining gages (2 and 4) indicated a small eccentricity was present in the specimen. Specimen C8-54-3's (Fig. 4.7) Demec gages produced very good data with little scatter. Specimen C8-72-1 (Fig. 4.8) had two Demec gages improperly installed. The remaining Demec gages produced data with little scatter. Specimen C8-72-2's (Fig. 4.9) Demec points for gage 1 quickly deflected beyond the capacity of the Demec gage, and thus, stopped producing data after 70 days. The remaining three gages produced consistent data with lower strains, and so the data from gage 1 was likely erroneous. The gage 1 data was not used in the averaged results. Specimen C8-72-3's (Fig. 4.10) Demec gages produced data that indicated the presence of some eccentricity.



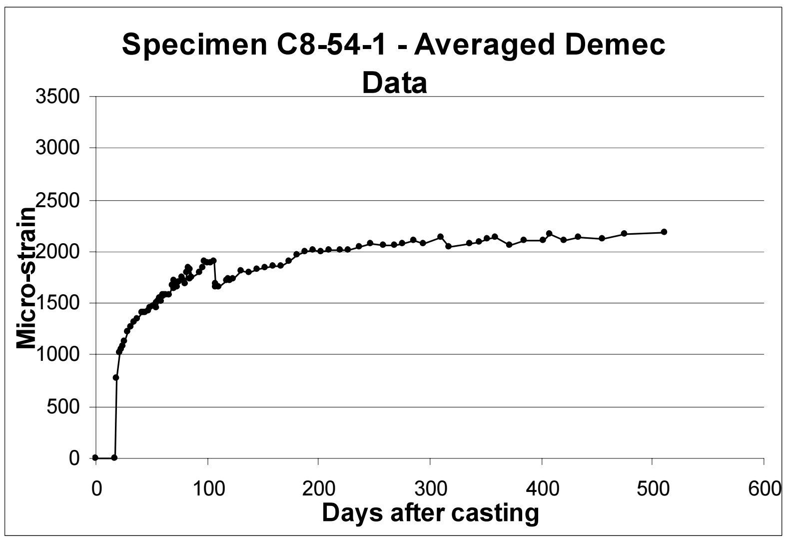
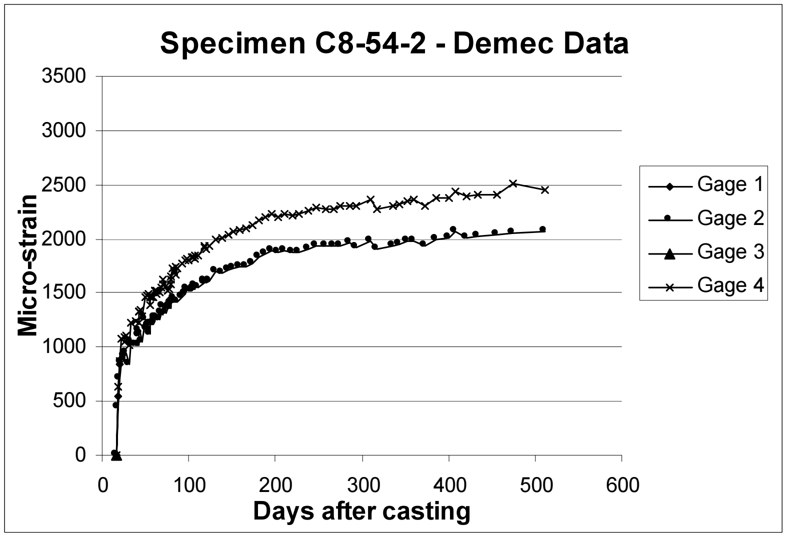


Figure 4.5 Mechanical Strain Gage Data for Specimen C8-54-1



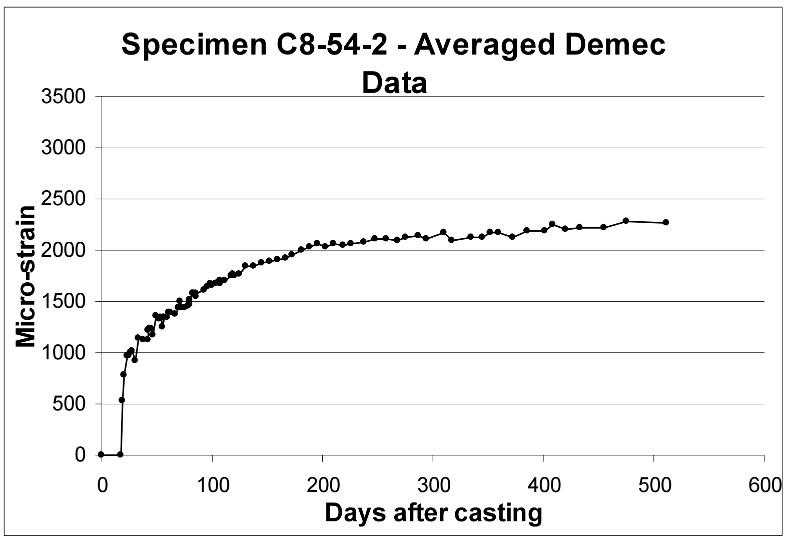
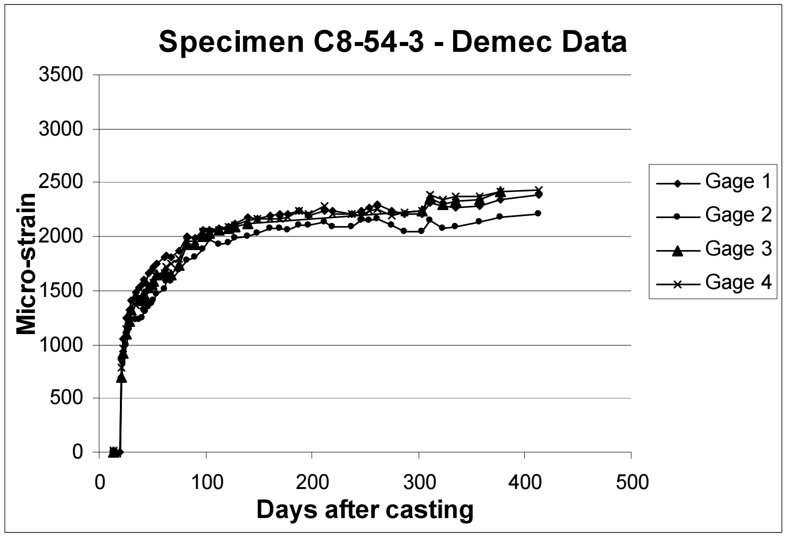


Figure 4.6 Mechanical Strain Gage Data for Specimen C8-54-2



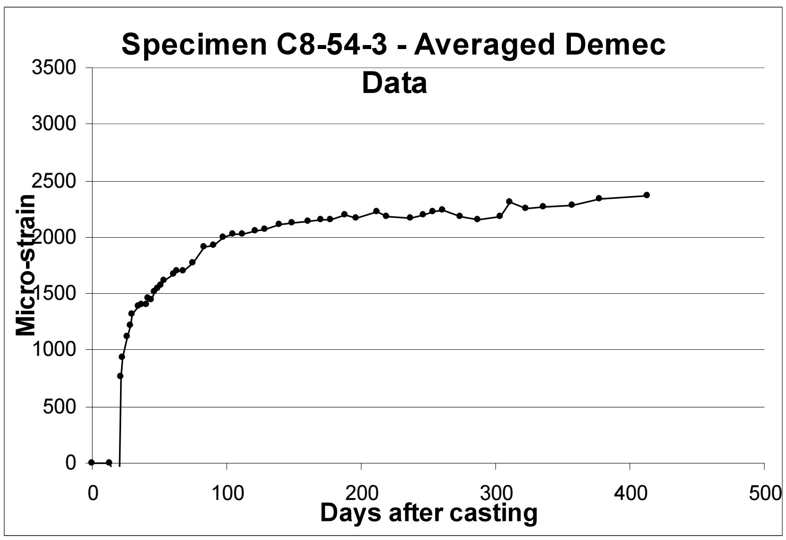
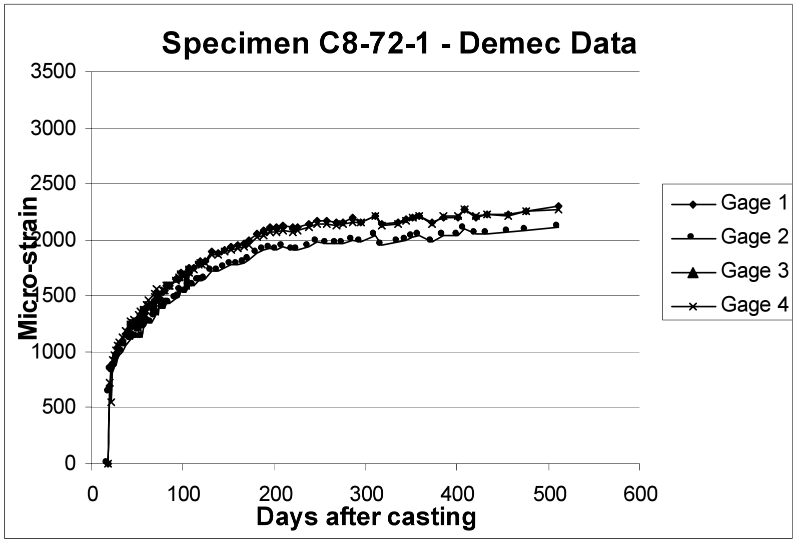


Figure 4.7 Mechanical Strain Gage Data for Specimen C8-54-3



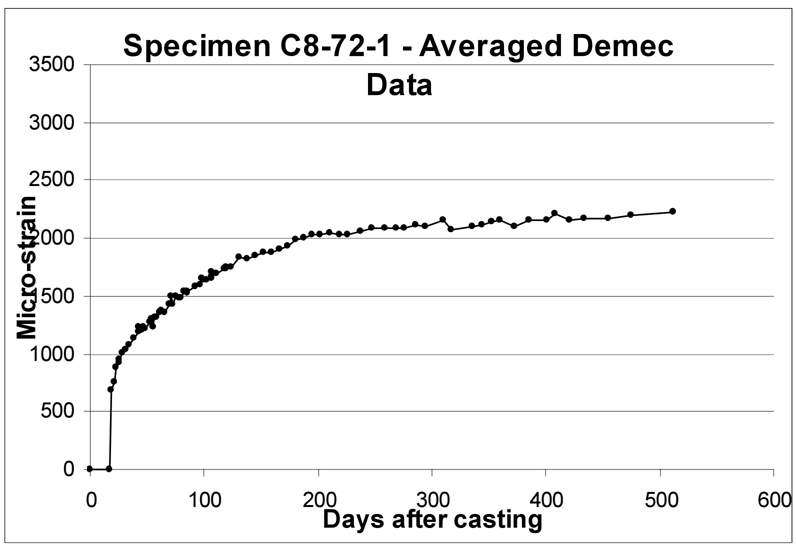
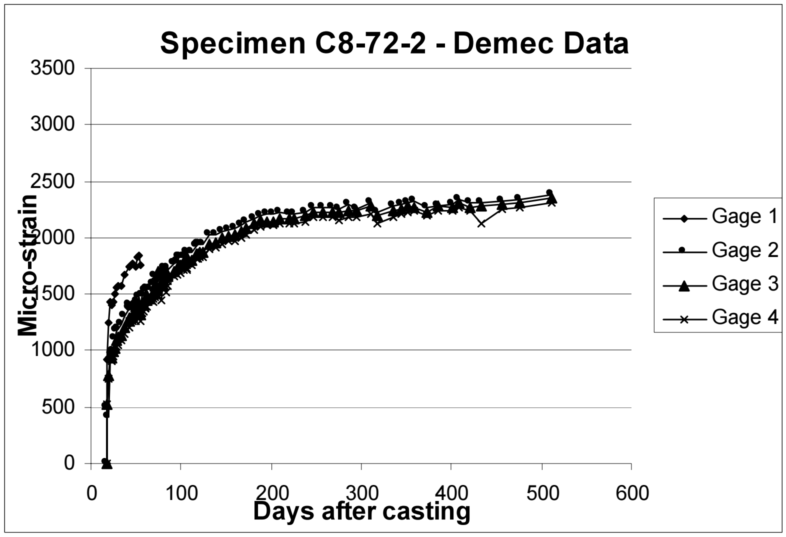


Figure 4.8 Mechanical Strain Gage Data for Specimen C8-72-1



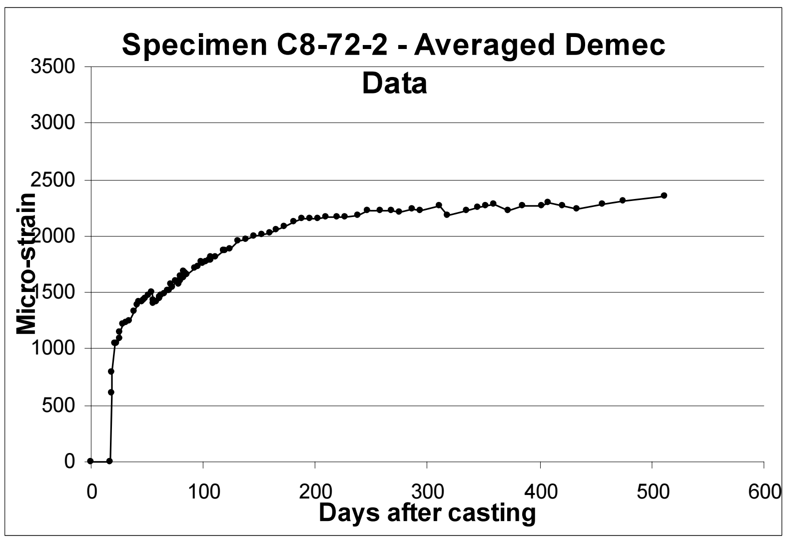
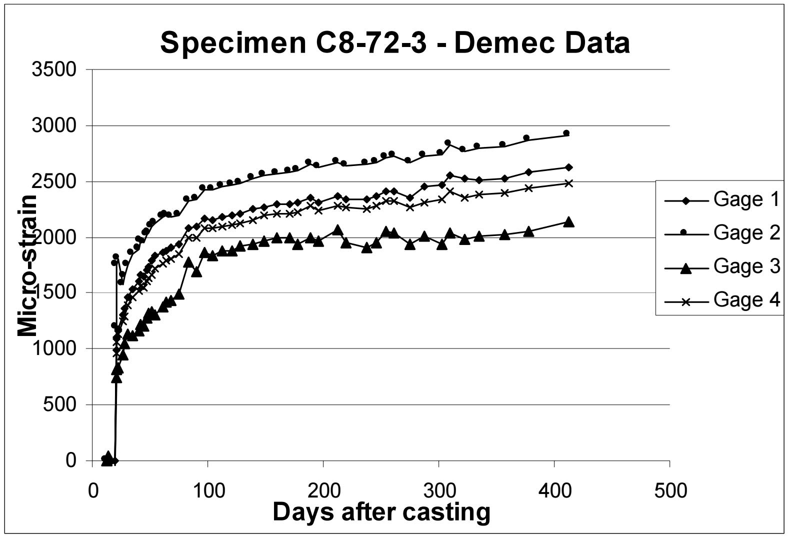


Figure 4.9 Mechanical Strain Gage Data for Specimen C8-72-2



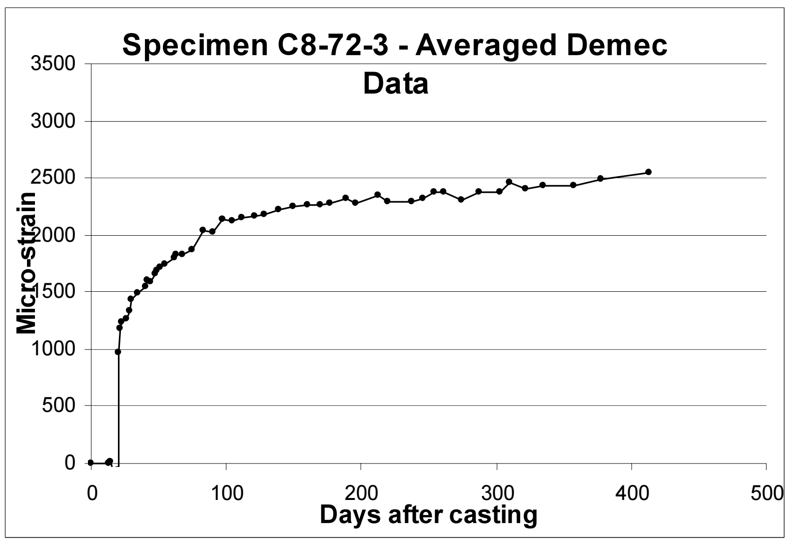


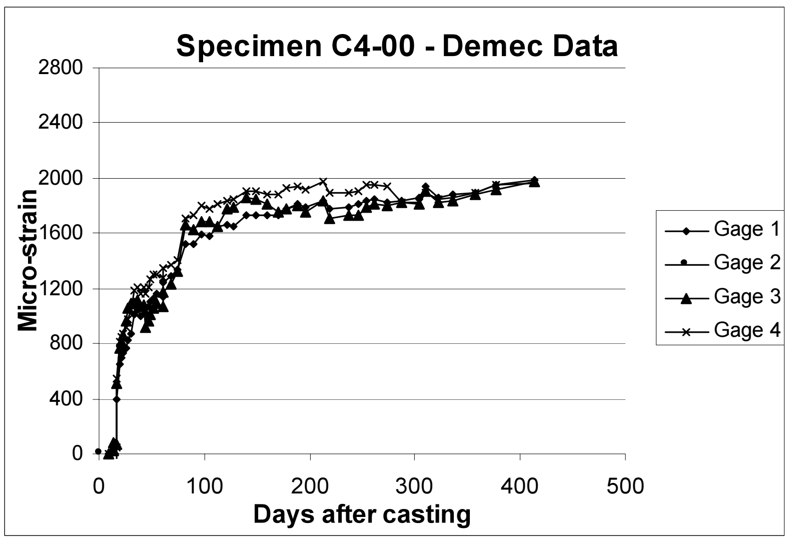
Figure 4.10 Mechanical Strain Gage Data for Specimen C8-72-3

4.2.1.2 4,000 psi

Strain data from the 4,000 psi concentrically-loaded specimens are presented in the following figures:

Reinf. ratio	Figures
0.0000	4.11
0.0036	4.12–4.14
0.0054	4.15–4.17
0.0072	4.18–4.20

Specimen C4-00 (Fig 4.11) had Demec gage 2 improperly installed so that it was unreadable. The other three gages produced data that showed good agreement.



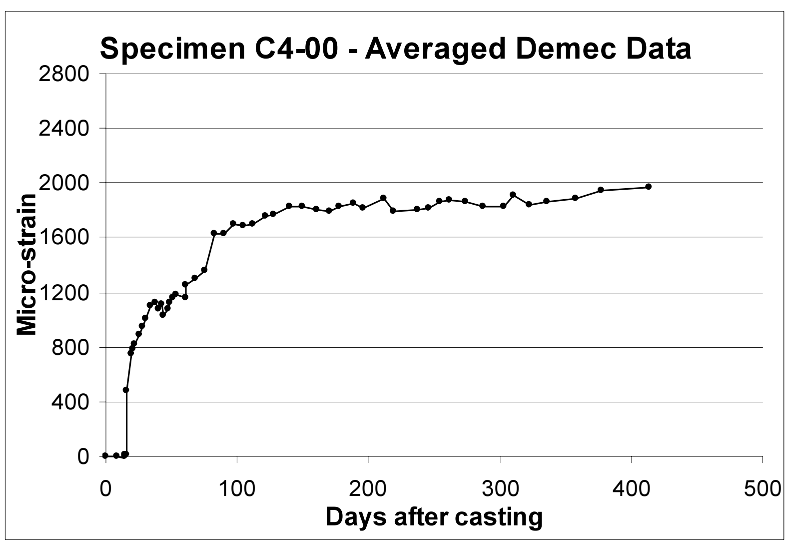
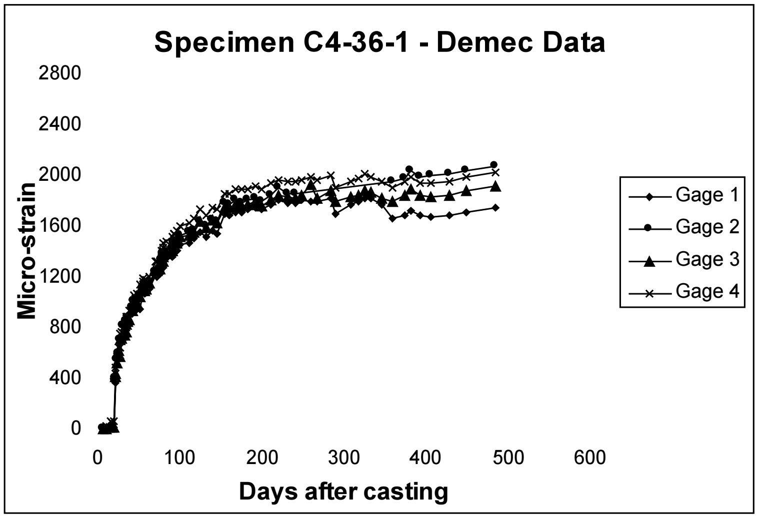


Figure 4.11 Mechanical Strain Gage Data for Specimen C4-00

Specimen C4-36-1's (Fig. 4.12) and C4-36-2's (Fig. 13) Demec gages produced data with little scatter. Specimen C4-36-3 (Fig. 4.14) likely had some eccentricity as indicated by the Demec gages.



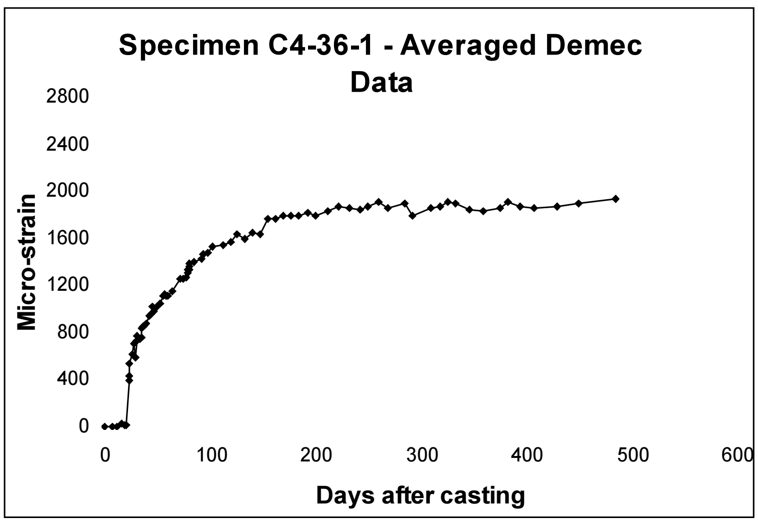
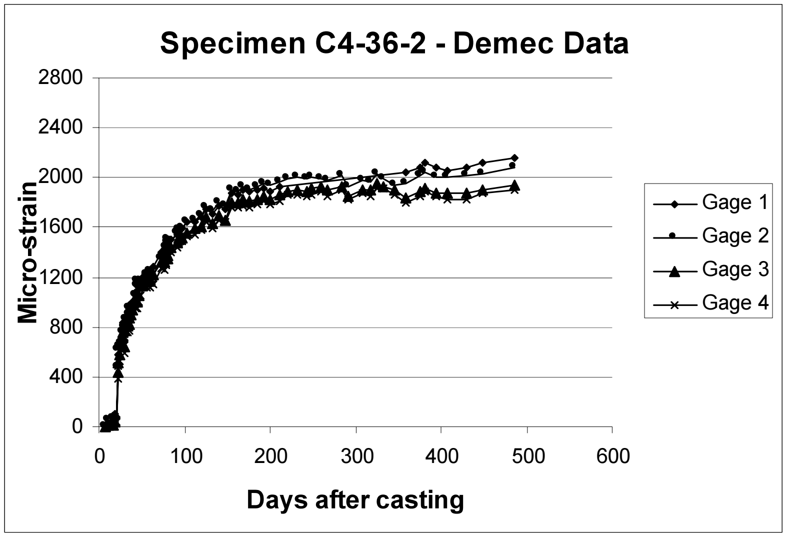


Figure 4.12 Mechanical Strain Gage Data for Specimen C4-36-1



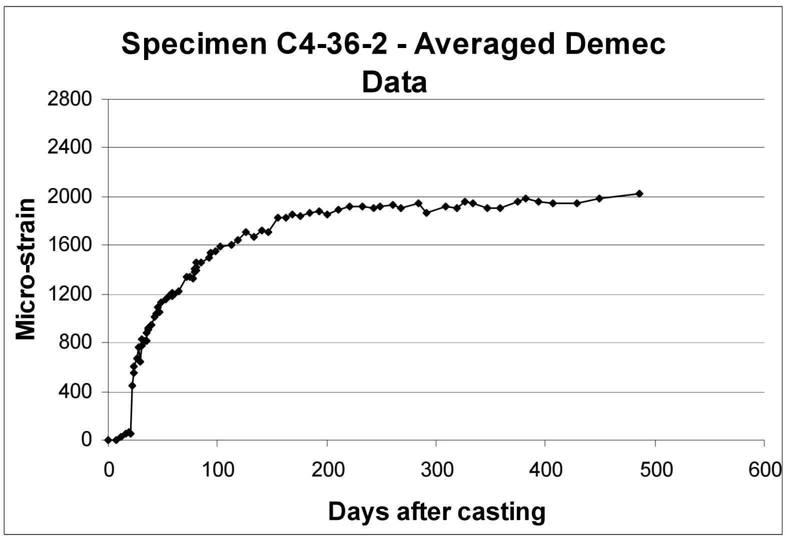
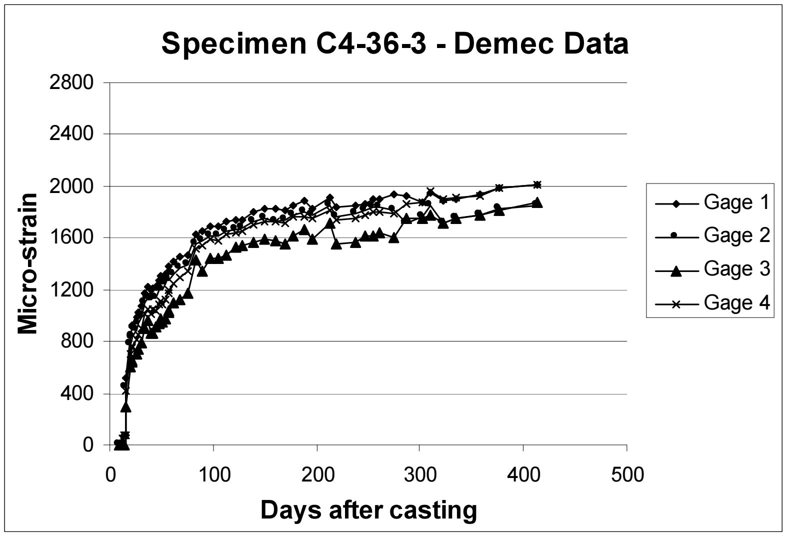


Figure 4.13 Mechanical Strain Gage Data for Specimen C4-36-2



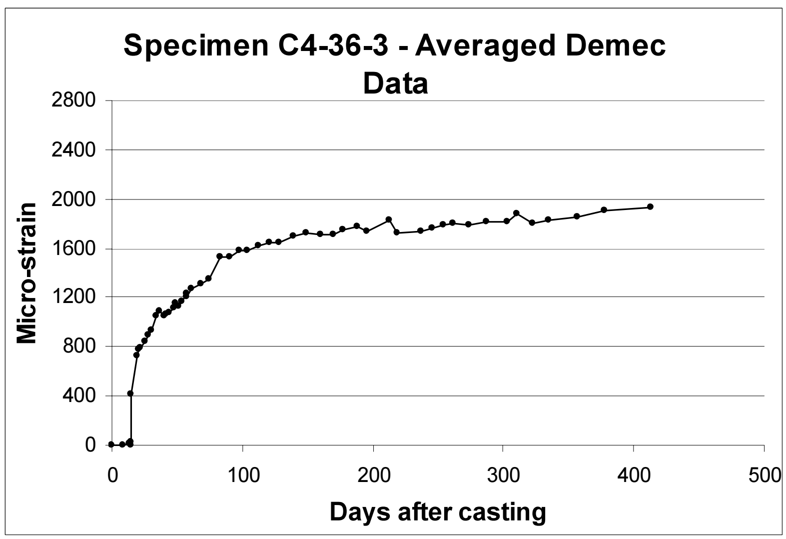
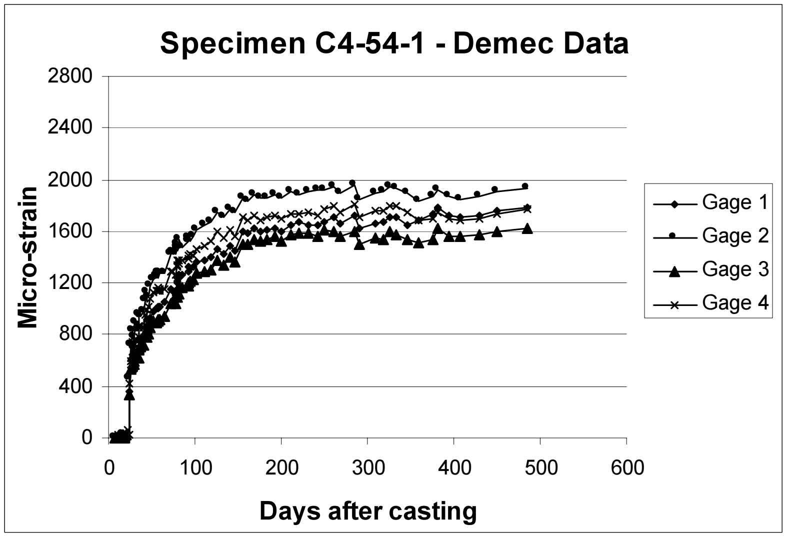


Figure 4.14 Mechanical Strain Gage Data for Specimen C4-36-3

Specimen C4-54-1's (Fig. 4.15) Demec data have some scatter most likely due to an eccentricity of the applied axial load. The Demec data of Specimen C4-54-2 (Fig. 4.16) indicated significant eccentricity of the applied axial load. Specimen C4-54-3's (Fig. 4.17) Demec gage 1 deformed beyond the capacity of the gage. Before doing so, gage 1 produced data that indicated some eccentricity of load was present. Because gage 1 produced the highest strains, the average strains after gage 1 failed would likely have been larger. The Demec data from the four gages on Specimen C4-72-1 (Fig. 4.18) indicated a large eccentricity of the applied axial load about the north-south axis. Specimen C4-72-2's (Fig. 4.19) Demec gage 1 showed a sudden reduction in strains soon after loading. This inconsistency is not compatible with the readings of the other three gages and is likely erroneous; therefore gage 1 was not used in computing the average strain response. Specimen C4-72-3 (Fig. 4.20) had Demec gage 4 installed improperly. The remaining three gages indicated an eccentricity that would have resulted in gage 4 measuring less-than-average strains. This means that the true average was probably less than that indicated in Figure 4.20.



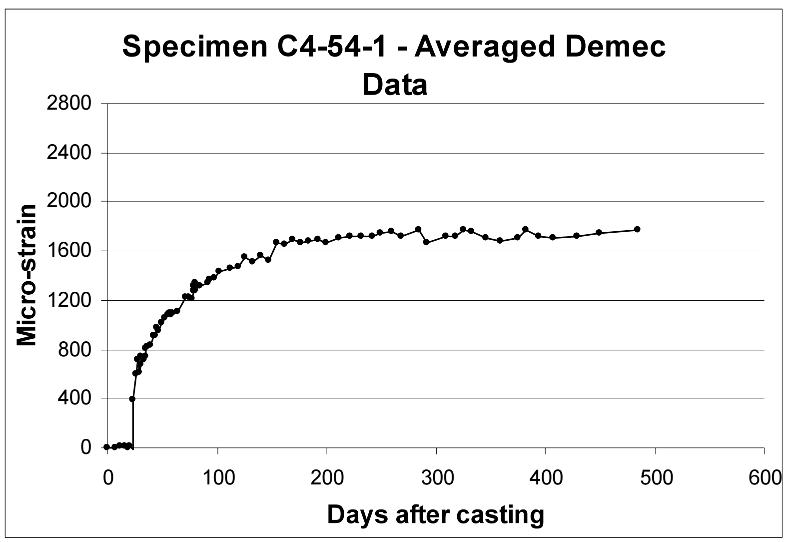
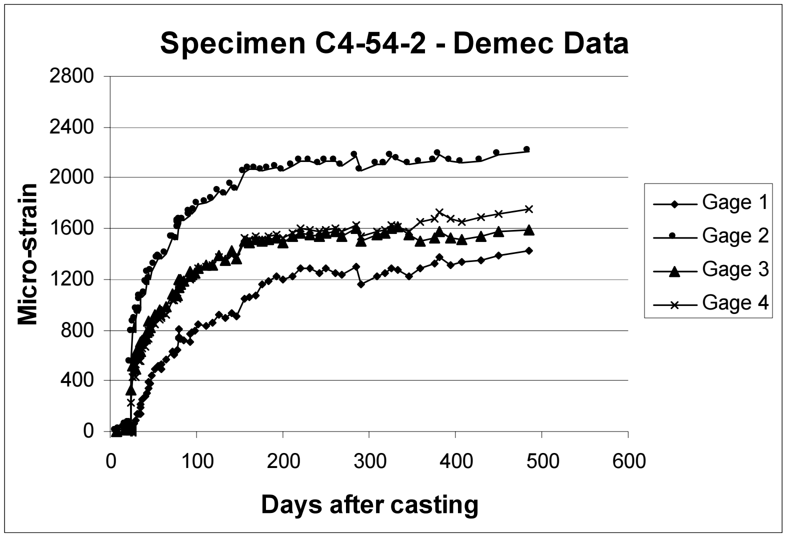


Figure 4.15 Mechanical Strain Gage Data for Specimen C4-54-1



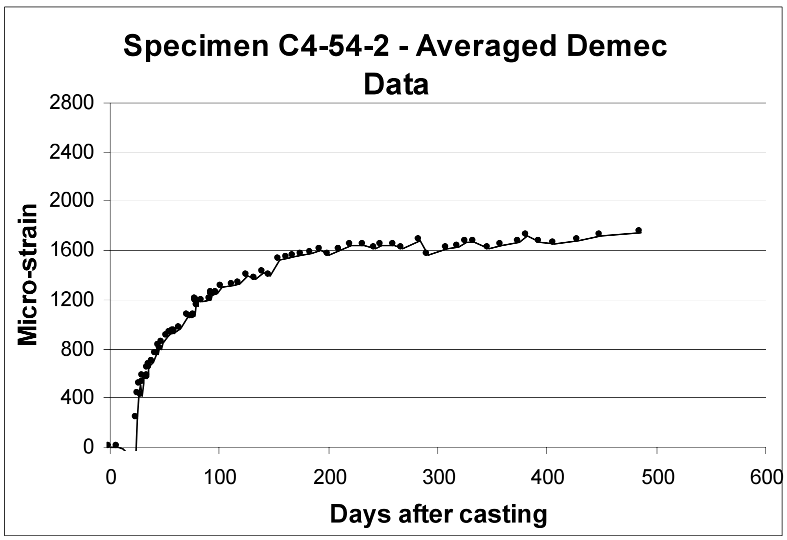
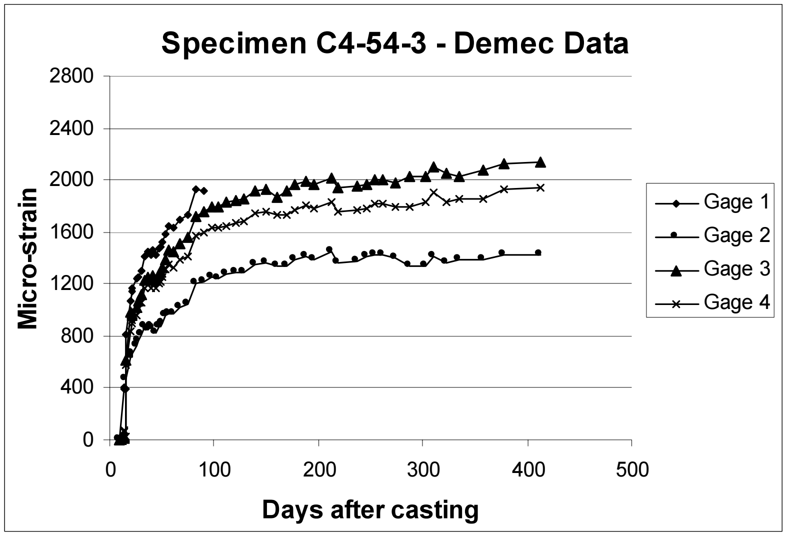


Figure 4.16 Mechanical Strain Gage Data for Specimen C4-54-2



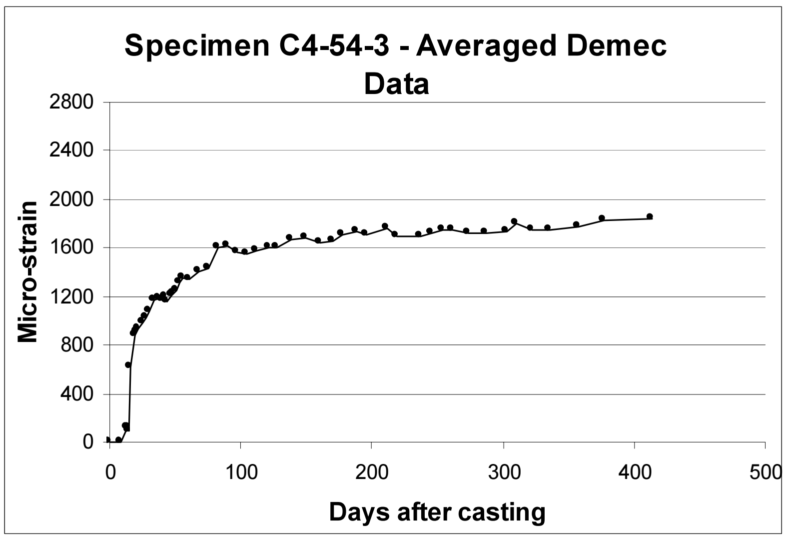


Figure 4.17 Mechanical Strain Gage Data for Specimen C4-54-3

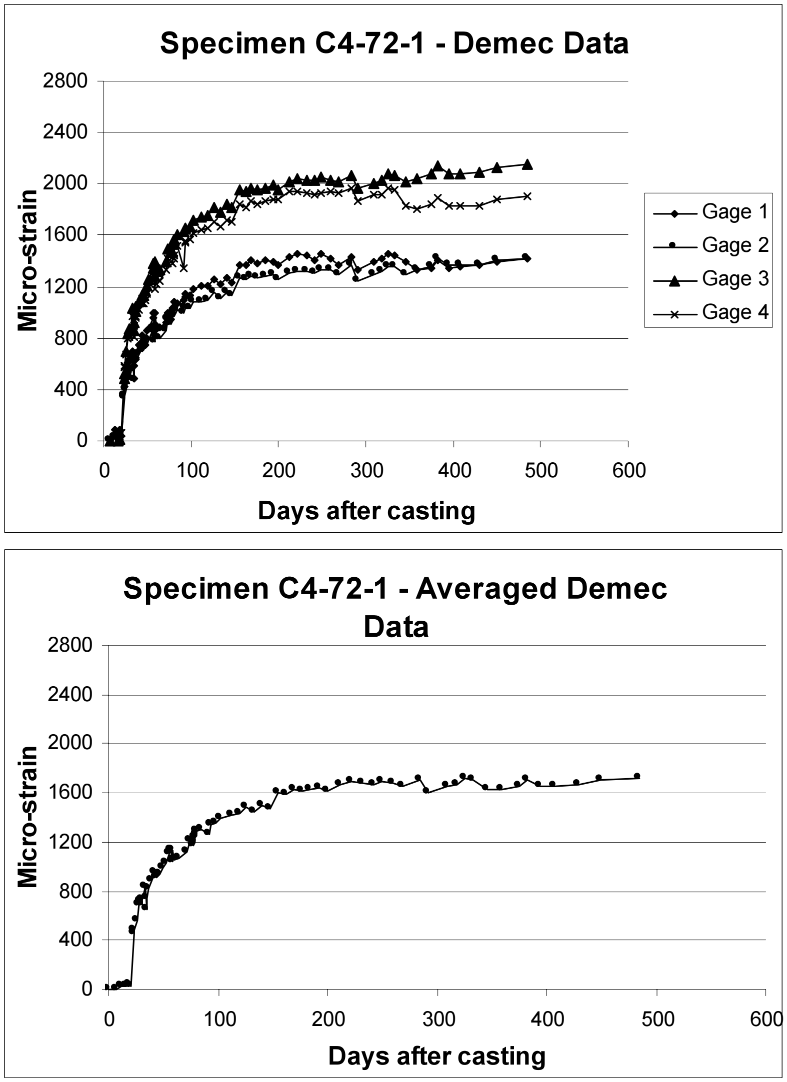
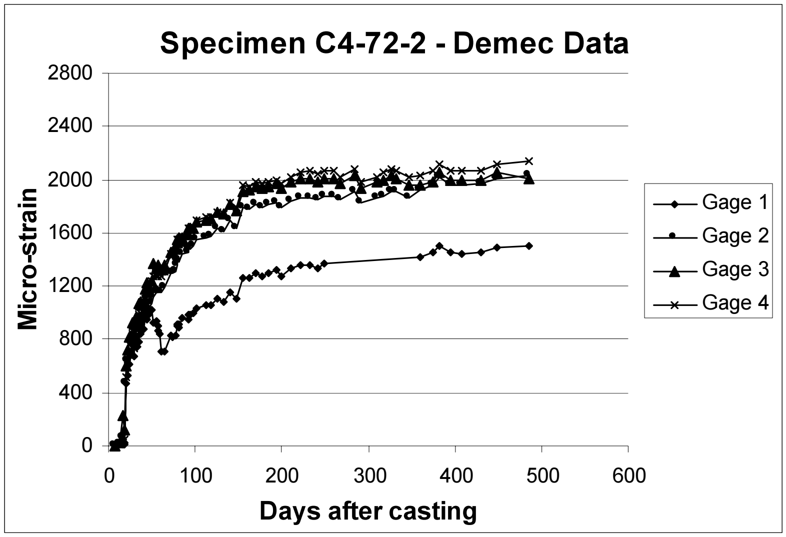


Figure 4.18 Mechanical Strain Gage Data for Specimen C4-72-1



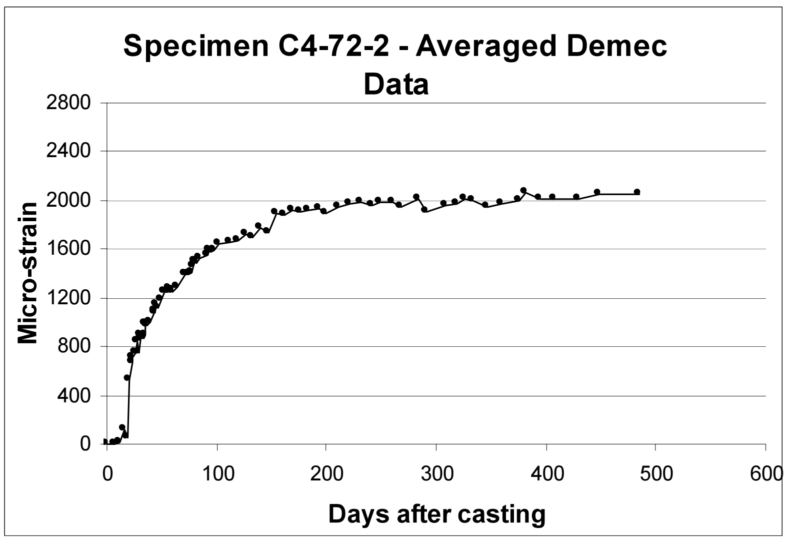
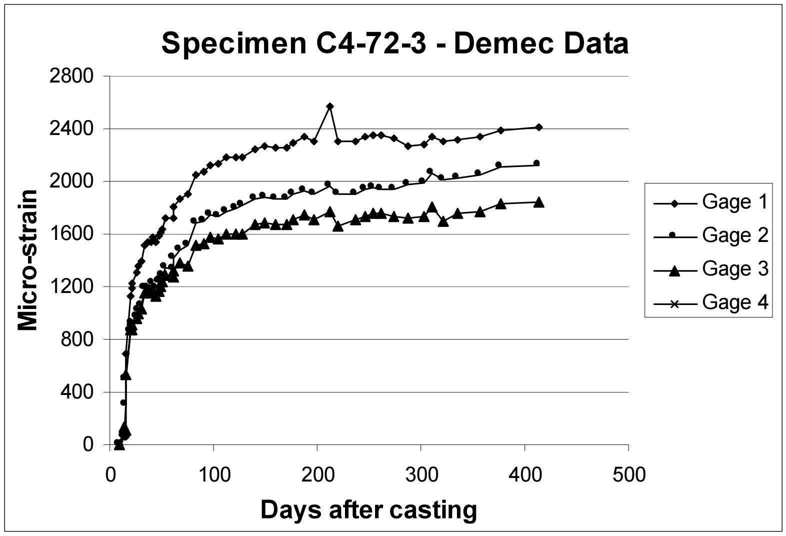


Figure 4.19 Mechanical Strain Gage Data for Specimen C4-72-2



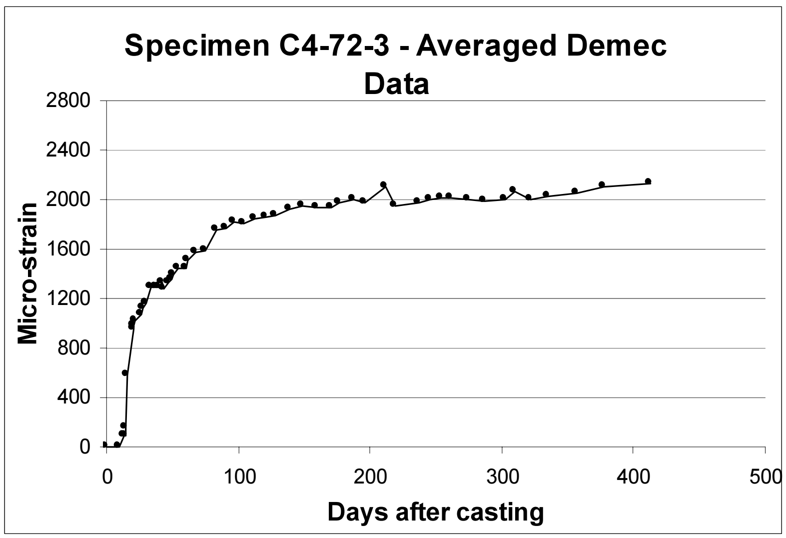


Figure 4.20 Mechanical Strain Gage Data for Specimen C4-72-3

4.2.1.3 General Discussion

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

4.2.2 Eccentrically Loaded Specimens

4.2.2.1 8,000 psi

Strain data from the 8,000 psi eccentrically-loaded specimens are presented in the following figures:

Reinf. ratio	Figures
0.0036	4.21
0.0072	4.22

Specimen E8-36's (Fig. 4.21) Demec gage readings clearly indicate effects of the eccentric axial load intentionally placed on the specimen. The Demec points for gage 4 deflected beyond the capacity of the gage after 80 days. Data up to 80 days from gages 3 and 4 were similar and thus, the average of readings from gages 3 and 4 would be expected to be similar to the data that are presented in Figure 4.21 if gage 4 had not stopped producing data. It should be noted that this is the specimen that was loaded to only $0.30f_c'A_g$, as compared with all other specimens which were loaded to $0.40f_c'A_g$. All of Specimen E8-72's (Fig. 4.22) Demec gages produced reasonable data. The effect of the eccentricity of the applied axial load is evident in the data.

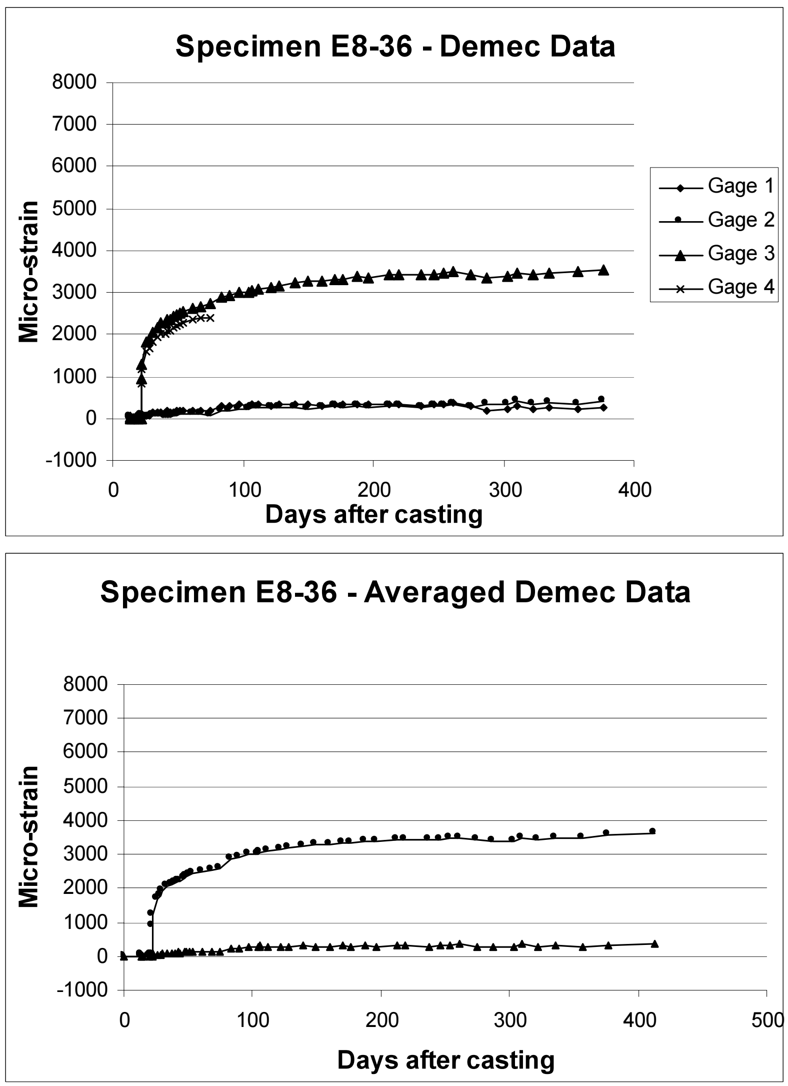


Figure 4.21 Mechanical Strain Gage Data for Specimen E8-36

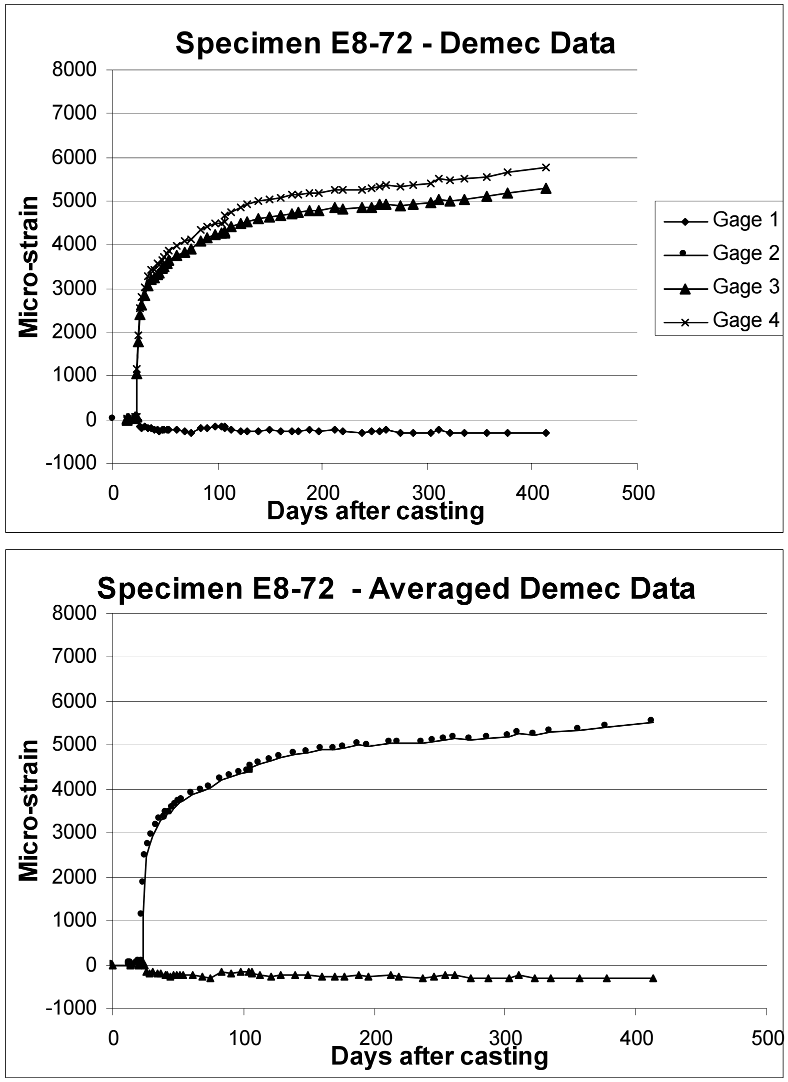


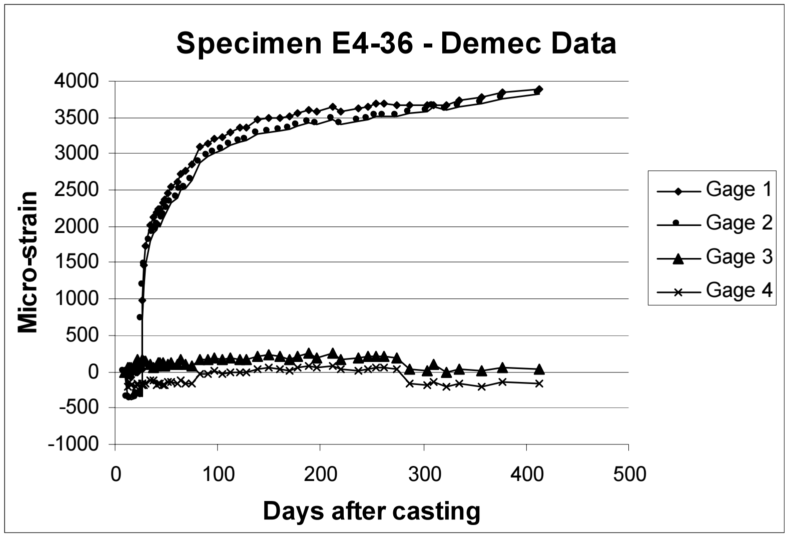
Figure 4.22 Mechanical Strain Gage Data for Specimen E8-72

4.2.2.2 4,000 psi

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

Reinf. ratio	Figures
0.0036	4.23
0.0072	4.24

The effect of the intentional eccentricity of the applied axial load is apparent in Specimen E4-36's (Fig. 4.23) Demec readings. The Demec data are reasonably consistent and have little scatter. The intentionally-eccentric load is also apparent in Specimen E4-72's (Fig. 4.24) Demec readings. There also seems to be an additional unintended eccentricity that caused the strains measured by gage 1 to be larger than the strains measured by gage 2, and the strains measured by gage 4 to be less than the strains measured by gage 3. Gage 1's Demec points deflected beyond the capacity of the gage and so data are missing after day 70. Data from gage 1 would have likely continued to be greater than those from gage 2 and thus, the average should be higher than presented.



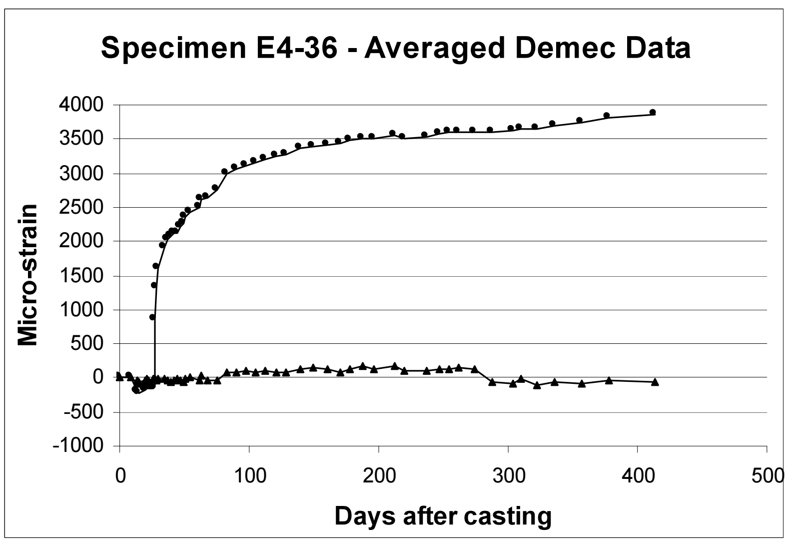
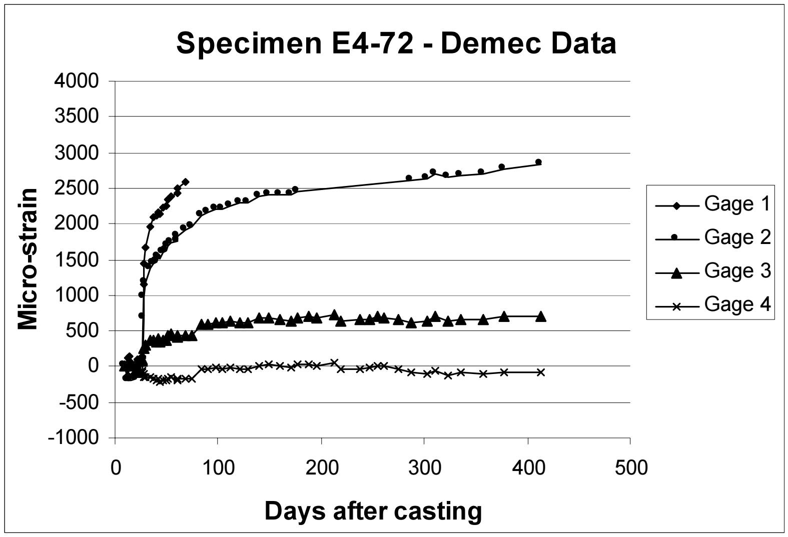


Figure 4.23 Mechanical Strain Gage Data for Specimen E4-36



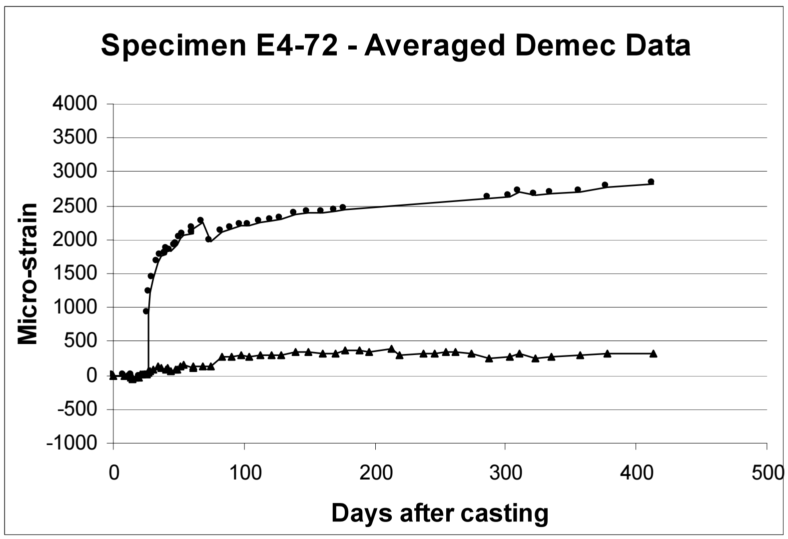


Figure 4.24 Mechanical Strain Gage Data for Specimen E4-72

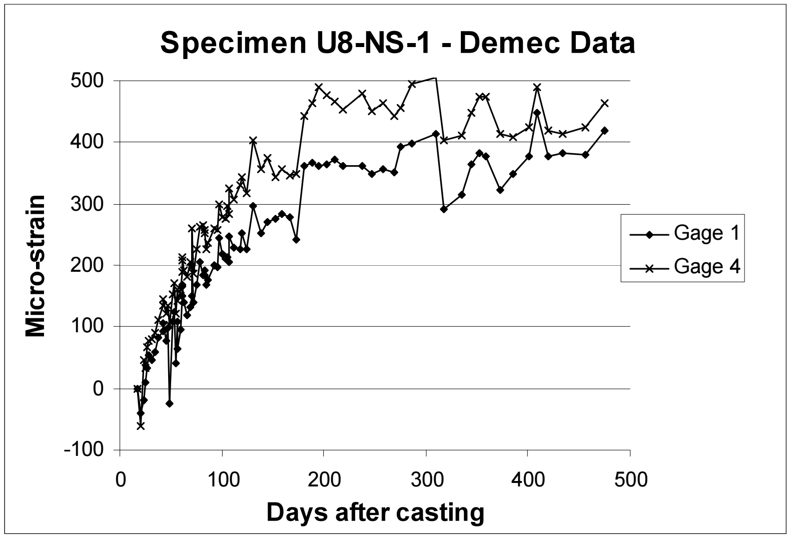
4.2.3 UNLOADED SPECIMENS

4.2.3.1 8,000 psi

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

Reinf. ratio	Figures
0.00NS	4.25–4.26
0.0000	4.27–4.28
0.0036	4.29
0.0054	4.30
0.0072	4.31

Specimen U8-NS-1 (Fig. 4.25) had reasonable datta produced by both Demec gages. Specimen U8-NS-2's (Fig. 4.26) Demec gages produced data with significant unexplained scatter. Specimen U8-00-1's (Fig. 4.27) Demec gages produced data with little scatter. Both of Specimen U8-00-2's (Fig. 4.28) Demec gages produced data, but the data showed significant scatter. Some scatter is present in the data for Specimen U8-36 (Fig. 4.29). Specimen U8-54's (Fig. 4.30) Demec gages produced data with significant scatter, although the data from the two gages followed the same trend. Both the Demec gages in Specimen U8-72 (Fig 4.31) were readable but they produced data with significant scatter.



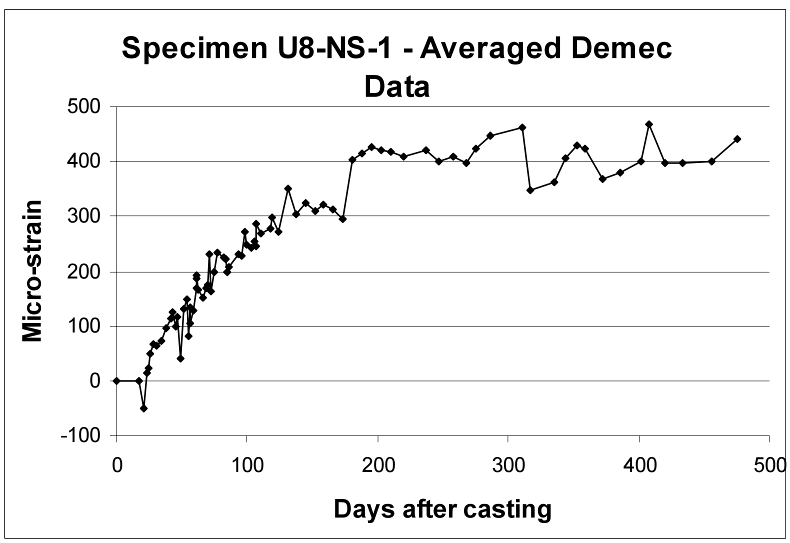
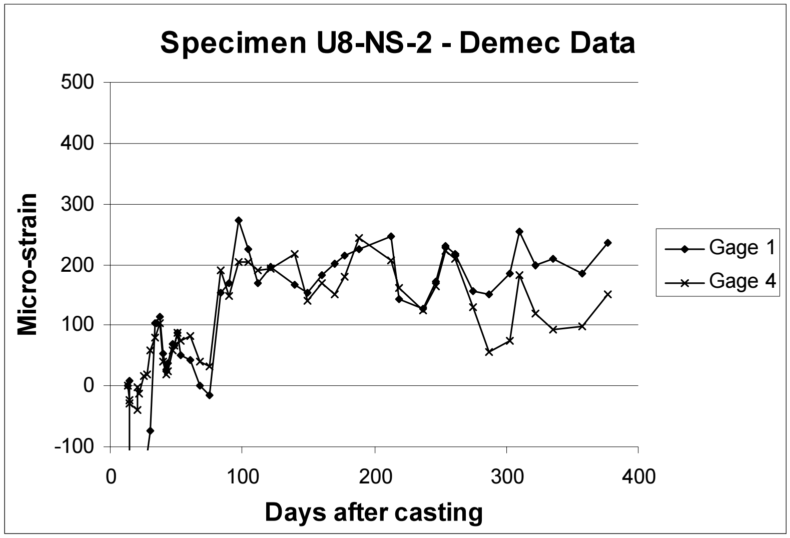


Figure 4.25 Mechanical Strain Gage Data for Specimen U8-NS-1



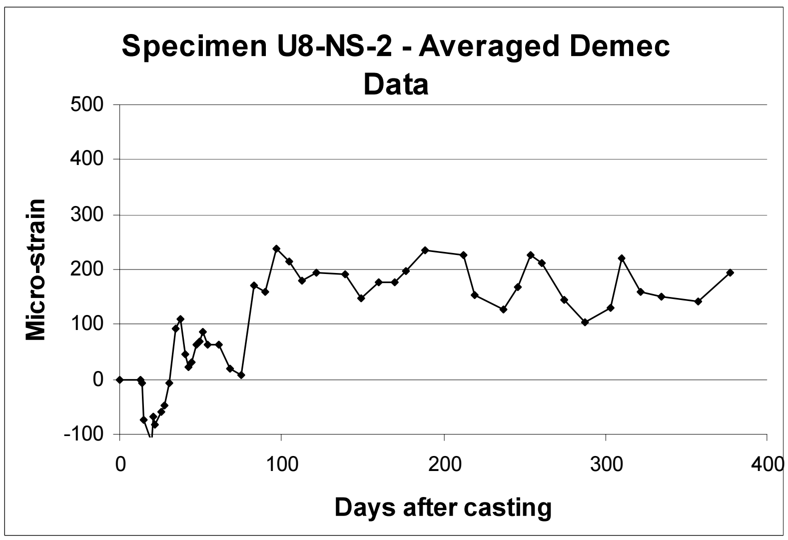
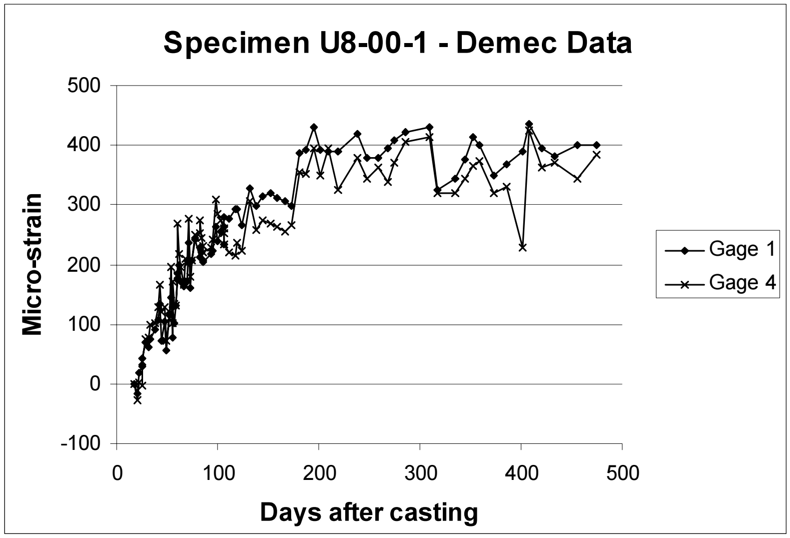


Figure 4.26 Mechanical Strain Gage Data for Specimen U8-NS-2



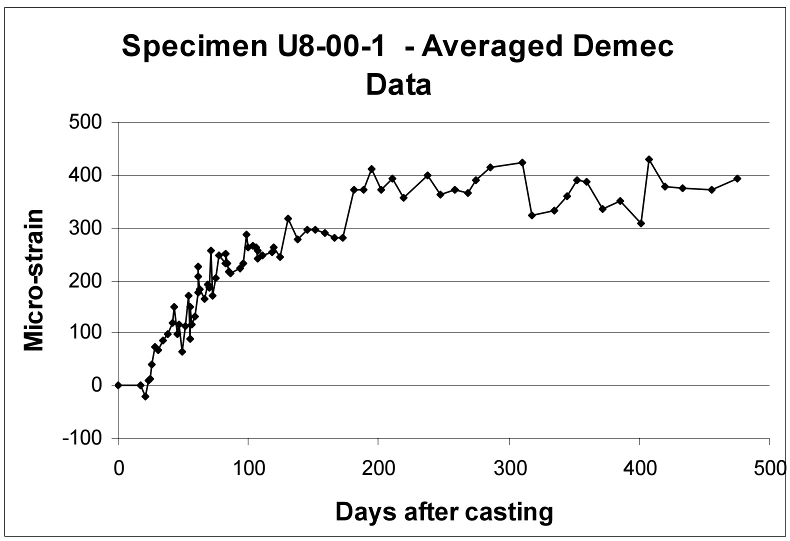
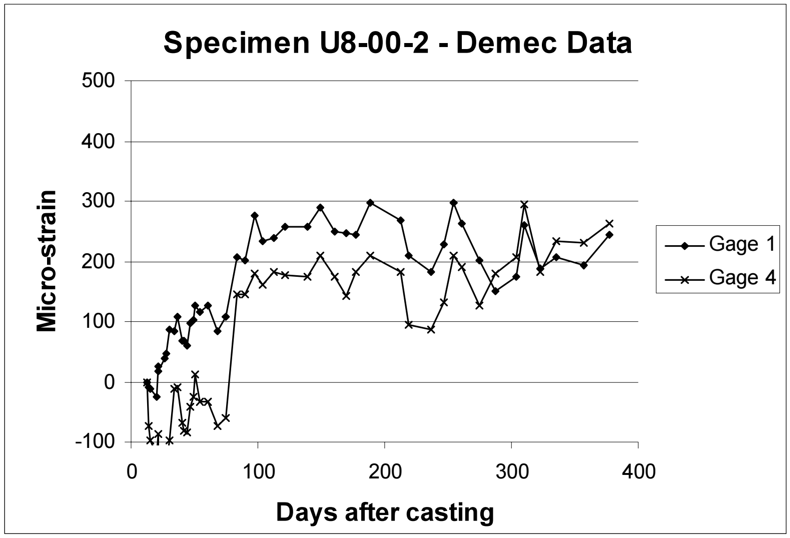


Figure 4.27 Mechanical Strain Gage Data for Specimen U8-00-1



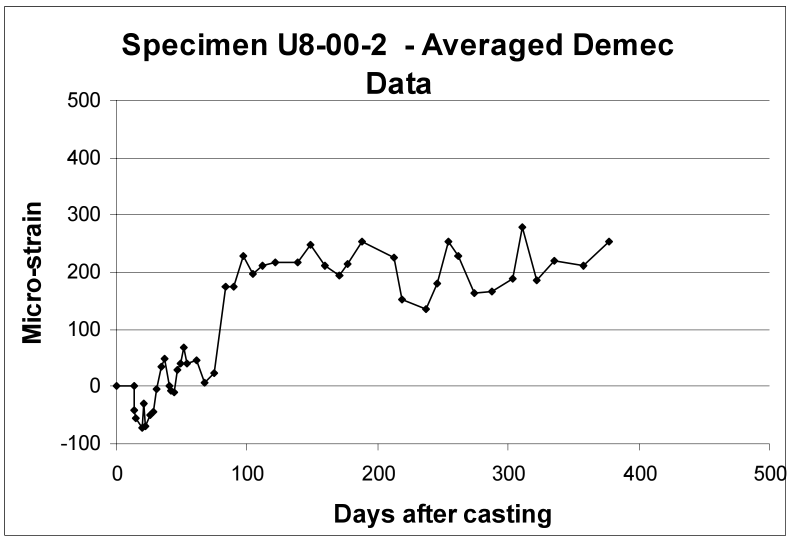
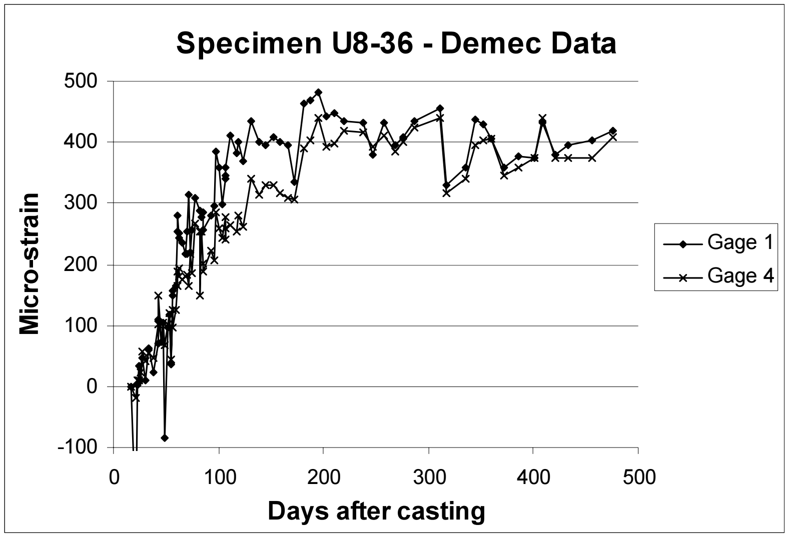


Figure 4.28 Mechanical Strain Gage Data for Specimen U8-00-2



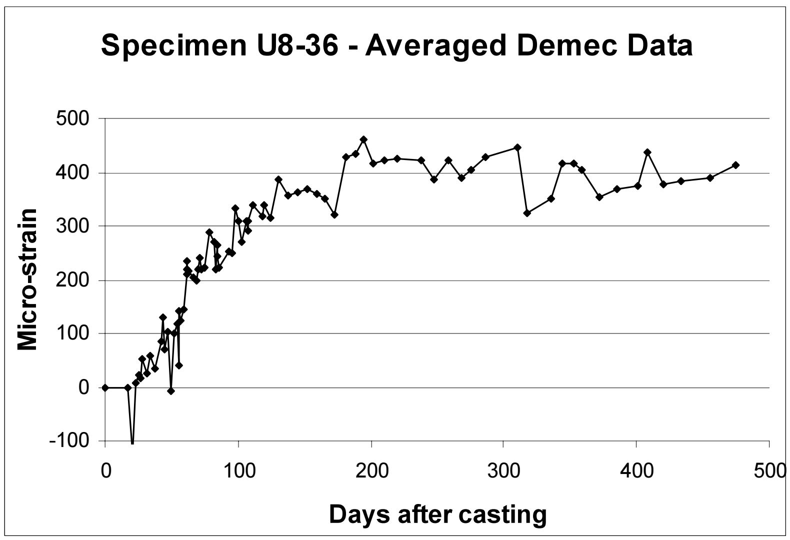
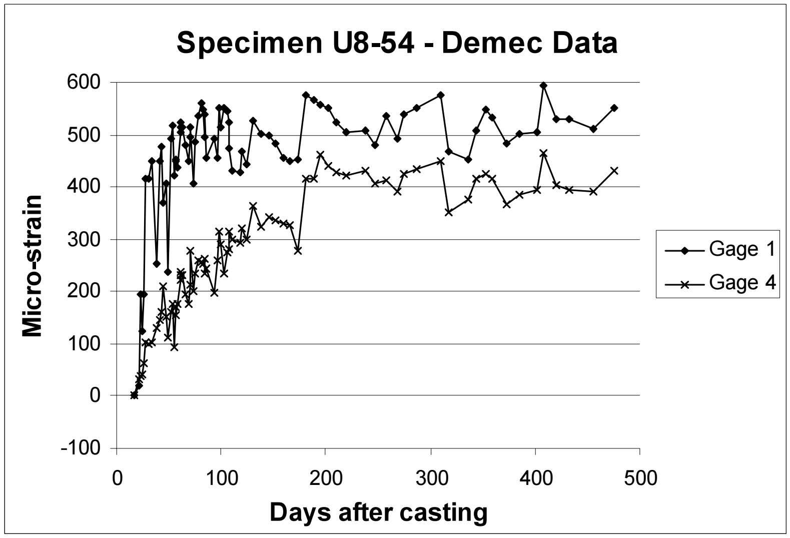


Figure 4.29 Mechanical Strain Gage Data for Specimen U8-36



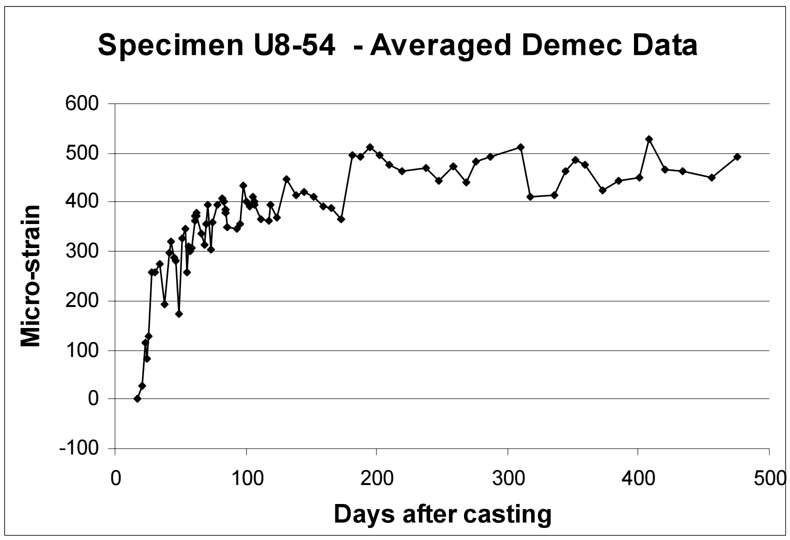
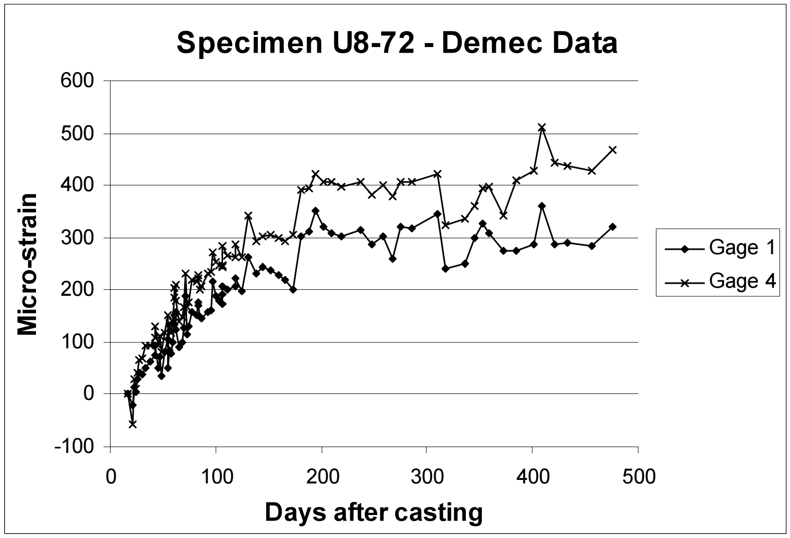


Figure 4.30 Mechanical Strain Gage Data for Specimen U8-54



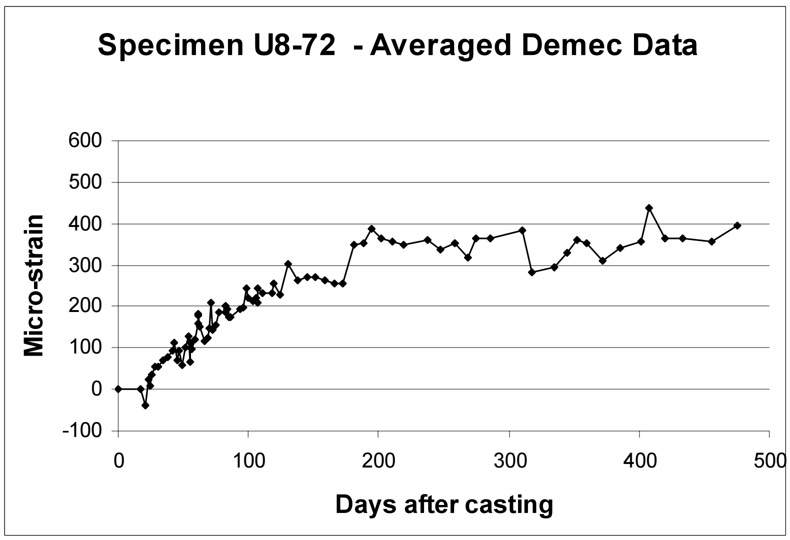


Figure 4.31 Mechanical Strain Gage Data for Specimen U8-72

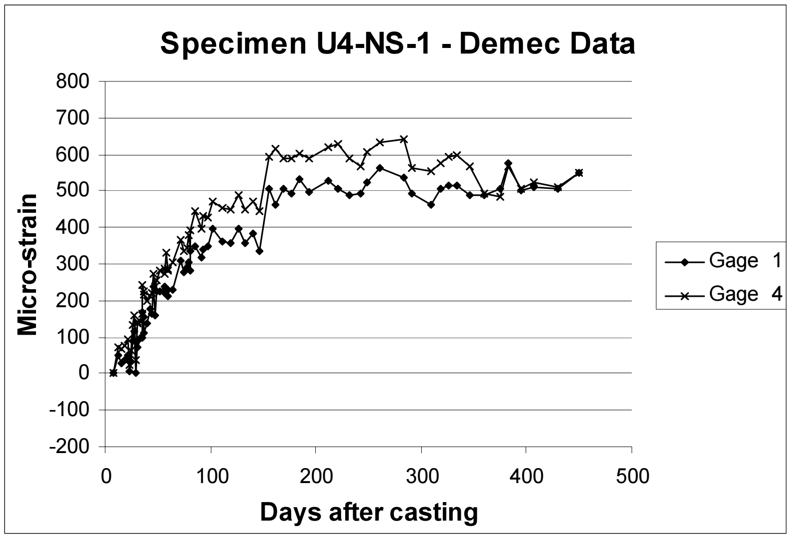
4.2.3.2 4,000 psi

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

Reinf. ratio	Figures
0.00NS	4.32–4.33
0.0000	4.34–4.35
0.0036	4.36
0.0054	4.37
0.0072	4.38

Specimen U4-NS-1's (Fig. 4.32) Demec gages provided strain data with little scatter. Only one of Specimen U4-NS-2's (Fig. 4.33) Demec gages was readable. Specimen U4-00-1's

(Fig. 4.34) Demec gages produced data with significant scatter. The two Demec gages in Specimen U4-00-2 (Fig. 4.35) produced data with little scatter. Specimen U4-36's (Fig. 4.36) Demec data from the two gages in the specimen had little scatter. Specimen U4-54's (Fig. 4.37) Demec gages produced data with very little scatter. Both Demec gages from Specimen U4-72 (Fig. 4.38) produced very consistent data.



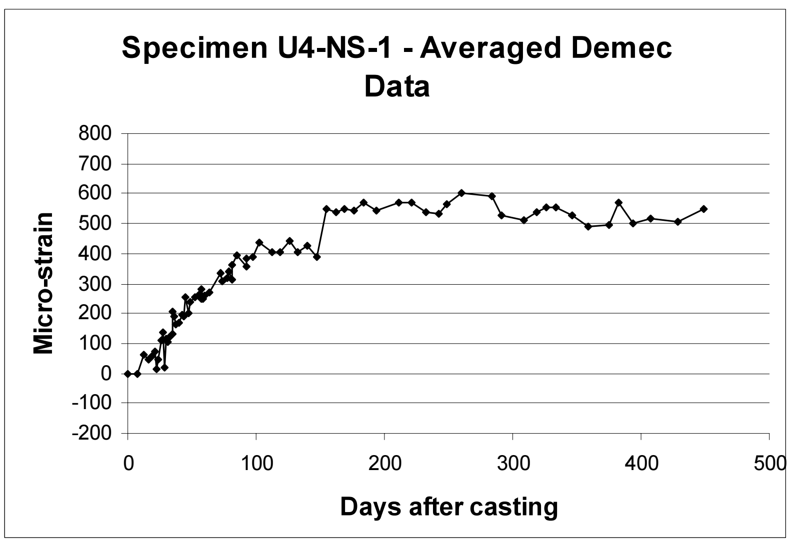
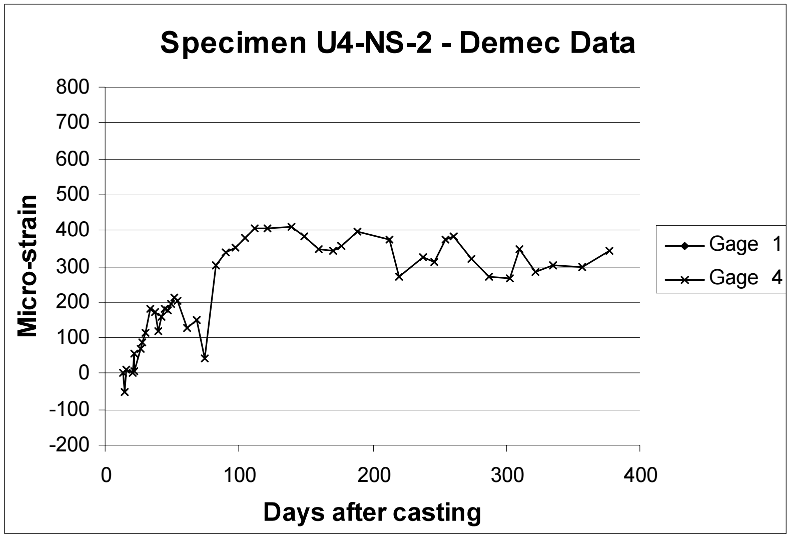


Figure 4.32 Mechanical Strain Gage Data for Specimen U4-NS-1



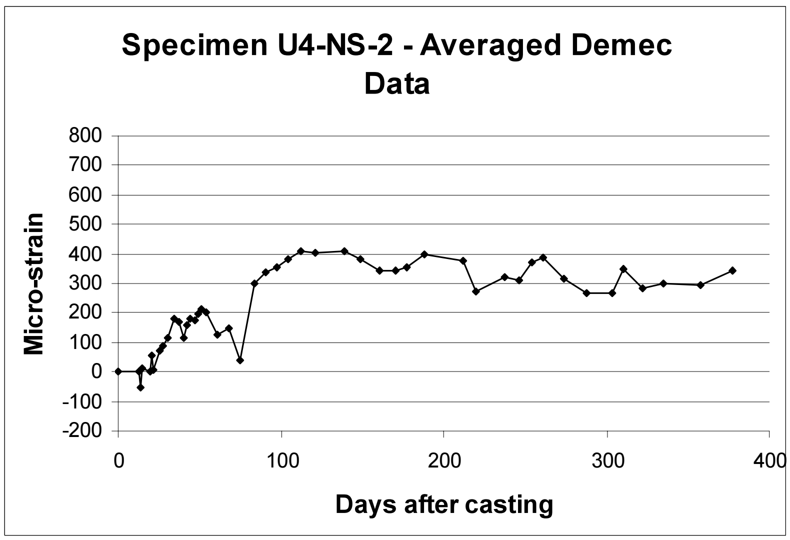
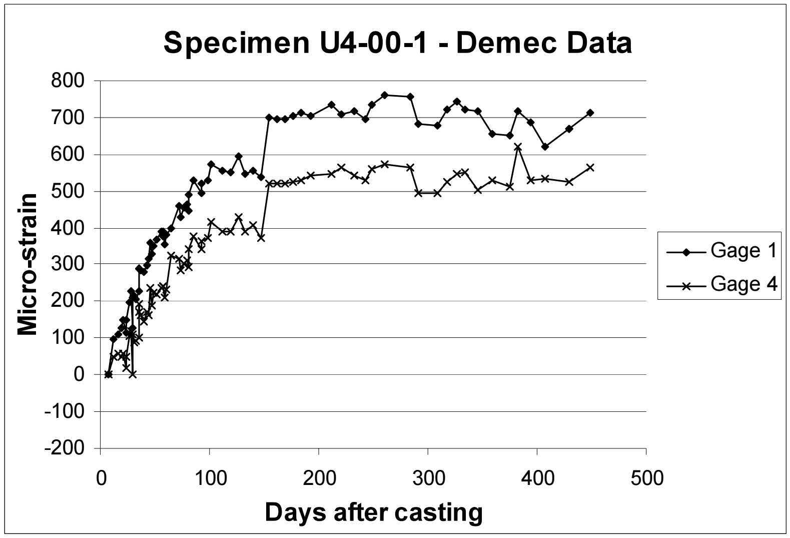


Figure 4.33 Mechanical Strain Gage Data for Specimen U4-NS-2



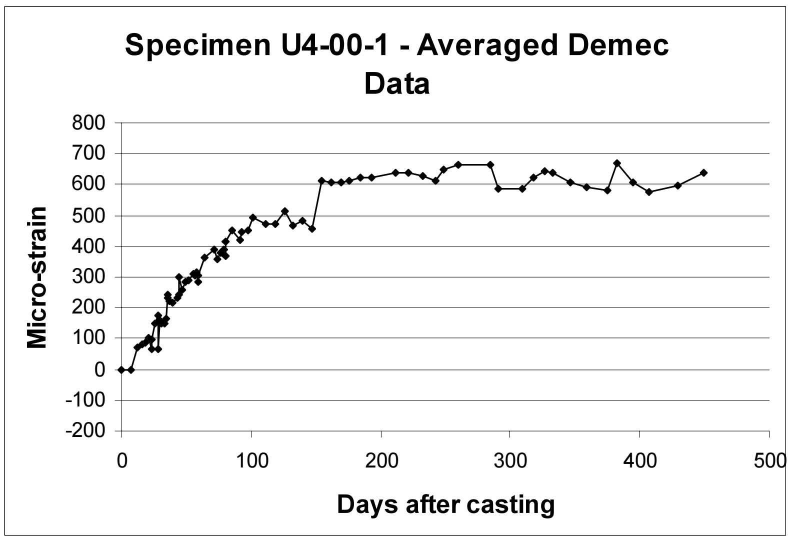
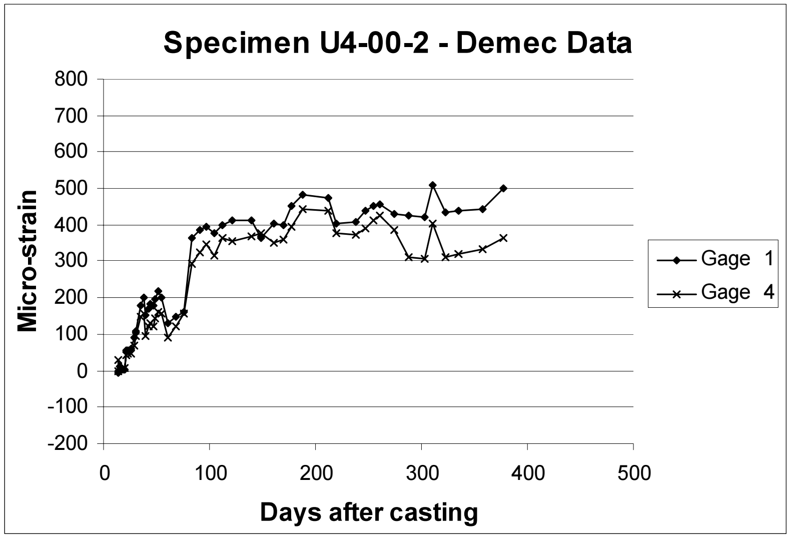


Figure 4.34 Mechanical Strain Gage Data for Specimen U4-00-1



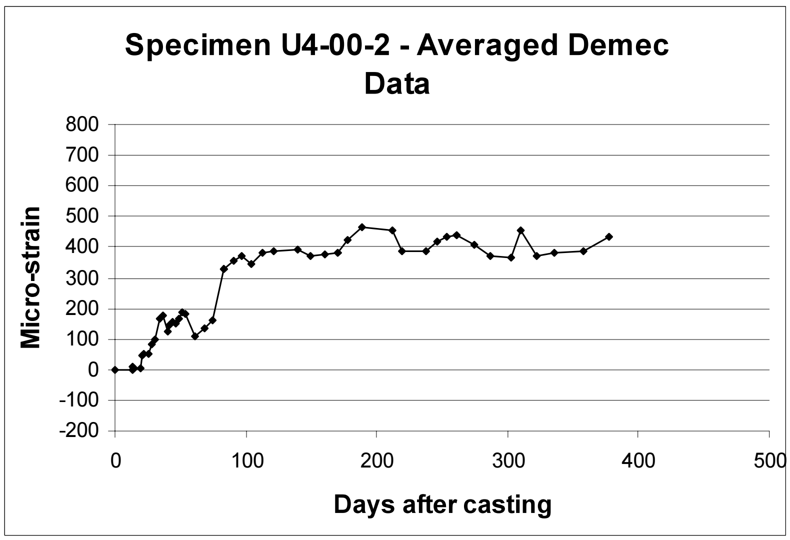
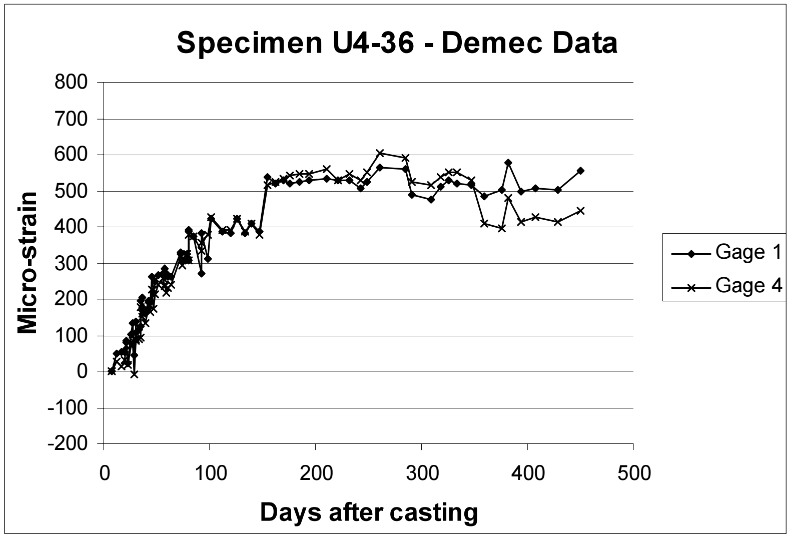


Figure 4.35 Mechanical Strain Gage Data for Specimen U4-00-2



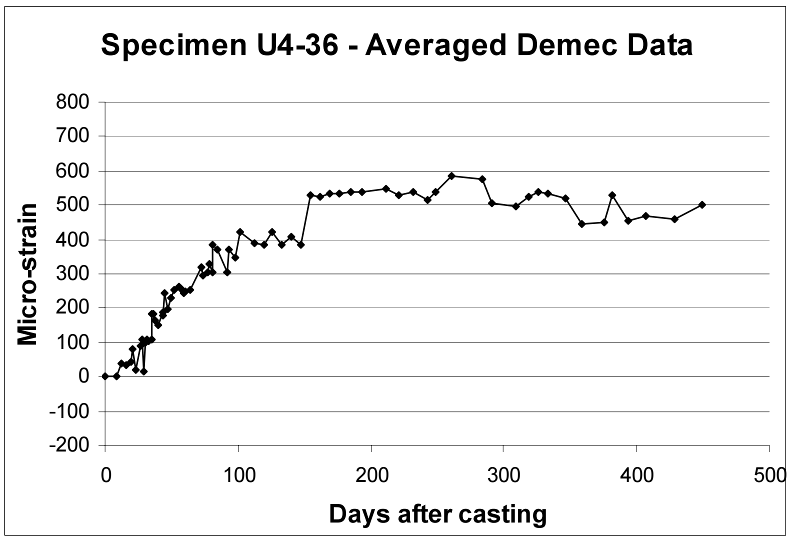
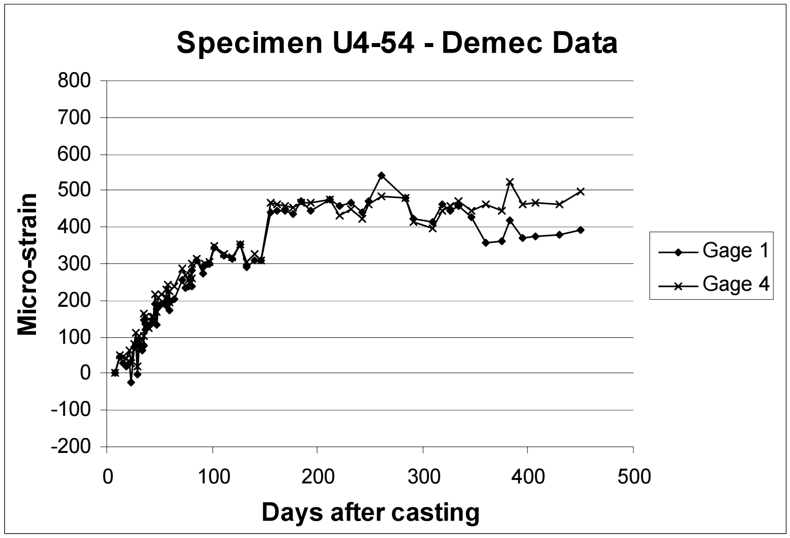


Figure 4.36 Mechanical Strain Gage Data for Specimen U4--36



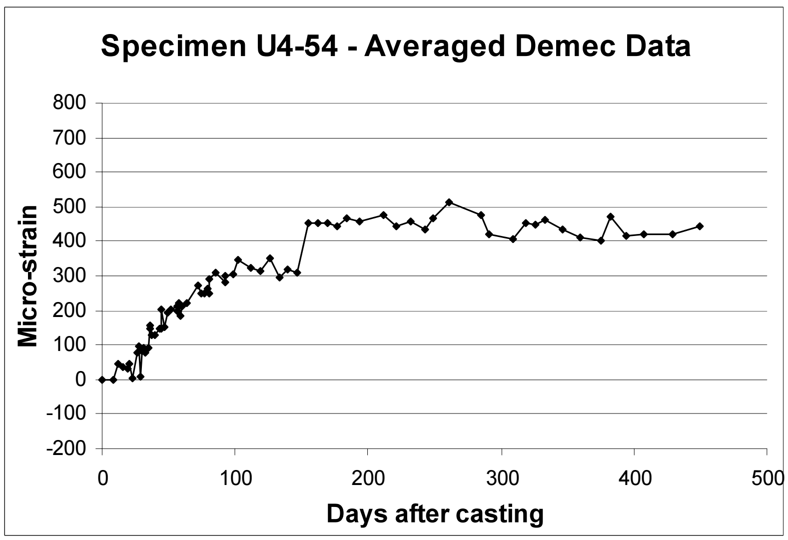
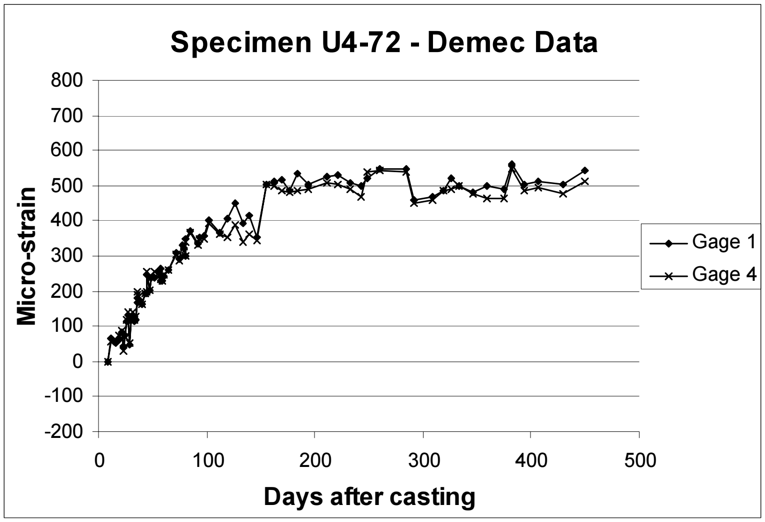


Figure 4.37 Mechanical Strain Gage Data for Specimen U4-54



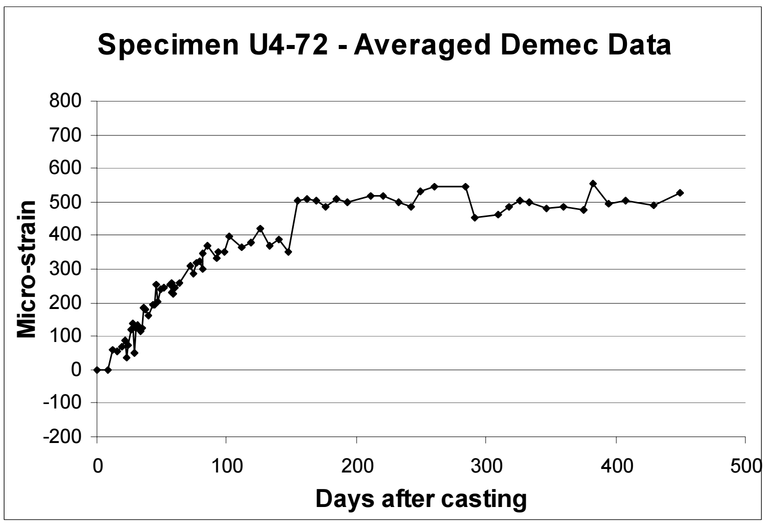


Figure 4.38 Mechanical Strain Gage Data for Specimen U4-72

4.3 COMPARISON OF CONCENTRICALLY LOADED SPECIMENS

In comparing the strain responses for various specimens, only the average Demec readings were considered. This was done to simplify the necessary graphs. For purposes of comparison, the results for the concentrically-loaded specimens are shown together in Figures 4.39 through 4.46. The data for the specimens are presented as follows:

Concrete Strength	Reinf. Ratio	Figure
8,000 psi	All	4.39
8,000 psi	0.0036	4.40
8,000 psi	0.0054	4.41
8,000 psi	0.0072	4.42
4,000 psi	All	4.43
4,000 psi	0.0036	4.44
4,000 psi	0.0054	4.45
4,000 psi	0.0072	4.46

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

4.3.1.1 8,000 psi

Average strain responses for the 8,000 psi concentrically-loaded specimens are presented in Figure 4.39. Because the specimens of Group 2 had lower concrete strengths than the specimens in Group 1, yet were loaded the same, they tended to experience greater strains. This difference is most notable early in the loading period. The specimens from Group 2 then appeared to creep at a slower rate than specimens from Group 1. It should be noted that the average strain from all nine specimens reached beyond 2,070 micro-strain, the nominal yield strain of 60 ksi steel.

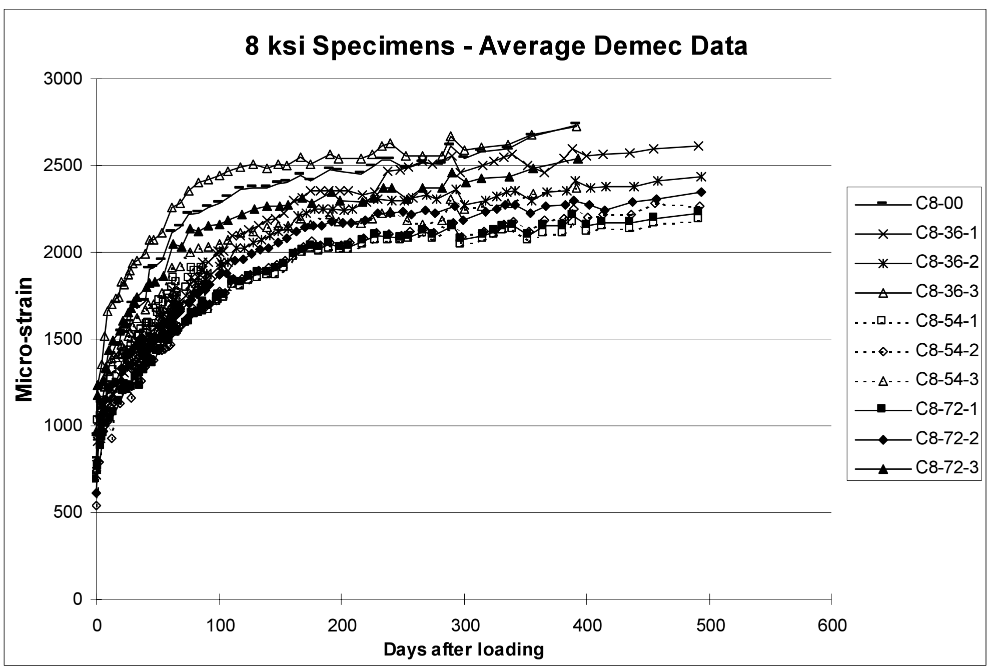


Figure 4.39 Comparison of 8 ksi Specimens

To clearly illustrate the difference between responses of specimens with the same longitudinal reinforcement ratio, strain data from specimens with equal reinforcement ratios were plotted in the same figure. Figure 4.40 shows the response of specimens having a reinforcement ratio of 0.0036. Specimen C8-36-3 is from Group 2 and clearly has greater initial strains than Specimens C8-36-1 and C8-36-2. The other two specimens had concrete strengths significantly greater than their design strengths yet still experienced total average axial strains well above 2,070 micro-strain.

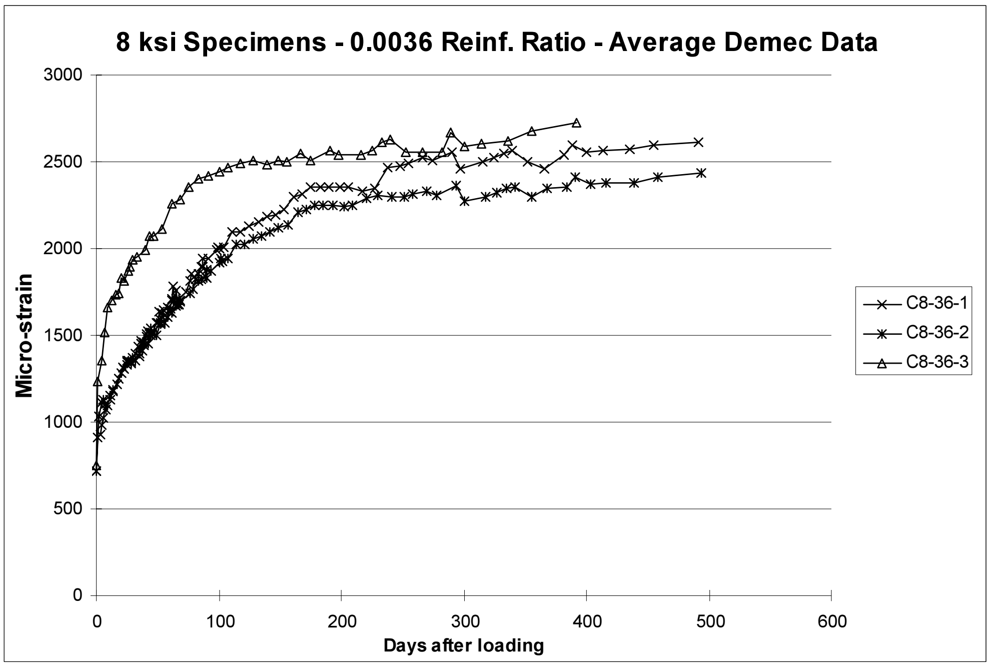


Figure 4.40 Comparison of 8 ksi Specimens–0.0036 Reinforcement Ratio

Figure 4.41 illustrates average strain responses from specimens with a reinforcement ratio of 0.0054. The response of Group 2 Specimen C8-54-3 and Group 1 Specimen C8-54-1 were nearly the same for the first 100 days of loading. The response of C8-54-1 then dropped suddenly due to a failed gage as explained in Section 4.2. All three specimens experienced maximum average strains greater than 2,070 micro-strain.

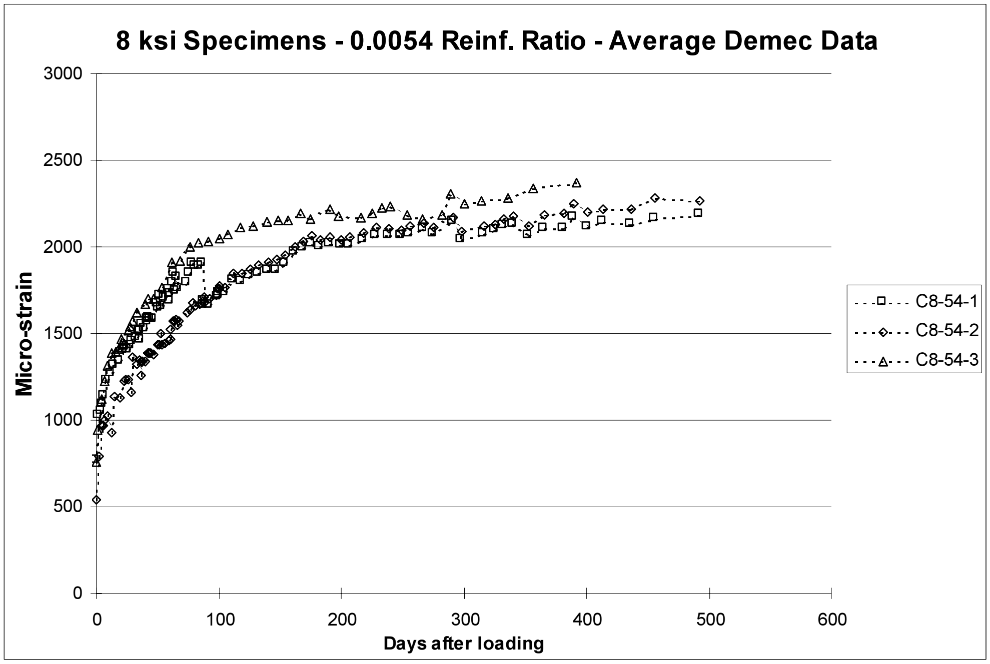


Figure 4.41 Comparison of 8 ksi Specimens–0.0054 Reinforcement Ratio

Figure 4.42 shows the average strain responses of specimens with reinforcement ratios of 0.0072. Specimen C8-72-3 was a Group 2 specimen with lower strength concrete, and had significantly larger initial and maximum strains, as expected. All three specimens experienced average strains greater than 2,070 micro-strain.

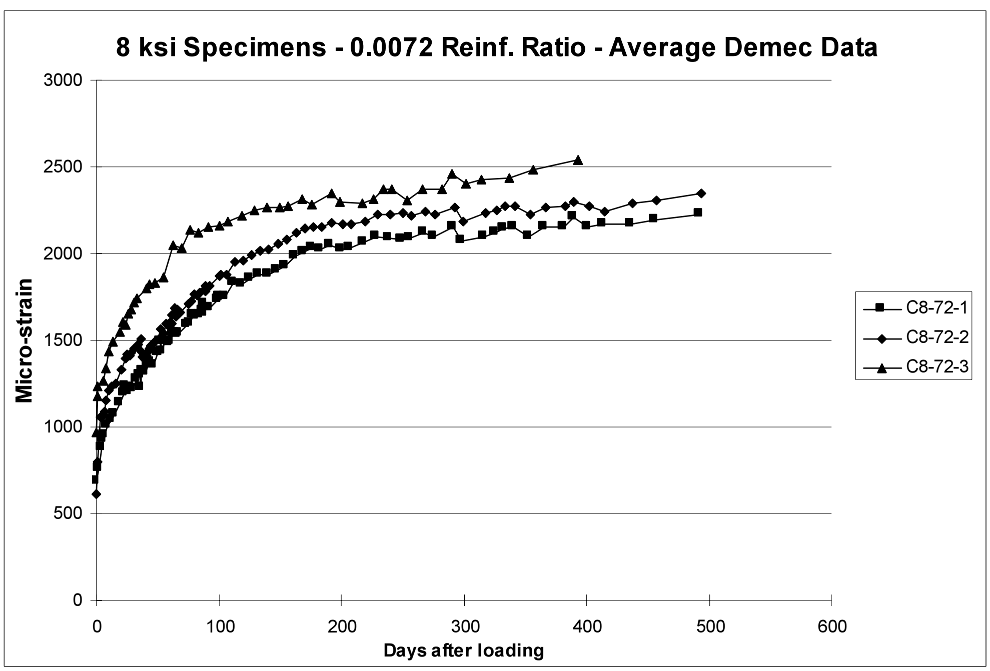


Figure 4.42 Comparison of 8 ksi Specimens–0.0072 Reinforcement Ratio

4.3.1.2 4,000 psi

A summary of the average axial strain responses for the 4,000 psi concentrically-loaded specimens is presented in Figure 4.43. Because the Group 4 specimens had lower concrete strengths than Group 3 specimens, yet were loaded the same, they tended to experience greater strains. This difference is less noticeable than for the 8,000 psi specimens because the difference in strengths was much smaller for the 4000 psi specimens. It should be noted that the maximum average strain response for only one of the nine specimens exceeded 2,070 micro-strain. Some of the specimen responses came very close to reaching 2,070 micro-strain and likely would have exceeded it with a few more months of loading.

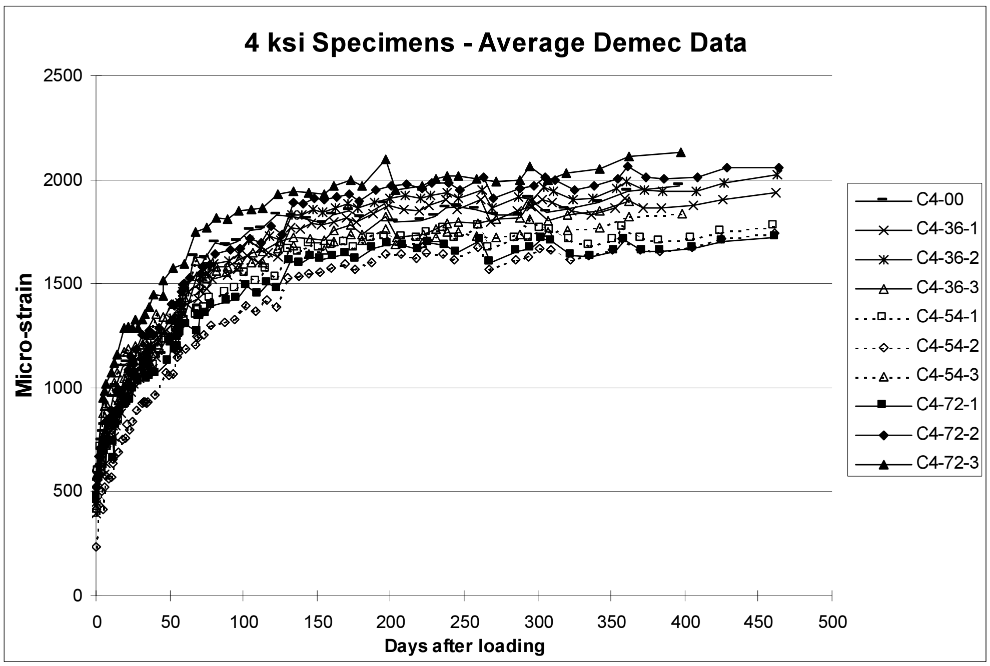


Figure 4.43 Comparison of 4 ksi Specimens

To clearly illustrate the differences between responses of specimens having the same longitudinal reinforcement ratio, responses of specimens with the same reinforcement ratio were plotted in the same figure. Figure 4.44 shows the average response of specimens with a reinforcement ratio of 0.0036. Specimen C4-36-3 was from Group 4. The slightly lower concrete strength of Group 4 specimens was not reflected by larger strains for Specimen C4-36-3. All three specimens experienced average strains below 2,070 micro-strain, although Specimen C4-36-2 very nearly reached this level of response. The minimum maximum strain attained by any of the specimens was approximately 1840 micro-strain. The least amount measured was 1740 micro-strain.

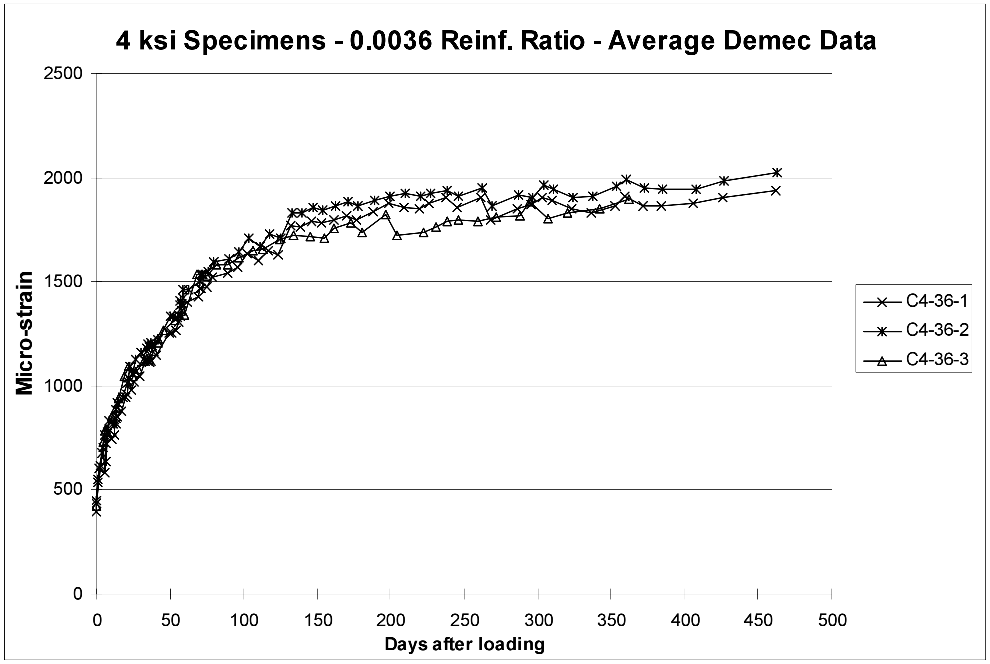


Figure 4.44 Comparison of 4 ksi Specimens–0.0036 Reinforcement Ratio

Figure 4.45 shows the response of specimens with a reinforcement ratio of 0.0054. Again, any significant differences related to concrete strength between the response of the specimen from Group 4 and the specimens from Group 3 were not evident. All three specimens experienced maximum average strains below 2,070 micro-strain.

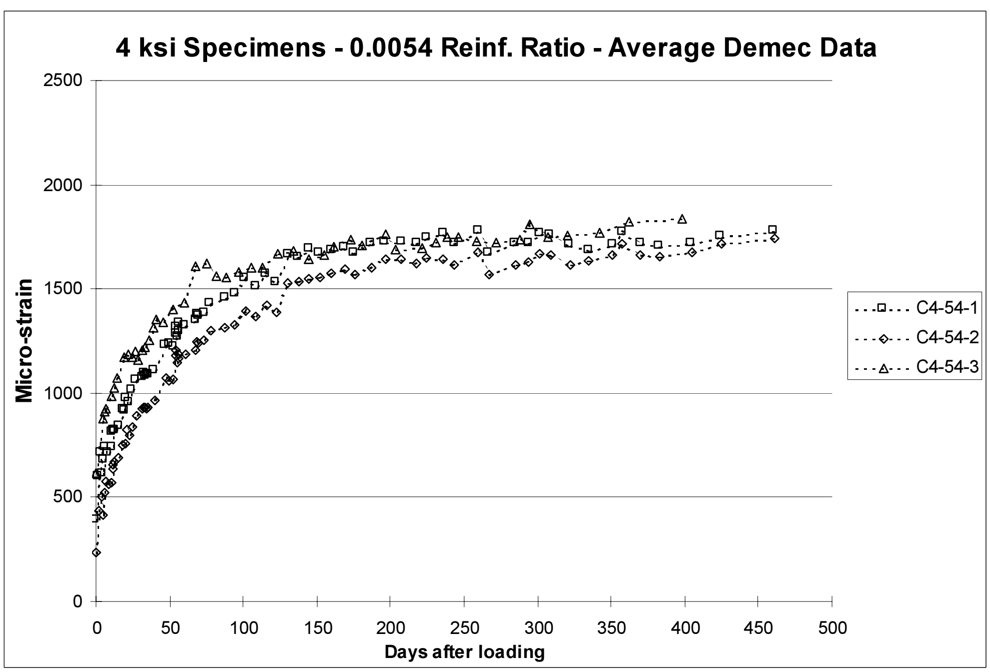


Figure 4.45 Comparison of 4 ksi Specimens–0.0054 Reinforcement Ratio

Figure 4.46 shows the response of specimens with a reinforcement ratio of 0.0072. Specimen C4-72-3 was the Group 4 specimen with lower-strength concrete and had a significantly larger initial and maximum strain than Specimen C4-72-1, as expected. Surprisingly, the Group 3 Specimen C4-72-2 experienced strains similar to the lower-strength Specimen C4-72-3. Only the response of Specimen C4-72-3 exceeded 2,070 micro-strain, although C4-72-2 was very near that level. The maximum strain attained by Specimen C4-72-1 was approximately 1720 micro-strain.

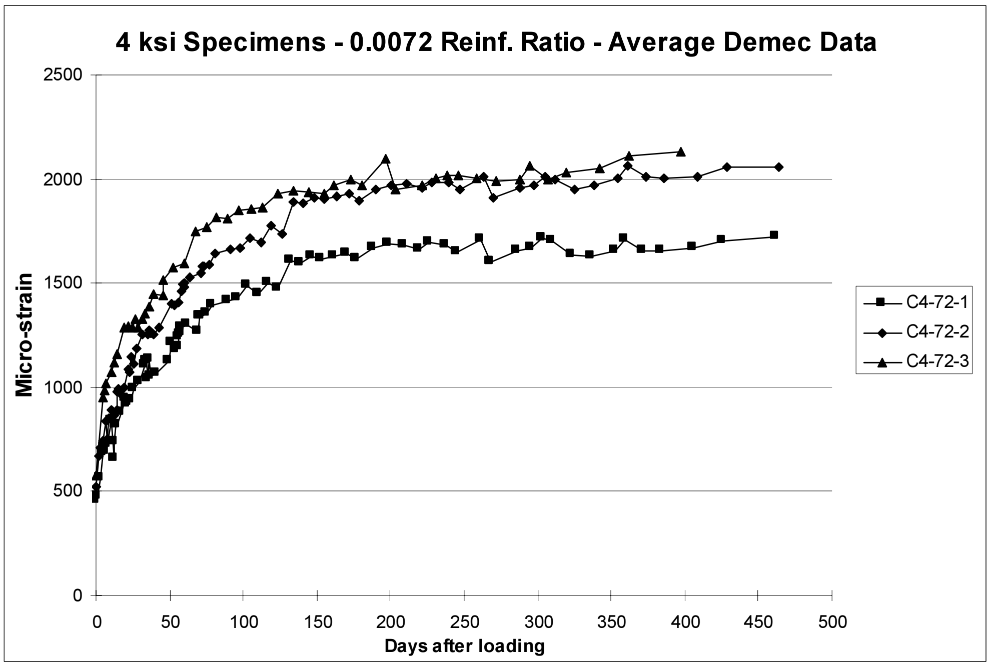


Figure 4.46 Comparison of 4 ksi Specimens–0.0072 Reinforcement Ratio

4.3.1.3 General Discussion

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

There was some concern that specimens had been mislabeled, resulting in some of the inconsistent strain responses (e.g., Specimen C4-72-2 [Fig. 4.46] having such high strains). This possibility was investigated and refuted. The number of electrical gages in each specimen was a function of the number of longitudinal No. 2 bars. The concern about mislabeling was dismissed by counting the number of wire leads exiting the questioned specimens.

4.4 CONCRETE CYLINDERS

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

Table 4.1

		Small Cylinders, (4 inch x 8 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	9,640	10,400	10,000	8,420	9,180	9,420
2	8,000	6,900	7,530	6,790*	6,520	6,920	7,030*
3	4,000	5,060	5,500	5,490	4,900	5,390	5,660
4	4,000	4,120	4,440	4,640*	4,220	4,460	4,690*

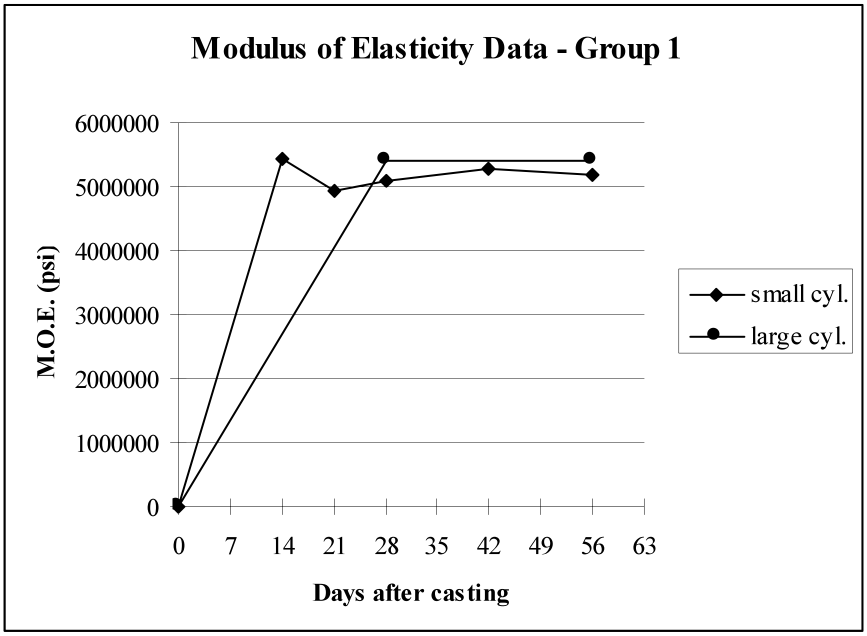
* Test performed on day 49.

Table 4.2

		Small Cylinders, (4 inch x 8 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	5,440	5,090	5,200	–	5,400	5,420
2	5,098	4,960	4,660	4,480*	4,890	5,100	4,490*
3	3,605	4,040	4,250	3,050	4,240	4,260	4,160
4	3,605	3,590	3,530	3,880*	3,920	3,680	4,010*

* Test performed on day 49.

Figures 4.47 through 4.50 present this information graphically.



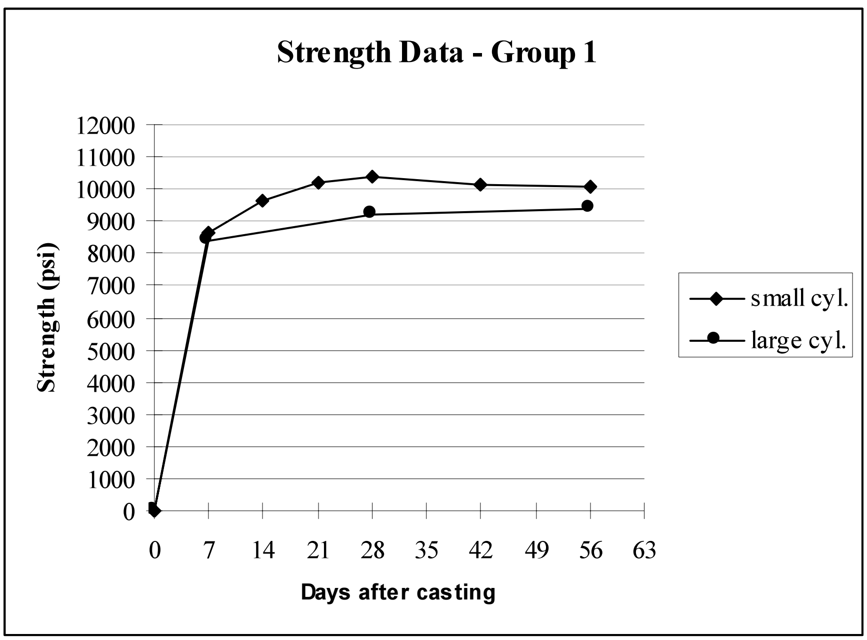
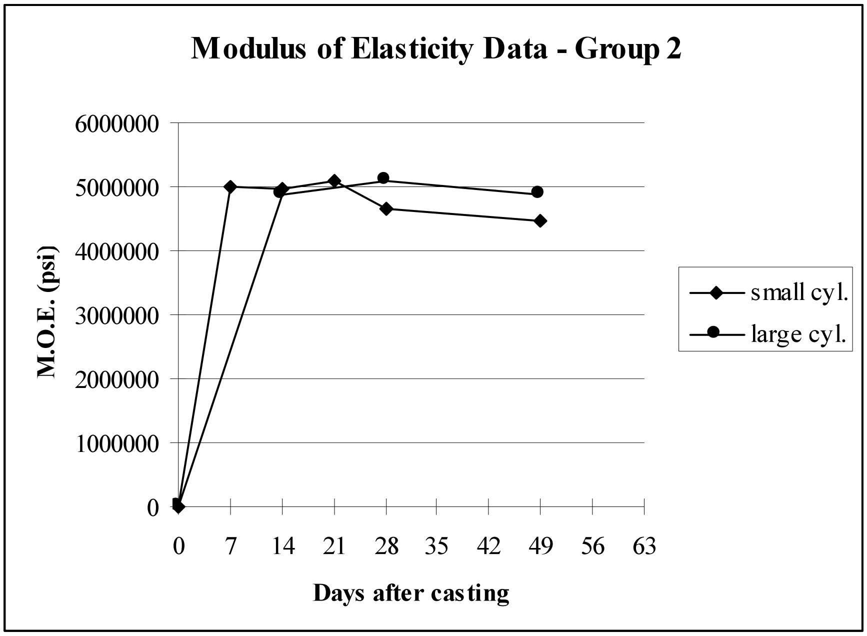


Figure 4.47 Strength and Modulus Data for Group 1



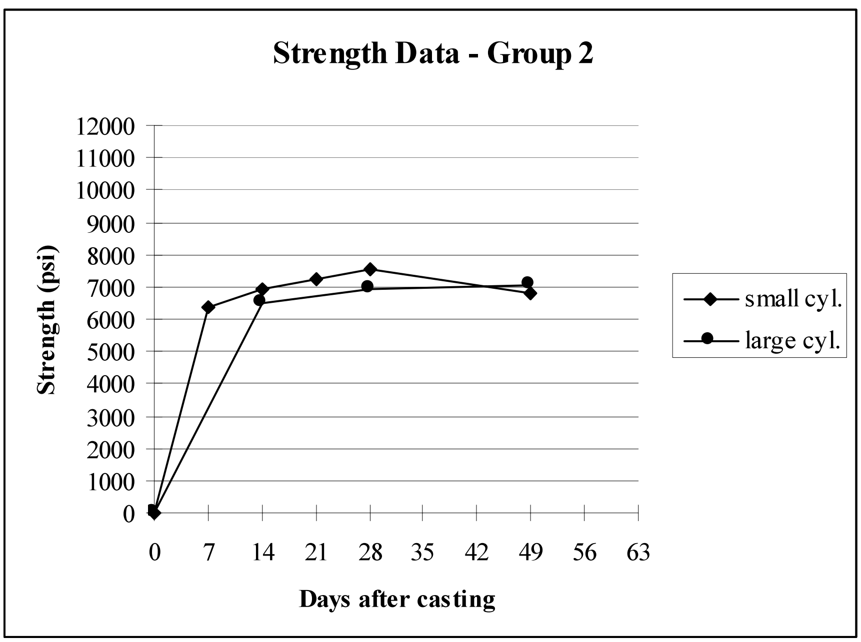
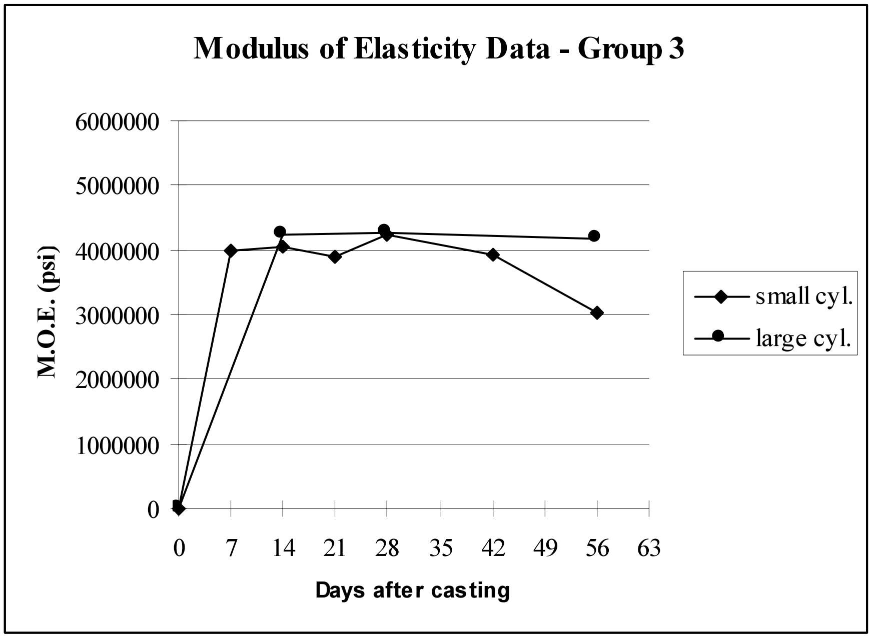


Figure 4.48 Strength and Modulus Data for Group 2



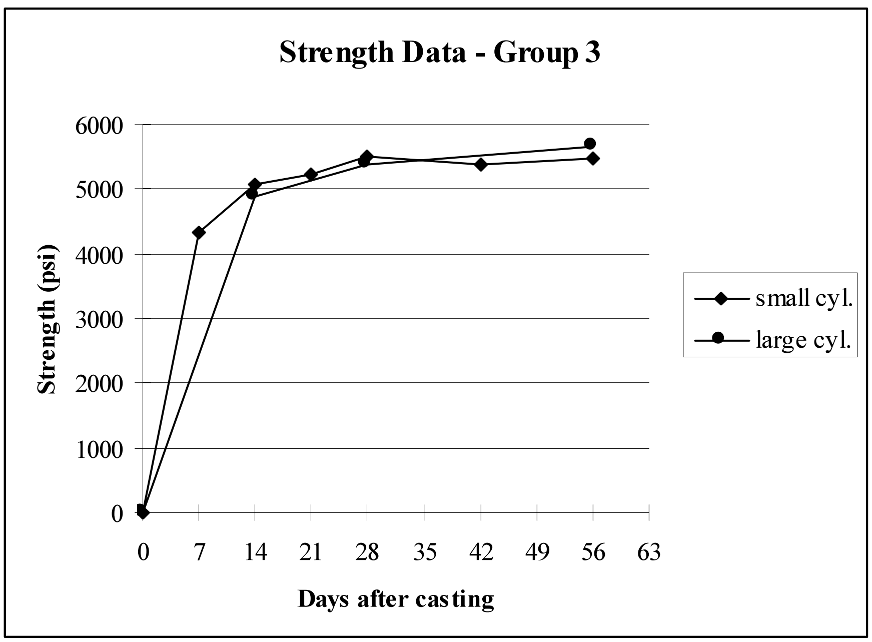
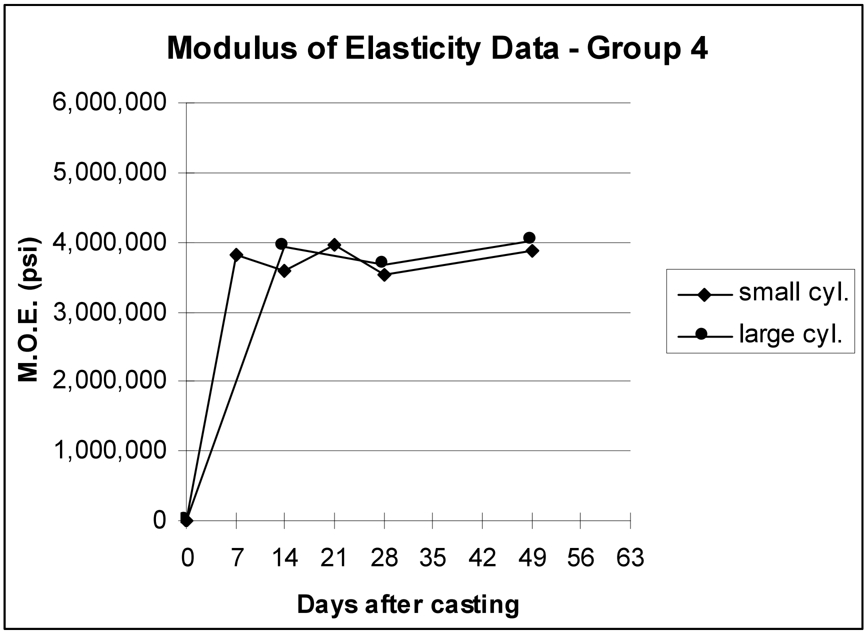


Figure 4.49 Strength and Modulus Data for Group 3



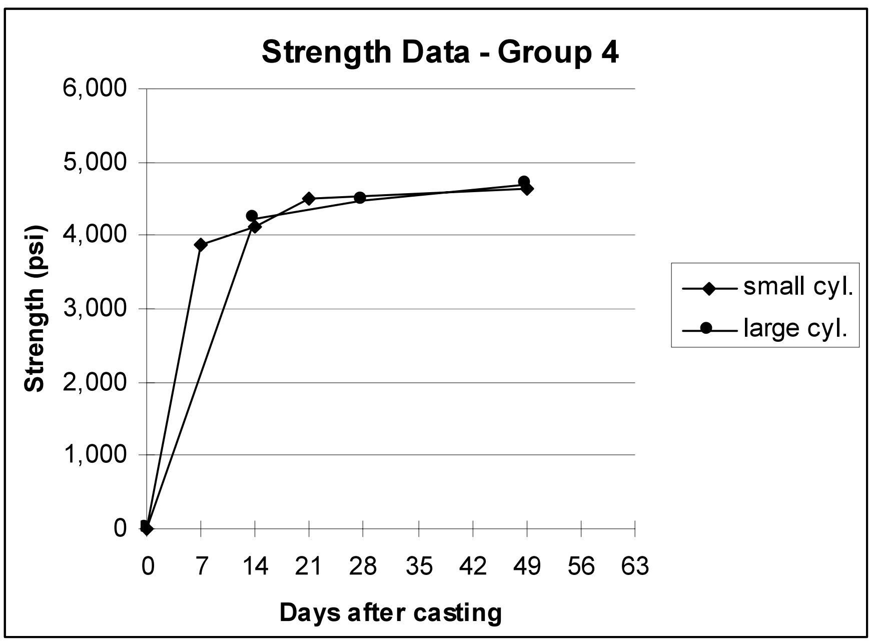


Figure 4.50 Strength and Modulus Data for Group 4

4.5 Reinforcing Steel

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

CHAPTER 5

COMPARISON OF EXPERIMENTAL RESULTS WITH ACI 209R-86

5.1 INTRODUCTION

This chapter 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 report 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:

$$\nu_u = 2.35\gamma_c$$

and

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

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 days to 1 year shrinkage minus the 7 days to 28 days shrinkage.

Initial Moist Curing:

For shrinkage of moist-cured concrete 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:

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

$$\text{Shrinkage } \gamma_\lambda = 1.40 - 0.010\lambda, \quad \text{for } 40 \le \lambda \le 80 \qquad \text{(ACI 2-15)}$$

$$= 3.00 - 0.030\lambda, \quad \text{for } 80 > \lambda \le 100 \qquad \text{(ACI 2-16)}$$

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 investigation, a γ_λ value of 1.0 was used for all cases.

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

During the first year after loading:

$$\text{Creep } \gamma_h = 1.14 - 0.023\,h, \qquad \text{(ACI 2-17)}$$

For ultimate values:

$$\text{Creep } \gamma_h = 1.10 - 0.017\,h, \qquad \text{(ACI 2-18)}$$

During the first year of drying:

$$\text{Shrinkage } \gamma_h = 1.23 - 0.038\,h, \qquad \text{(ACI 2-19)}$$

For ultimate values:

$$\text{Shrinkage } \gamma_h = 1.17 - 0.029\,h, \qquad \text{(ACI 2-20)}$$

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

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

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

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

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.

<u>Temperature other than 70 degrees Fahrenheit:</u>

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 investigation, 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 inside the concrete specimens.

Slump:

$$\text{Creep } \gamma_s = 0.82 + 0.067s \qquad \text{(ACI 2-23)}$$

$$\text{Shrinkage } \gamma_s = 0.89 + 0.041s \qquad \text{(ACI 2-24)}$$

where s is the observed slump in inches.

Fine Aggregate Percentage:

$$\text{Creep } \gamma_\psi = 0.880 + 0024\psi \qquad \text{(ACI 2-25)}$$

For $\psi \leq 50$ percent

$$\text{Shrinkage } \gamma_\psi = 0.30 + 0.014\psi \qquad \text{(ACI 2-26)}$$

For $\psi > 50$ percent

$$\text{Shrinkage } \gamma_\psi = 0.90 + 0.002\psi \qquad \text{(ACI 2-27)}$$

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

Cement Content:

$$\text{Shrinkage } \gamma_c = 0.75 + 0.00036c \qquad \text{(ACI 2-28)}$$

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

Air Content:

$$\text{Creep } \gamma_a = 0.46 + 0.09\alpha, \qquad \text{(ACI 2-29)}$$

but not less than 1.0

$$\text{Shrinkage } \gamma_a = 0.95 + 0.008\alpha \qquad \text{(ACI 2-30)}$$

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.

Table 5.2 Summary of ACI 209-86 Correction Factors and Ultimate Coefficients and Strains for 8 ksi Specimens

	C8-00	C8-36			C8-54			C8-72			U8–(Unloaded Specimens)			
		1	2	3	1	2	3	1	2	3	00-2	36	54	72
Loading Age (days)	**21**	**20**	**17**	**22**	**19**	**19**	**22**	**19**	**19**	**20**	**-**	**-**	**-**	**-**
creep, γ_{la}	0.87	0.88	0.90	0.87	0.88	0.88	0.87	0.88	0.88	0.88	-	-	-	-
shrinkage	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Initial Cure (days)	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**
creep	-	-	-	-	-	-	-	-	-	-	-	-	-	-
shrinkage, γ_{cp}	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
Humidity (percent)	**38**	**37**	**37**	**38**	**37**	**37**	**38**	**37**	**37**	**38**	**38**	**37**	**37**	**37**
creep, γ_λ	1	1	1	1	1	1	1	1	1	1	1	1	1	1
shrinkage, γ_λ	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Avg. Thickness (inches)	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**
creep, γ_h	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
shrinkage, γ_h	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Vol.-Surface Ratio (inches)	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**
creep, γ_{vs} (not used)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
shrinkage, γ_{vs} (not used)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Temperature (deg. F)	**75**	**76**	**76**	**75**	**76**	**76**	**75**	**76**	**76**	**75**	**75**	**76**	**76**	**76**
creep	1.08	1.12	1.12	1.08	1.12	1.12	1.08	1.12	1.12	1.08	-	-	-	-
shrinkage	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Slump (inches)	**6.5**	**7.5**	**7.5**	**6.5**	**7.5**	**7.5**	**6.5**	**7.5**	**7.5**	**6.5**	**6.5**	**7.5**	**7.5**	**7.5**
creep, γ_s	1.26	1.32	1.32	1.26	1.32	1.32	1.26	1.32	1.32	1.26	1.26	1.32	1.32	1.32
shrinkage, γ_s	1.16	1.20	1.20	1.16	1.20	1.20	1.16	1.20	1.20	1.16	1.16	1.20	1.20	1.20
Fine Aggregate (percent)	**34**	**34**	**34**	**34**	**34**	**34**	**34**	**34**	**34**	**34**	**34**	**34**	**34**	**34**
creep, γ_ψ	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
shrinkage, γ_ψ	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78	0.78
Concrete Content (pcy)	**700**	**700**	**700**	**700**	**700**	**700**	**700**	**700**	**700**	**700**	**700**	**700**	**700**	**700**
creep	-	-	-	-	-	-	-	-	-	-	-	-	-	-
shrinkage, γ_c	1.18	1.18	1.18	1.18	1.18	1.18	1.18	1.18	1.18	1.18	1.18	1.18	1.18	1.18
Air Content (percent)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
creep	-	-	-	-	-	-	-	-	-	-	-	-	-	-
shrinkage, γ_a	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Sum of Creep Factors	**1.08**	**1.20**	**1.22**	**1.08**	**1.20**	**1.20**	**1.08**	**1.20**	**1.20**	**1.09**	**-**	**-**	**-**	**-**
Sum of Shrinkage Factors	**1.03**	**1.06**	**1.06**	**1.03**	**1.06**	**1.06**	**1.03**	**1.06**	**1.06**	**1.03**	**1.03**	**1.06**	**1.06**	**1.06**
Ultimate Creep Coeff., ν_u	**2.55**	**2.81**	**2.87**	**2.53**	**2.83**	**2.83**	**2.53**	**2.83**	**2.83**	**2.56**	**-**	**-**	**-**	**-**
Ultimate Shrinkage, $(\varepsilon_{sh})_u$	**801**	**829**	**829**	**801**	**829**	**829**	**801**	**829**	**829**	**801**	**801**	**829**	**829**	**829**

Table 5.3 Summary of ACI 209-86 Correction Factors and Ultimate Coefficients and Strains for 4 ksi Specimens

	C4-00	C4-36			C4-54			C4-72			U4–(Unloaded Specimens)			
		1	2	3	1	2	3	1	2	3	00-2	36	54	72
Loading Age (days)	**16**	**23**	**22**	**15**	**25**	**24**	**15**	**23**	**21**	**15**	-	-	-	-
creep, γ_{la}	0.90	0.86	0.87	0.91	0.86	0.86	0.91	0.86	0.87	0.91	-	-	-	-
shrinkage	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Initial Cure (days)	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**	**5**
creep	-	-	-	-	-	-	-	-	-	-	-	-	-	-
shrinkage, γ_{cp}	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
Humidity (percent)	**37**	**39**	**39**	**37**	**39**	**39**	**37**	**39**	**39**	**37**	**37**	**39**	**39**	**39**
creep, γ_{λ}	1	1	1	1	1	1	1	1	1	1	1	1	1	1
shrinkage, γ_{λ}	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Avg. Thickness (inches)	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**	**8**
creep, γ_h	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
shrinkage, γ_h	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Vol.-Surface Ratio (inches)	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**	**2**
creep, γ_{vs} (not used)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
shrinkage, γ_{vs} (not used)	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Temperature (deg. F)	**77**	**76**	**76**	**77**	**76**	**76**	**77**	**76**	**76**	**77**	**77**	**76**	**76**	**76**
creep	1.15	1.13	1.13	1.15	1.13	1.13	1.15	1.13	1.13	1.15	-	-	-	-
shrinkage	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Slump (inches)	**6.5**	**6.0**	**6.0**	**6.5**	**6.0**	**6.0**	**6.5**	**6.0**	**6.0**	**6.5**	**6.5**	**6.0**	**6.0**	**6.0**
creep, γ_s	1.26	1.22	1.22	1.26	1.22	1.22	1.26	1.22	1.22	1.26	1.26	1.22	1.22	1.22
shrinkage, γ_s	1.16	1.14	1.14	1.16	1.14	1.14	1.16	1.14	1.14	1.16	1.16	1.14	1.14	1.14
Fine Aggregate (percent)	**56**	**56**	**56**	**56**	**56**	**56**	**56**	**56**	**56**	**56**	**56**	**56**	**56**	**56**
creep, γ_{ψ}	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
shrinkage, γ_{ψ}	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
Concrete Content (pcy)	**500**	**500**	**500**	**500**	**500**	**500**	**500**	**500**	**500**	**500**	**500**	**500**	**500**	**500**
creep	-	-	-	-	-	-	-	-	-	-	-	-	-	-
shrinkage, γ_c	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06
Air Content (percent)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**
creep	-	-	-	-	-	-	-	-	-	-	-	-	-	-
shrinkage, γ_a	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Sum of Creep Factors	**1.26**	**1.16**	**1.16**	**1.27**	**1.15**	**1.15**	**1.27**	**1.16**	**1.17**	**1.27**	-	-	-	-
Sum of Shrinkage Factors	**1.20**	**1.18**	**1.18**	**1.20**	**1.18**	**1.18**	**1.20**	**1.18**	**1.18**	**1.20**	**1.20**	**1.18**	**1.18**	**1.18**
Ultimate Creep Coeff., ν_u	**2.96**	**2.72**	**2.73**	**2.98**	**2.69**	**2.71**	**2.98**	**2.72**	**2.75**	**2.98**	-	-	-	-
Ultimate Shrinkage, $(\varepsilon_{sh})_u$	**936**	**920**	**920**	**936**	**920**	**920**	**936**	**920**	**920**	**936**	**936**	**920**	**920**	**920**

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. 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:

$$\varepsilon_{initial} = \frac{P}{[A_g(1-\rho_g) + n\rho_g A_g]\cdot E_{ci}}$$ ***Equation 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

$\rho_g = A_{st}/A_g$

The effective modulus was used in conjunction with the transformed section to predict 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:

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

where

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

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

$$\varepsilon_{initial} + \left(\varepsilon_{creep}\right)_t = \frac{P}{[A_g(1-\rho_g) + A_g\rho_g n_{eff.}]\cdot E_{eff.}} \qquad \textbf{\textit{Equation 5.2}}$$

where all values are as before except:

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

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

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:

$$P_{resisting} = \left(\varepsilon_{sh}\right)_t \; E_{st} \; A_g \; \rho_g \qquad \textbf{\textit{Equation 5.3}}$$

where all terms are as before except:

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

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

$$(\varepsilon_{shrinkage})_t = (\varepsilon_{sh})_t - \left\{\frac{(\varepsilon_{sh})_t E_{st} A_g \rho_g}{[A_g(1-\rho_g) + A_g\rho_g n_{eff.}]\cdot E_{eff.}}\right\} \qquad \textbf{\textit{Equation 5.4}}$$

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

$$\left(\varepsilon_{total}\right)_t = \left\lfloor \varepsilon_{initial} + \left(\varepsilon_{creep}\right)_t \right\rfloor + \left(\varepsilon_{shrinkage}\right)_t \qquad \textbf{\textit{Equation 5.5}}$$

These equations where used to predict strain histories for the specimens by varying time after loading and calculating total strain at that time. Predicted responses are presented with measured strain data presented previously in Chapter 4 of this report. Results of the comparisons are presented 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

Curves plotted with solid lines represent the measured Demec strains modified for temperature with the loading day plotted as day zero. Dashed curves represent the responses predicted using ACI 209-86 considering the measured conditions (e.g., temperature and slump).

5.4 General Discussion

5.4.1 8,000 psi Specimens

Predicted results and experimental measurements for the specimens in Group 1 (Figure 5.1) appear to correspond reasonably well. Computed strains fall within the scatter of the measured results.

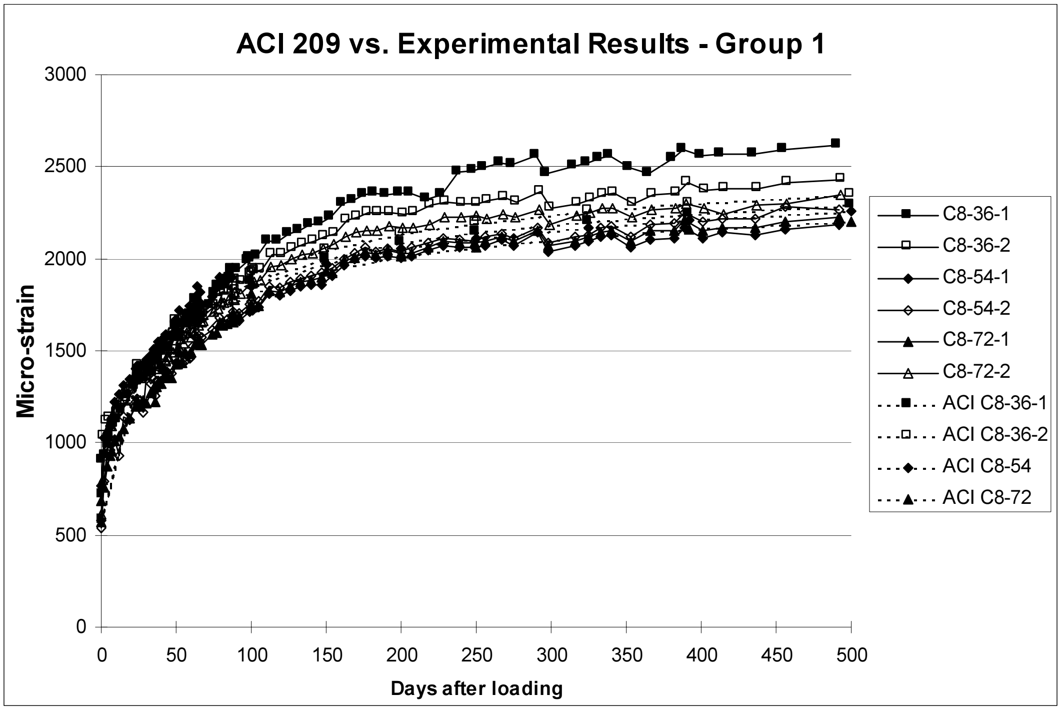


Figure 5.1 Comparison of Strains Predicted by ACI 209 and Strains Measured in Columns of Group 1

For the specimens of Group 2 (Figure 5.2) the experimental results indicate strains that are generally higher than those predicted by the ACI 209 method. Measured strains tended to increase at a higher rate for early ages than the predicted strains. This may be due to the specimens in Group 2 being in an environment with high temperature during the first few months of loading. Only a single average temperature was used in the ACI 209 procedure. However, average temperature during the first two months of loading was 91 degrees Fahrenheit. The average temperature used in the ACI 209 procedure was 75 degrees Fahrenheit.

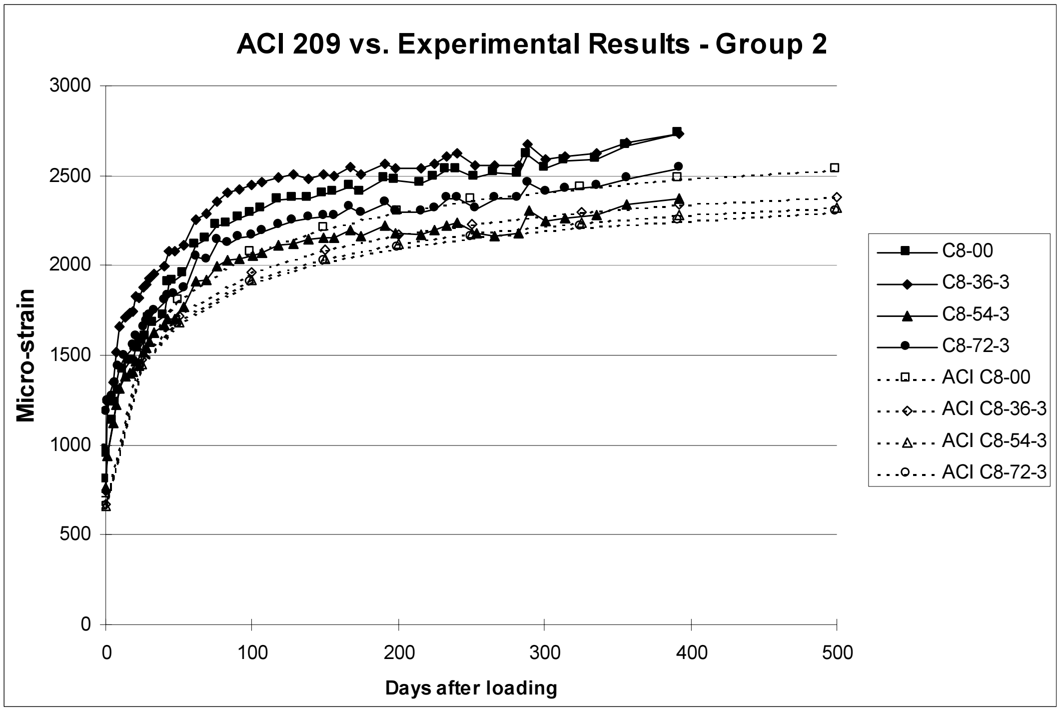


Figure 5.2 Comparison of Strains Predicted by ACI 209 and Strains Measured in Columns of Group 2

5.4.2 4,000 PSI SPECIMENS

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

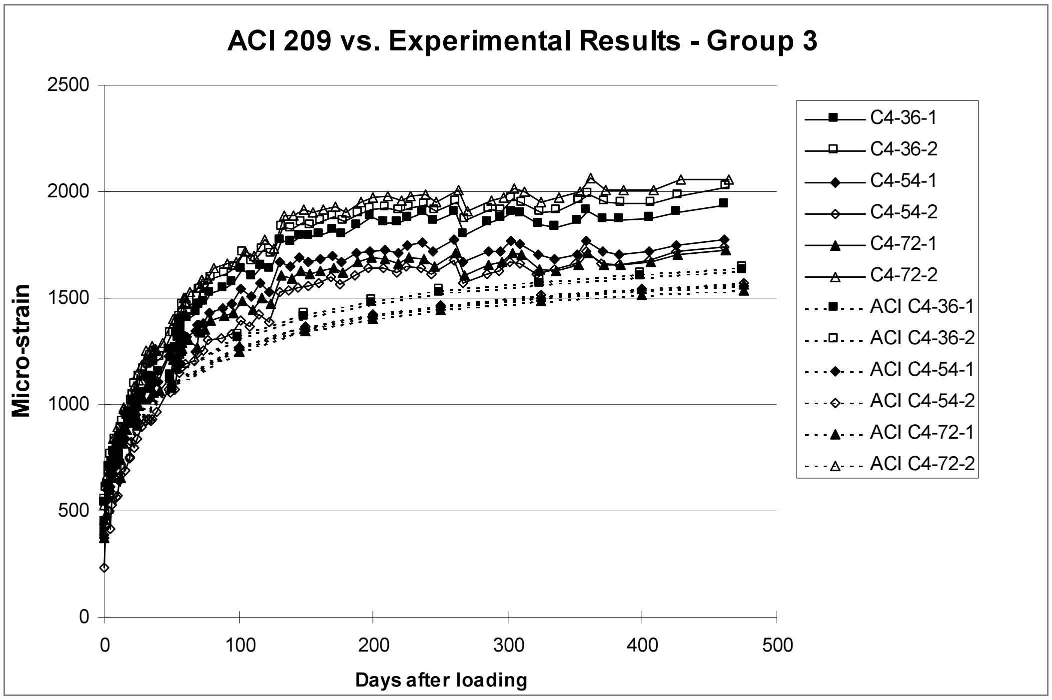


Figure 5.3 Comparison of Strains Predicted by ACI 209 and Strains Measured in Columns of Group 3

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

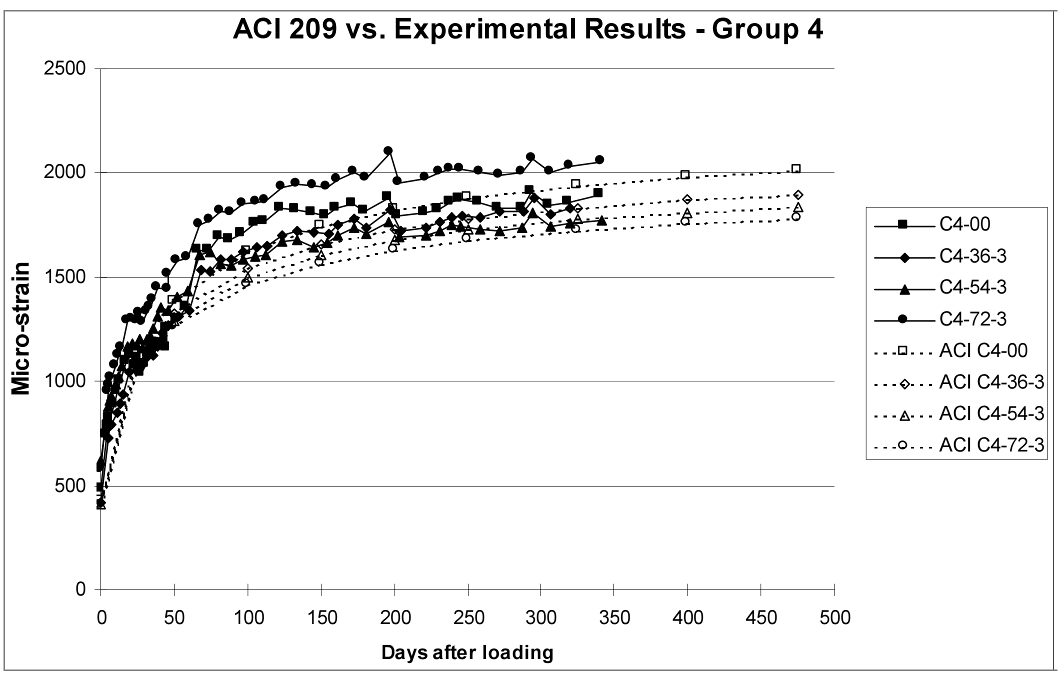


Figure 5.4 Comparison of Strains Predicted by ACI 209 and Strains Measured in Columns of Group 4

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Introduction

This investigation was carried out to examine the long-term behavior of axially loaded reinforced concrete columns and to determine if current longitudinal reinforcement requirements for columns can be reduced. Specifically, it has been hypothesized that the current specification/code minimum longitudinal reinforcement ratio for reinforced concrete columns could be reduced from the current one percent requirement. This experimental program and related analyses were designed to determine the validity of such a reduction. All columns in this study had reinforcement percentages less than one percent and were loaded for longer than 12 months. Plots of measured strain versus time (Figures 4.39 through 4.46) indicate that the rate of strain increase after twelve months of loading was very small. Results of the experimental investigation were compared to strain predictions based on an analytical method reported by ACI-209.

6.2 Experimental Investigation

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

1. Concrete Strength
   Nominal strengths of 4,000 psi and 8,000 psi.
2. Reinforcement Ratio
   Reinforcement percentages of 0.36 percent, 0.54 percent, and 0.72 percent.
3. Eccentricity
   No eccentricity and eccentricity of the axial load equal to 0.10 times the column diameter.

6.3 Comparison of Experimental Results with Predicted Analytical Results

Experimental results were presented in Chapter 4 and compared with predicted results in Chapter 5. An analytical method recommended by ACI 209 was used to predict the strain responses of the specimens. Age at loading, ambient humidity, ambient temperature, concrete strength, and several other parameters were considered in the analysis. The predicted values were either equal to or slightly less than the measured values.

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

6.4 Conclusions

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

1. Temperature and humidity affect creep and shrinkage significantly.
2. Strain response predictions made using the ACI 209 method agreed reasonably well with measured data but tended to under-predict strains when higher temperatures were encountered early in the loading period.
3. It was necessary to correct measured strains for temperature effects to produce reasonable results.
4. As the ratio of dead-to-live load increases, the amount of steel required to prevent passive yielding increases.
5. As concrete compressive strength increases, the amount of steel needed to prevent passive yielding increases.
6. If the conditions which cause creep in concrete are at the standard values as defined by ACI Committee 209, then for many material strengths and live load-to-dead load ratios, the minimum percentage of longitudinal reinforcement can be reduced below one percent. This is demonstrated in Table 1.1 in Chapter 1.
7. If the conditions which cause creep in concrete are at the upper-bound values as reported by ACI Committee 209, then for many material strengths and live load-to-dead load ratios, the minimum percentage of longitudinal reinforcement cannot be reduced below one percent. This is demonstrated in Table 1.2 in Chapter 1.
8. From Conclusions 6 and 7, it appears that it may be acceptable to reduce the minimum reinforcement requirement for certain conditions, but in general, it cannot be reduced. To permit the minimum amount of steel to be reduced, a design equation or table that accounts for material strengths, live load-to-dead load ratio, and creep and shrinkage factors could be developed. Unfortunately, the factors that affect creep and shrinkage are not typically within the designer's control (e.g., loading age, temperature, humidity, air content, and cement content). If the worst case is assumed for these factors, the minimum amount of steel that is needed in nearly all cases to preclude passive yielding of longitudinal reinforcement is more than one percent of the gross cross section.

9. The compression steel in all eccentrically-loaded columns reached strains well beyond yield strain. None of the columns failed, but significant visible curvature was present in all four. Test specimens had significant concrete cover ($\frac{3}{8}$ inch or 1.5 d_b) and transverse reinforcement (ρ_s = .025). Had the specimens been fabricated with less cover and reduced transverse reinforcement with larger spacing, it is possible that spalling of cover and instability of longitudinal reinforcement might have occurred for the curvatures experienced in theses specimens.

6.5 FURTHER RESEARCH

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

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

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