

**BEHAVIOR OF HIGH-STRENGTH CONCRETE MEMBERS SUBJECTED
TO COMBINED FLEXURE AND AXIAL COMPRESSION LOADINGS**

by
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DEDICATION

To my father
AYTAÇ MERTOL

For his unlimited support and encouragement over the years,
which made me understand him better day by day.

BIOGRAPHY

Halit Cenan Mertol completed his Bachelor of Science degree in Department of Civil Engineering in 1999 in Middle East Technical University, Ankara, Turkey. He also received his Master of Science degree from the same university in 2002, on the topic of Carbon Fiber Reinforced Masonry Infilled Reinforced Concrete Frame Behavior under the supervision of advisor Dr. Tuğrul Tankut. After completion of his degree, he moved to the United States to pursue his Doctor of Philosophy degree in North Carolina State University to work with Dr. Sami Rizkalla on the characteristics of high-strength concrete members subjected to combined flexure and axial compression.

ABSTRACT

MERTOL, HALIT CENAN. Behavior of High-Strength Concrete Members Subjected to Combined Flexure and Axial Compression Loadings. (Under the direction of Dr. Sami Rizkalla.)

The use of high-strength concrete (HSC) in structures and bridges has become a common practice worldwide. In bridges, HSC could lead to a reduction in number and depth of the girders as well as an increase in the span length. These features reduce the complexity of a project with reduced number of piers, construction time and cost. Furthermore, the enhanced durability of HSC could result in reduction of the maintenance costs and increase the service life of the structure. In buildings, the sizes of the members could be significantly reduced which could help in the design and construction of higher structures with larger spans. However, due to lack of research data, most of the design codes worldwide limit the applicability of HSC.

A total of 21 plain concrete specimens were tested under combined flexure and axial compression to evaluate the stress-strain distribution of HSC in the compression zone of flexural members. The variables considered in this investigation were mainly the strength of concrete and the age of the specimen. The measured stress-strain curves and stress block parameters, including the influence of the concrete strength, were compiled with the data in the literature to evaluate the fundamental characteristics of high-strength concrete in the compression zone of flexural members. A total of 42 cylindrical specimens and 18 prism specimens were used to evaluate the creep and shrinkage properties of HSC. The variables considered in this investigation were the concrete compressive strength, specimen size, curing type, age of concrete at loading and loading stress level. The creep coefficients and shrinkage strains were obtained for the range of concrete compressive strength, evaluated and compiled with the current predictions according to the design codes.

Using the test results of this research and other researches in literature, revisions to the LRFD Bridge Design Specifications (2004) are recommended to extend the applicability of its compressive and combined compressive and flexural design provisions to concrete compressive strengths up to 18 ksi.

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1 INTRODUCTION

1.1 General

The use of high-strength concrete (HSC), ranging from 10 to 18 ksi, has become a common practice worldwide. In bridges, HSC could lead to a reduction in number and depth of the girders as well as an increase in the span length. These features reduce the complexity of a project with reduced number of piers, construction time and cost. Furthermore, the enhanced durability of HSC could result in reduction of the maintenance costs and increase the service life of the structure. In buildings, the sizes of the members could be significantly reduced which could help in the design and construction of higher structures with larger spans.

Development of HSC dates back to 1930's, but these early developments were economically prohibitive for practical applications. In the 1960's, superplasticizers were developed in Japan and Germany and made it possible to decrease the water to cement ratio of concrete while maintaining its workability. In the 1970's, the combined use of superplasticizers and ultra-fine materials such as silica fume, finely ground granulated blast furnace slag or anhydrous gypsum led to further improvement of concrete performance measures including its strength. Since the mid-1980's, HSC has gained popularity in both precast and cast-in-place construction for either reinforced or prestressed members. In Japan, concrete compressive strengths as high as 11.4 ksi were used in the 1970's for railway bridges (ACI 363R-92 1997).

In the early 1990's, the United States Department of Transportation Federal Highway Administration (USDOT-FHWA) began sponsoring the use of HSC in several demonstration projects. Since 1993, a number of HSC bridges have been constructed across the United States. The USDOT-FHWA compilation project (Russell et al. 2003a, 2003b) reports on 19 such bridges in 14 states. While the highest design concrete compressive strength in these bridges was reported as 14 ksi in Texas, the achieved strength at the design age reached as high as 15.9 ksi in South Dakota.

However, due to lack of sufficient research data, most of the design codes worldwide limit the applicability of HSC. The AASHTO LRFD Bridge Design Specifications (2004), first published in 1994, includes an article (5.4.2.1) limiting its applicability to a maximum concrete compressive strength of 10 ksi, unless physical tests are made to establish the relationship between concrete compressive strength and its other properties. Many design provisions stipulated in the AASHTO LRFD Bridge Design Specifications (2004) are still based on test results obtained from specimens with compressive strengths less than 10 ksi. Although such a strength limit is not explicitly imposed by other codes such as the ACI 318-05 Building Code Requirements for Structural Concrete (2005), except in the provisions for development length, their applicability to HSC is not fully addressed either.

The United States National Cooperative Highway Research Program (NCHRP) of the National Academies has initiated four separate projects to extend the AASHTO LRFD Bridge Design Specifications (2004), allow broader use of HSC, and meet the needs of the bridge design community. NCHRP Project 18-07 addressed prestress losses in pretensioned concrete girders (Tadros et al. 2003). NCHRP Project 12-56 is addressing shear in reinforced and prestressed concrete members. NCHRP Project 12-60 is addressing bond and development length in reinforced and prestressed concrete. The objective of NCHRP Project 12-64, which is the focus of this thesis, is to recommend revisions to the LRFD Bridge Design Specifications to extend the applicability of its compressive and combined compressive and flexural design provisions for reinforced and prestressed concrete members to concrete compressive strengths up to 18 ksi.

1.2 Statement of Problem

The flexural design of reinforced and prestressed concrete structural members in the United States is based on the ultimate strength approach. Before 1930's, the allowable stress (working stress) theory was used in the design codes. In the early 1930's the extensive investigation performed by Richart et al. (1931a, 1931b, 1931c and 1932) on the concentrically loaded columns led the transformation of the design theory for columns into ultimate strength approach. In the mid 1950's, Hognestad (1955) developed rectangular stress block parameters which created the basis of the ultimate strength principle used in the current codes for flexure analysis and design.

The current rectangular stress block specified by ACI 318-05 Building Code Requirements for Structural Concrete (2005) and AASHTO LRFD Bridge Design Specifications (2004) is based on normal-strength concrete (NSC) up to 10 ksi. As the improvement of the concrete compressive strength encourages the designers to use HSC in design, the rectangular stress block parameters must be evaluated for the use of HSC and if necessary new rectangular stress block parameters must be introduced for HSC design.

Concrete is a time dependent material. In particular, concrete creeps under sustain load, and shrinks due to the changes in the moisture content of the surrounding environment. These physical changes increase by time. The information on creep and shrinkage of concrete can be used to determine the prestressing losses, long term deformations and cracking of the civil engineering structures. The evaluation of creep and shrinkage of concrete is very important especially for long-span and high-rise structures. The current code equations for creep and shrinkage predictions are based on NSC; therefore, there is a need to evaluate these characteristics for HSC.

1.3 Objectives and Scope

The research program was conducted at the Constructed Facilities Laboratory (CFL) of North Carolina State University (NCSU) from 2003 and 2006. The objectives of the study are to:

1. Evaluate the data published by other researches related to stress block parameters, ultimate compressive strain, Poisson's Ratio, creep and shrinkage of HSC,
2. Conduct experimental program to determine the stress block parameters, ultimate compressive strain, Poisson's Ratio, creep and shrinkage behavior of HSC,
3. Use results of the experimental program and the data from other researches to recommend revisions to the AASHTO LRFD Bridge Design Specifications (2004) to extend the applicability of flexural and compression provisions for reinforced and prestressed concrete members to concrete compressive strengths up to 18 ksi.

1.4 Thesis Arrangement

The literature review on stress-strain relationship, ultimate compressive strain, Poisson's Ratio, creep and shrinkage for normal and HSC is presented in Chapter 2. It includes a review of eccentric bracket specimen tests which provides the basis to evaluate the stress block parameters for concrete. Proposed stress-strain models for unconfined concrete and proposed rectangular stress block parameters by different researchers are presented. The literature includes comprehensive review of the stress block parameters in different design codes from all over the world. In addition, this chapter covers the experimental research performed on Poisson's Ratio, creep and shrinkage of HSC. Proposed relationships currently used for the prediction of creep and shrinkage of HSC are presented in details for future comparison purposes.

Chapter 3 describes the specimens investigated in this research and the methods used in testing to evaluate the stress-strain relationship, Poisson's Ratio, creep and shrinkage of HSC. The composition of materials, test set-ups and the procedures are also illustrated in details.

The test results obtained from all the specimens are presented in Chapter 4. The measured response including the behavior of the specimens is also illustrated. The test results are compiled, analyzed, evaluated and compared with the test data and available relationships presented in Chapter 2.

The analytical work performed in the light of the test results are presented in Chapter 5. It also includes the proposed relationships for stress block parameters, Poisson's Ratio, creep and shrinkage of HSC. Statistical and parametric analyses are carried out to justify the proposed relationships. Furthermore, the proposed relationships for creep and shrinkage are used to develop new requirements for the minimum longitudinal reinforcement ratio for compression members.

A summary of the testing program and analytical work is presented in Chapter 6. Based on the research, conclusions and recommendations are made.

2 BACKGROUND

2.1 General

Concrete is a composite material consisting of cement, water, fine and coarse aggregate. The cement particles hydrate and form the cement paste, the main binder in concrete. The cement paste hardens in time due to a chemical reaction between cement and water. The cement paste binds the fine and coarse aggregate together to form the hardened concrete. The strength of concrete will increase as long as the unhydrated cement particles continue to hydrate with available water.

HSC consists of the similar ingredients as NSC, however, with different proportions and different types. After numerous researches to achieve a stronger concrete, it was observed that to decrease the water to cement ratio in a concrete increases the concrete compressive strength. This can be performed either by increasing the amount of cement or decreasing the amount of water. To increase the amount of cement excessively in a concrete mixture may generate some thermal problems which are due to the increased temperature of concrete during the hydration of cement. It also increases the cost. Use of water in concrete enhances the workability of the unhardened mixture and ensures the hydration of cement particles. Therefore, the problems associated with the increased amount of cement and reduction of the water have to be optimized to obtain HSC.

High compressive strengths of concrete require not only increased amount of cement, but also addition of mineral admixtures such as silica fume and fly ash to the concrete mixture. The heat generated by the hydration of these admixtures is lower than that generated by hydration of cement. The heat of hydration of the concrete can be lowered to acceptable levels by balancing the proportions with these admixtures. Furthermore, these admixtures have very fine particles which will fill the spaces between the cement grains. A more compact mixture will be obtained and the strength will increase further due to compactness of the mixture.

The introduction of the chemical admixtures such as superplasticizers and retarders into the concrete technology made it possible to achieve a workable mixture for a desired

length of time although the amount of water is reduced. Without these chemical admixtures, HSC can not be obtained.

Mahter and Hime (2002) stated that a water to cement ratio of 1.2 by volume which is equivalent to 0.4 by mass is needed to hydrate all the cement particles in a concrete mixture. When more water is added, the excessive water after hydration of cement will generate some voids in the structure of hardened concrete which will decrease the strength of concrete. In case of water to cement ratios less than 0.4 by mass, some of the cement particles will always remain unhydrated which might seem to be a problem. On the contrary, having unhydrated cement particles is not a negative factor as the strength of unhydrated cement particles is much higher than that of hydrated cement particles. Furthermore these unhydrated particles will work as fillers so that a more compact mixture will be achieved (Berntsson et al. 1990).

To improve the concrete compressive strength further, the size of the coarse aggregates is reduced. It was observed that using smaller sized coarse aggregate in a concrete mixture would increase the strength of concrete further (ACI 363R-92 1997).

Since the type, size and properties of ingredients of concrete changes from one location to another location, the properties of concrete also changes. But the process of the failure of concrete is always the same. There are three sources of failures in concrete: (1) the hydrated cement paste, (2) the aggregate and (3) the interface between the hydrated cement paste and the aggregate. Strengthening these failure sources is the only way to strengthen the concrete. For NSC, the failure occurs when the hydrated cement paste and the interface binding the hydrated cement paste and aggregate fail. As a result, the properties of concrete resemble more to the properties of the hydrated cement paste. But for HSC, the hydrated cement paste and the interface between the hydrated cement paste and the aggregate are generally stronger than the aggregate. As a result, the failure plane cuts through the aggregates. This means that the strength of aggregate is the main source that defines the characteristics of HSC. The strength and behavior in this case are limited to the strength and behavior of aggregate. A stronger aggregate should be used in order to enhance the strength and behavior of concrete. The behaviors of NSC and HSC using the same materials but different proportions are shown in Figure 2-1. The bold line in the

figure represents the failure plane in the concrete. The stress-strain relationships for the cement paste and aggregate as well as NSC and HSC are presented in Figure 2-2.

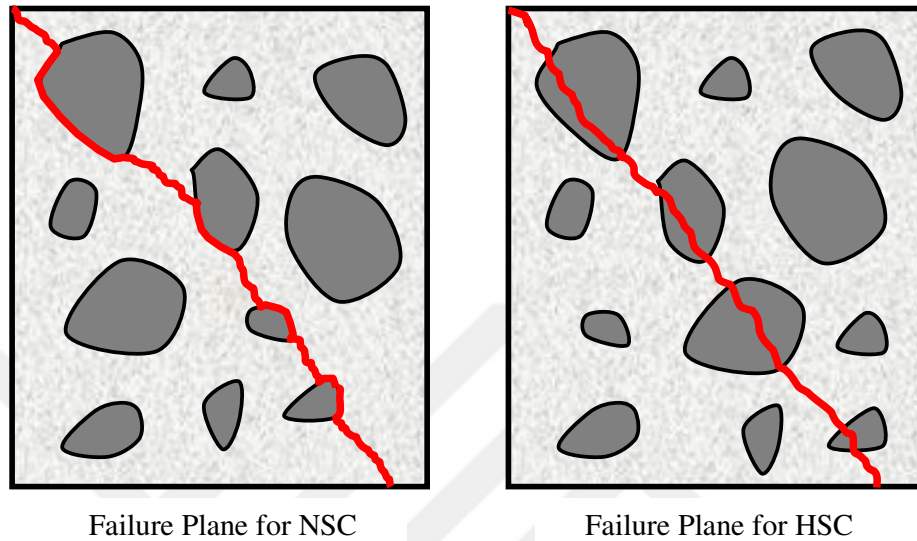


Figure 2-1 – Failure Planes in NSC and HSC

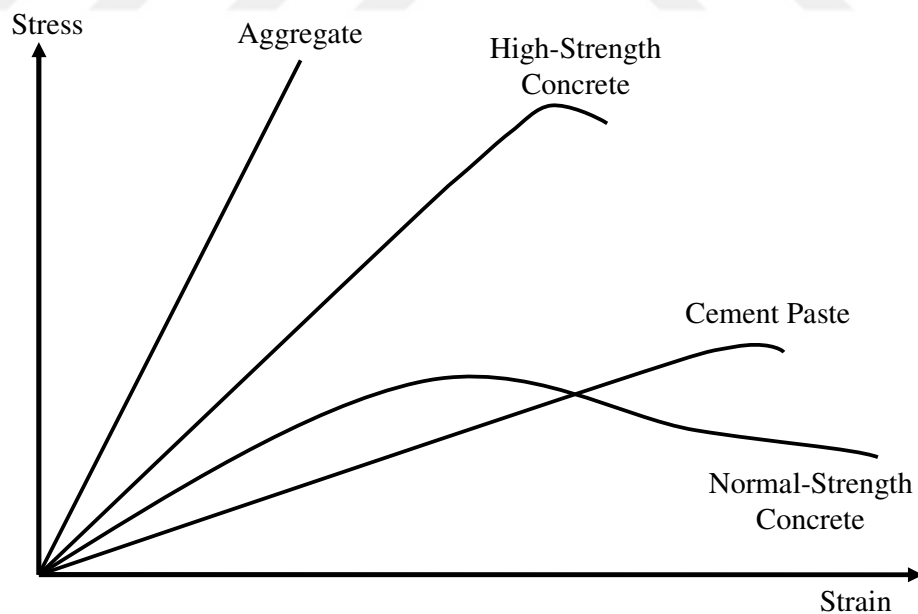


Figure 2-2 – Stress-Strain Relationships for Cement Paste and Aggregate

One of the most significant characteristic differences between NSC and HSC is the failure mode at ultimate. NSC fails more gradually whereas HSC fails suddenly with an explosive manner. The failure mode of HSC occurs almost immediately after the

maximum load is reached. This indicates that the descending branch of stress-strain relationship for HSC is steeper and relatively shorter than that of NSC. The strain at the maximum stress achieved in NSC is in the order of $2000 \mu\epsilon$ whereas HSC has a strain of approximately $3000 \mu\epsilon$ or more at the maximum stress. Also the strain at ultimate for NSC is several times greater than that of HSC. Stress-strain relationships for different concrete compressive strengths obtained by Dahl (1992) are shown in Figure 2-3. The figure indicates that, as the concrete compressive strength increases, the strain at the peak stress increases slightly. Furthermore, the shape of the ascending branch of the stress-strain curve becomes more linear and steeper, and the slope of the descending part also becomes steeper.

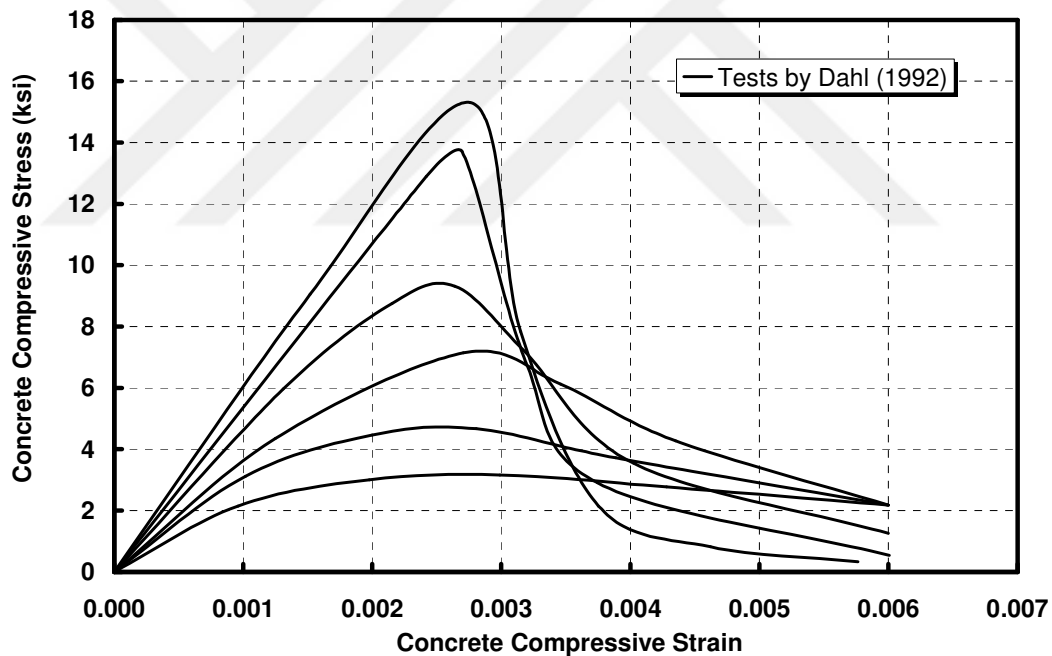


Figure 2-3 – Stress-Strain Relationships for Different Concrete Compressive Strengths

Before the application of any load on concrete, micro cracks develop in the interface between the cement paste and the aggregate due to drying of the cement paste, which is constrained by the non-shrinking aggregate. When NSC is loaded to failure, four distinct stages can be observed in the stress-strain relationship. The first stage, the linear elastic range for concrete, occurs before reaching 30 to 40 percent of cylinder concrete compressive strength (f'_c). The second stage, the beginning of the non-linearity of stress-

2 Background

strain relationship, can be observed between 30 and 50 percent of concrete compressive strength of cylinder. Micro cracks developed in the drying stage increase in length, width and numbers; however a stable system of micro cracks exists. In the third stage, between 50 and 75 percent of the concrete strength, cracks start to form in the cement paste. This causes an unstable system of cracks in the cement paste, which increases the non-linearity of the stress-strain relationship. After 75 percent of the cylinder compressive strength, the forth stage starts with rapid propagation of the cracks in the cement paste and the interface between the cement paste and the aggregate. A continuous crack system occurs in between which causes a rapid increase of the strain. After the ultimate stress is reached, a slow mode of failure happens at the end of this stage, since the cracks are interrupted by the aggregates and must move around the aggregate. On the oter hand, when HSC is loaded to failure, it shows only two stages. The linear elastic stage is before 85 percent of the concrete compressive strength of cylinder. The second stage happens quite fast after 85 percent of the concrete compressive strength of cylinder which is followed by a sudden failure, when cracks pass through the weaker aggregates.

HSC made with the materials explained previously will have the following advantages in comparison to NSC:

- Increased durability due to compactness and reduced permeability
 - More resistance to the chloride damage to reinforcing steel
 - More resistance to acid or other chemical attacks
 - Increased freeze thaw resistance
 - Increased abrasion resistance
 - The HSC structures will require less maintenance, fewer repairs and will last longer
- Design advantages for bridges
 - Increased span length
 - For standard precast prestressed bridge girders, increasing concrete strength from 5000 psi to 7000 psi increased span capabilities of AASHTO girders by about 15 percent (Fiorato 1989)
 - Reduced number of girders in a span

- Reduced section height in a span
- Design advantages for buildings
 - Greater heights can be achieved
 - Smaller sections with reduced dead weight, longer spans with fewer beams
 - Reduction in member size increases the rentable area
 - The higher the concrete compressive strength, the smaller the column cross-section and the more economical the column becomes (Moreno 1998)
 - Changing the specified concrete compressive strength in the core of 735 ft high Bourke Place in Melbourne, Australia from 5800 psi to 8700 psi increased each floors rentable space by 32 yd² resulting in an effective benefit of approximately \$100.000 per floor for the client (Burnett 1989)
 - Reduced axial shortening of the columns due to reduced creep and shrinkage behavior
 - Reduced interstory drift due to the increased stiffness
 - Earlier stripping of the formworks due to high early strength
 - For a fixed column size, with an increase in the concrete compressive strength, there is a significant reduction in the reinforcing steel required.
 - Relative reduction in the percentage of steel in the order of 40 percent for 8000 psi concrete and 67 percent for 12000 psi concrete (Smith and Rad 1989)
 - Reduction of structural steel consumption in the 85 story high-rise T&C Tower in Kaohsiung, Taiwan: the steel ratio is reduced from 1.66 to 1.00 (Hwang et al. 1999)

2.2 Flexural Behavior of HSC

When an under-reinforced concrete beam is loaded incrementally up to failure, the concrete cross-section exhibits 4 different stages. The first three stages are shown in Figure 2-4. The shaded areas represent the concrete in compression. The first stage is before cracking of concrete, when the extreme bottom concrete fiber tensile stresses (f_{t1})

2 Background

reaches the modulus of rupture of concrete (f_r). The tensile stresses are both resisted by concrete and steel whereas the compression stresses are only resisted by concrete. The neutral axis (NA) is a little below the midpoint of the section due to the effect of reinforcement in lowering the center of gravity of the cross-section. In the second stage, cracks initiate from the bottom concrete fiber to have a new equilibrium of forces. From now on the tensile stresses are resisted only by steel (f_s) and compressive stresses are resisted by compression zone of concrete over the neutral axis. The neutral axis shifts upwards and the cracks propagate to the neutral axis. The third stage starts with yielding of steel ($f_{s3} = f_y$). Since the behavior of steel is elasto-plastic, after yielding, stresses in steel do not increase although the strain increases. On the other hand, the concrete has not reached its highest compressive strain at the top fiber of the section. As the strain at the top fiber increases, the compressive stresses in concrete also increase. This raises the neutral axis upwards due to the new equilibrium of forces.

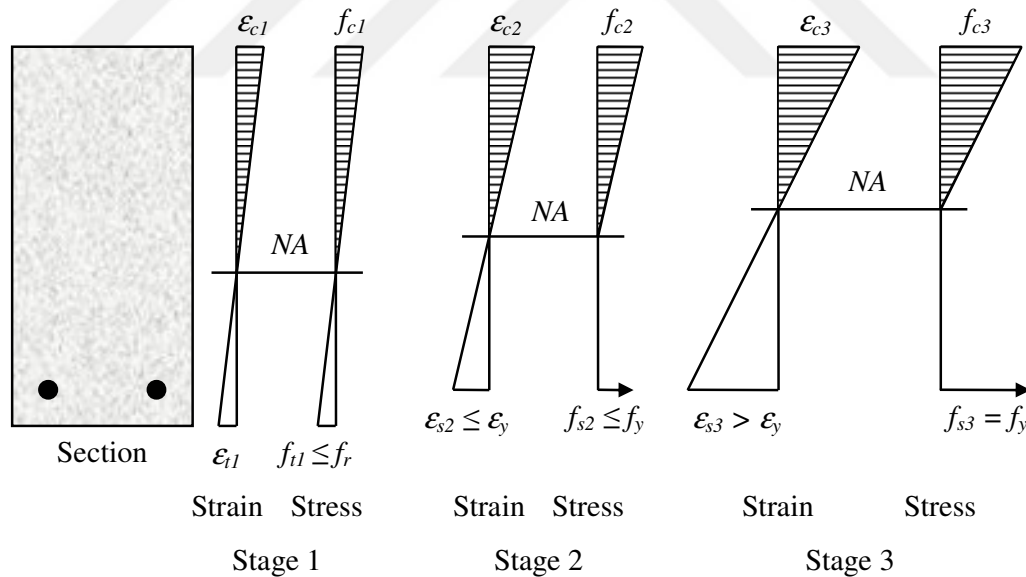


Figure 2-4 – First Three Stages of Concrete Cross-Section Loaded Incrementally

The fourth stage is the failure of concrete beam where the section reaches its ultimate strength capacity. Concrete reaches maximum compressive strain (ϵ_{cu}) at the extreme compression fiber. The concrete in the compression zone has a non-linear stress

distribution similar to its stress-strain relationship which is referred to as the actual stress distribution. This failure stage of concrete cross-section is shown in Figure 2-5.

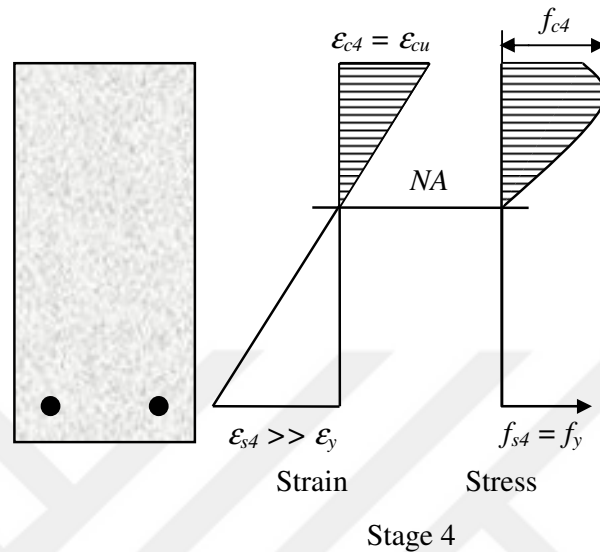


Figure 2-5 – Failure Stage of Concrete Cross-Section (Stage 4)

The design considering the forth stage, the failure stage of the section, is called the ultimate strength design. The procedure for ultimate strength design incorporates four basic assumptions in the calculation of the ultimate strength which are:

1. Plane sections remain plain after deformation; which is based on the development of the beam theory. This implies that the linear compatibility of strains is preserved until the failure of the structural member.
2. The strain in the reinforcement is equal to the strain in the concrete at the same level; which assumes a perfect bond between concrete and reinforcement.
3. The forces can be calculated by using stress-strain relationships of both concrete and steel reinforcement by using the strain compatibility of the section.
4. The tensile stress developed in concrete below the neutral axis is neglected.

The origin of the ultimate strength theory of beams in flexure dates back to Galilei's work in 1638. After the formulation of Hooke's Law in 1680's and the establishment of the fundamentals of theory of elasticity in 1820's by Navier, design turned to allowable stress method which was mathematically more simple. Although theory of elasticity was widely utilized worldwide by the end of 19th century, many researches were performed on

ultimate strength theories to understand the behavior of reinforced concrete beams. The work performed by M. R. von Thullie on flexural theory in 1897 and the parabolic stress distribution of concrete introduced by W. Ritter's in 1899 were good examples on ultimate strength design theory (Hognestad 1951).

2.2.1 Stress Block Parameters

The actual stress distribution in the compression zone of concrete can be mathematically defined by three parameters, k_1 , k_2 and k_3 whose use was initially introduced by F. Stüssi in 1932 (Hognestad 1952). The parameter k_1 is defined as the ratio of the average compressive stress to the maximum compressive stress. The parameter k_2 is the ratio of the depth of the resultant compressive force to the depth of neutral axis. The parameter k_3 is the ratio of the maximum compressive stress ($\sigma_{\max}$) achieved in the structural member to the compressive strength of concrete cylinder (f'_c). The design values of the stress block parameters are determined at the ultimate strain (ϵ_{cu}), which corresponds to the maximum moment of the section. These parameters are shown in Figure 2-6.

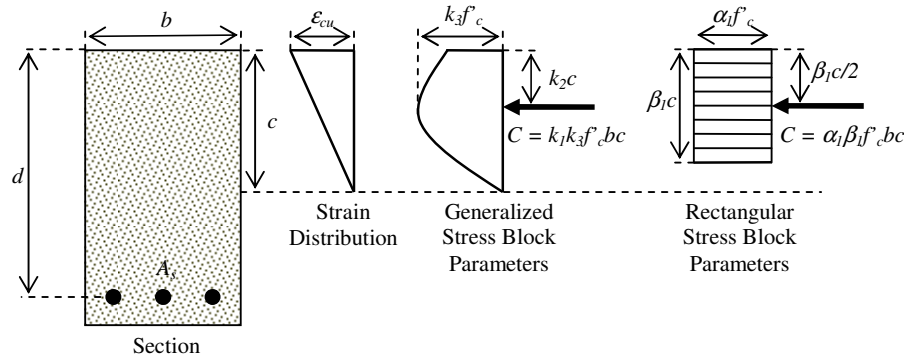


Figure 2-6 – Stress Block Parameters for Rectangular Sections

The k_3 , $k_1 k_3$, and k_2 values can be obtained from the equilibrium of the external and internal forces, as follows:

$$k_3 = \frac{\sigma_{\max}}{f'_c} \quad \text{Equation 2-1}$$

$$C = k_1 k_3 f'_c b c \rightarrow k_1 k_3 = \frac{C}{f'_c b c} \quad \text{Equation 2-2}$$

$$M = k_1 k_3 f'_c bc(d - k_2 c) \rightarrow k_2 = \frac{d}{c} - \frac{M}{(C)c} \quad \text{Equation 2-3}$$

The three-parameter generalized stress block can be reduced to a two-parameter equivalent rectangular stress block, by keeping the resultant of the compression force at the mid-depth of the assumed rectangular stress block. This does not mean that this method is based on the assumption of rectangular stress distribution. The use of the rectangle is only a mathematical device to approximate the effect of the actual stress distribution of concrete. It would be possible to use any curved shape that would give the same resultant force and the center of gravity, but the rectangular area appears to be entirely satisfactory and gives the simplest possible mathematical solution. The use of a rectangular stress block was first proposed by von Emperger in 1904 (Mattock et al. 1961) and since that time this idea was improved by the contributions of many researchers. The rectangular stress block parameters, α_1 and β_1 are presented in Figure 2-6 and can be defined as:

$$k_2 c = \frac{\beta_1 c}{2} \quad \text{Equation 2-4}$$

$$C = \alpha_1 \beta_1 f'_c bc = k_1 k_3 f'_c bc \quad \text{Equation 2-5}$$

therefore

$$\alpha_1 = \frac{k_1 k_3}{2 k_2} \quad \text{Equation 2-6}$$

$$\beta_1 = 2 k_2 \quad \text{Equation 2-7}$$

The behavior of concrete in flexure is not the same as that of concrete cylinder in pure compression. The primary reason is the distribution of stresses in concrete; the strain gradient effect in flexure helps concrete to achieve higher strains than that in pure compression. Other reasons are the shape and size effects of the concrete cylinder compared to the real reinforced concrete structural member. Furthermore, the rate of loading of a structural member is always much slower than that of a concrete cylinder. Also the water rise of a structural concrete member in the casting process may increase this difference more. However, the stress distribution of concrete in flexure may still be represented adequately by the stress-strain relationship of the concrete cylinder using an

empirical constant (k_3) to account for all of these differences. This constant is determined by comparing the beams tested in flexure to the companion cylinders tested under compression. This method is widely used in the analysis and design of structural members. Initially a concrete cylinder is tested in a compression machine to obtain the maximum stress in pure compression which is also referred to as cylinder compressive strength. Many researchers have proposed various equations to get the stress-strain relationship using only one variable, the cylinder compressive strength. By using one of these relationships, the stress-strain curve for specified cylinder compressive strength is obtained for concrete. Next step is to reduce the relationship by using this constant (k_3) that accounts for the factors stated before. The reduced curve is used in design and analysis of the structural members.

The stress-strain relationship of concrete members in flexure is difficult to determine by direct experimental means. The strains and applied loads can be measured easily whereas to obtain the stresses in concrete requires a numerical differentiation of the measured quantities. Hognestad et al. (1955) derived the equations to calculate the concrete stress as a function of continuously monitored strain of the most compressed fibers and applied loads on the concrete member. The detailed derivation is given in Chapter 5.

In the next section, the tests related to determination of stress-strain relationship of concrete members in flexure are explained.

2.2.2 Eccentric Bracket Specimen Tests

Many researchers have been working on the stress-strain distribution of concrete since the beginning of the 20th century. Hognestad et al. (1955) developed a test set-up and derived equations which were milestones in evaluation of the stress-strain relationship of concrete. In this test set-up, the compression zone of a flexural member with a rectangular cross-section is simulated by varying the axial load and the moment on the section. By many researchers, this test set-up was referred to as Hognestad Test Set-Up and these specimens were referred to as “C-Shaped Specimens” or “Eccentric Bracket Specimens”. The eccentric bracket specimen tests by different researchers will be presented in details in the following sections. These specimens are mostly HSC however; NSC specimens are also presented for comparison purposes.

2.2.2.1 Hognestad et al. (1955)

Hognestad et al. (1955) conducted an experimental program and developed a test method to investigate the distribution of concrete stresses in flexure. This method formed the basis for the future research on determination of stress block parameters of concrete. The test variables included were concrete compressive strength and age of concrete. For this purpose a total of 20 bracket specimens with cylinder concrete compressive strengths ranging from 0.775 to 7.61 ksi were tested under combined axial load and bending. The details of the specimens are presented in Figure 2-7. The central unreinforced test region had a cross-section of 5×8 in. and was 16 in. long. The brackets were heavily reinforced to obtain a failure in the central unreinforced test region. The specimens were cast horizontally and tested vertically.

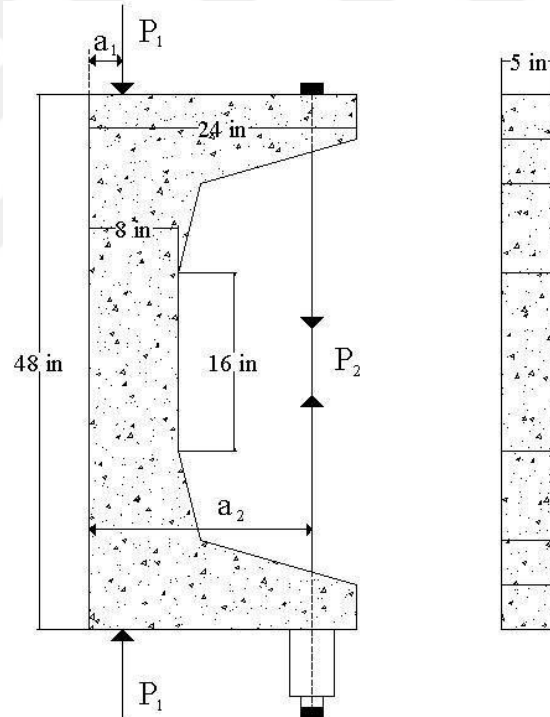


Figure 2-7 – Test Specimen by Hognestad et al. (1955)

The test method consisted of applying a major load, P_1 , using a testing machine and a minor load, P_2 , that could be varied independently to maintain the neutral axis at one face of the test specimen throughout the test. This eliminated any complications resulting from tensile stresses in the concrete. The major load was applied through a 3/16 in. roller at a constant rate from zero to failure. The minor load was applied using a hydraulic jack through one or two tie rods and varied throughout the test to obtain zero strain within ± 5

$\mu\epsilon$ at one face of the specimen. Strains were measured using 6 in. strain gages, two at the neutral surface, two at the compression surface and one at mid-depth on each of the two side faces. The test duration was 15 minutes which corresponded to a rate of 3.1 to 4.2 microstrains per second on the compression face. Three or four 6×12 in. concrete cylinders were tested with each specimen. The testing ages were 7, 14, 28 and 90 days.

The stress-strain relationships and numerical values were obtained which characterized the properties of the stress block. The numerical data is presented in Appendix A. The results confirmed the research data on tests of reinforced concrete structural members.

2.2.2.2 Soliman et al. (1967)

Soliman et al. (1967) investigated the stress-strain relationship of confined concrete to understand the plastic deformation capacity of critical regions reinforced with longitudinal and transverse reinforcement. The test variables included in the testing program were spacing, size and type of transverse reinforcement; the shape of the concrete cross-section; and the thickness of the cover. Sixteen specimens were tested under combined axial load and bending. A schematic view of the test set-up is shown in Figure 2-8. All of the specimens had a cross-sectional dimension of 4×6 in. except for two specimens that had cross-sectional dimensions of 3×6 and 5×6 in., respectively. The central test region was 20 in. long.

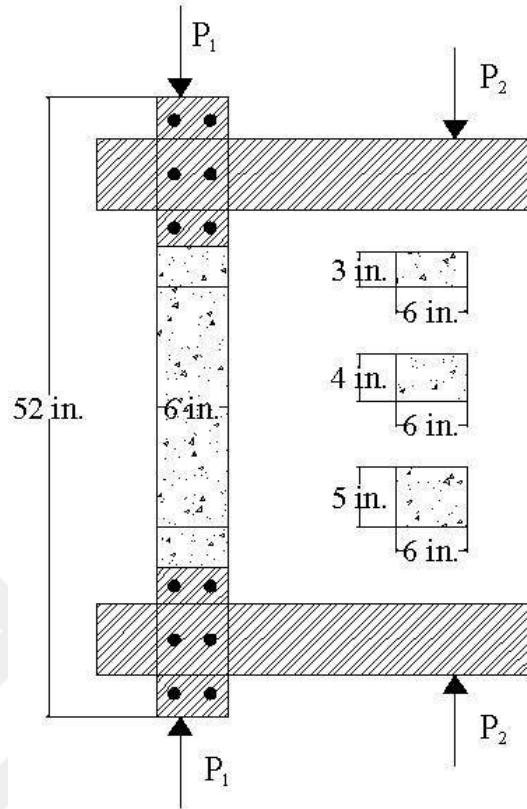


Figure 2-8 – Test Specimen by Soliman et al. (1967)

The specimens were tested under the action of major load, P_1 , applied directly to the specimen and a minor load, P_2 , applied through two steel brackets fixed to the ends of the specimen. The two loads were applied to maintain the neutral axis depth constant near the tension side of the specimen throughout the entire range of loading. The major load was applied using a 220 kip hollow-ram jack reacting against the laboratory floor. The minor load was applied to the brackets by jacking against a 7/8 in. diameter high tensile steel bar. A 66 kip hollow-ram jack was used to apply the load. A 220 kip and an 11 kip load cells were used to measure the major and minor loads, respectively. Each steel bracket was composed of two 6 in. channel beams, each welded to a 6×12×3/4 in. steel plate by means of which the brackets were fixed to the specimen. Six symmetrically loaded 7/8 in. holes were drilled in each plate for this purpose. Four 2.36 in. polyester-base electric strain gages were used to determine the position of the neutral axis. Five mechanical demountable strain gages were used to measure the strains along each side of the specimen. Two clinometers (optical devices for measuring elevation angles) were located on the top of the brackets to measure the total rotation of the specimen. Three deflection gages 6 in. apart were used to measure the deflection at three points along the test region.

The test duration was around 120 minutes up to first signs of cracking. Afterwards, the applied load was controlled by increasing the strain at critical section by about 0.0007 microstrains.

It was concluded that the effect of transverse reinforcement could not be solely expressed as a function of the volumetric ratio. The experimental evidence indicated that the expressions proposed can be used to describe the stress-strain relationship of confined concrete accurately.

2.2.2.3 Sargin et al. (1971)

Sargin et al. (1971) performed an experimental investigation on complete stress-strain relationship of confined concrete to explain and formulate the behavior of concrete confined by rectilinear lateral reinforcement. The test variables included were concrete compressive strength; size, spacing and grade of lateral reinforcement; strain gradient; and thickness of cover. For this purpose, total of 63 specimens were tested under concentric and eccentric loading. For the purpose of this research, only the eccentric bracket specimens are presented in this section. A total of 4 plain and 10 reinforced concrete specimens with cylinder compressive strengths ranging from 3.0 to 4.7 ksi were tested under combined axial load and bending. A schematic view of the test set-up is shown in Figure 2-9. The central test region had 5×5 in. cross-section and was 10 in. long. Thirteen specimens were cast vertically whereas only one specimen was cast horizontally.

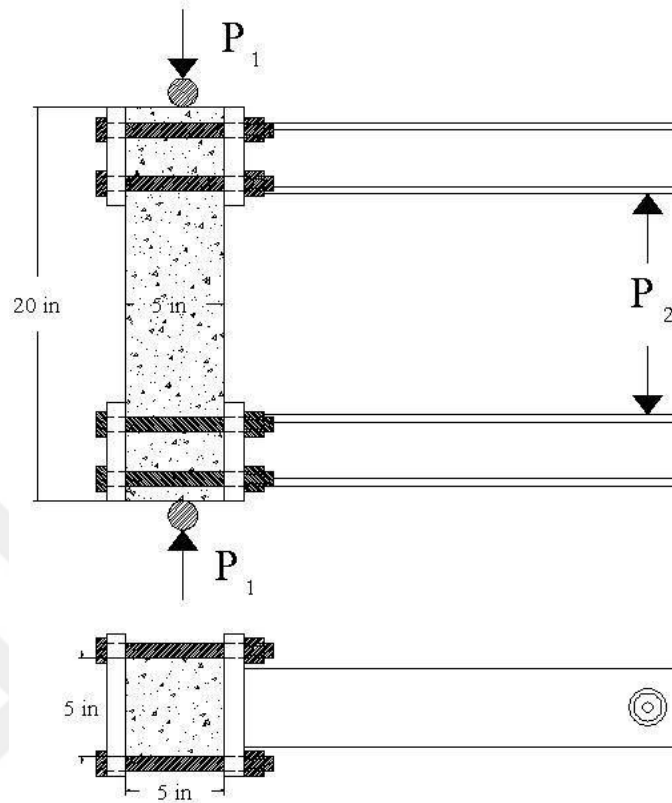


Figure 2-9 – Test Specimen by Sargin et al. (1971)

Two independent loads were applied to have zero strain on one face of the specimen. The primary load, P_1 , was applied through cylindrical rollers and grooved plates by a screw-type Riehle machine of 200 kip capacity. The top roller was also used as a load-cell for measuring the load applied to the specimen. The secondary load, P_2 , was applied using a 10 kip hydraulic jack, through two steel brackets consisting of 4 in. wide flange I beams attached to two ends of the specimen by means of 7/8 in. diameter high tensile steel bolts. A 20 kip load cell was used to measure this load. Since the stiffness of the testing machine was not enough to follow the descending branch of the load deformation curves, a spring system was used to prevent any sudden release of energy from the testing machine. Displacement transducers with 5 in. length were used to measure the longitudinal strains on opposite two faces of specimen. All the specimens were tested vertically. Test durations varied from 45 to 90 minutes.

The parameters of an analytical stress-strain relationship for plain and laterally reinforced concrete was determined from a regression analysis of the test results. It was concluded that the amount of confinement provided by lateral reinforcement was not only dependent on the volumetric ratio of the lateral reinforcement but also on the type of lateral reinforcement (rectangular, spiral, etc.), spacing and grade of lateral reinforcement and the quality of confined concrete.

2.2.2.4 Nedderman (1973)

Nedderman (1973) used the same method and specimens similar in dimensions developed by Hognestad et al. (1955) (Figure 2-7) to determine the concrete stress block coefficients for normal weight concrete. The only test variable was concrete compressive strength. 13 bracket specimens with cylinder compressive strengths ranging from 11.49 ksi to 14.14 ksi were tested to failure under combined axial load and bending. Due to the test set-up problems, only 9 could be tested successfully. The brackets were not reinforced using steel. Instead, the minor load was clamped to the bracket with large clamps made with 1/2 in. round steel bars and 3/4 in. thick plates were used as an auxiliary reinforcement in order to have failure in the testing region. The casting positions were not mentioned.

The major load, P_1 , was applied using a Tinius Olsen 400 kip capacity testing machine with decreasing load increments close to the assumed failure load. The major load was applied through a combination of 3/8 in. thick masonite pad and 40 in² aluminum plate. The minor load, P_2 , was applied using a 60 kip capacity hydraulic jack through four round steel rods and measured using pressure dial gage attached to the jack. The location of the six strain gages was identical to specimens by Hognestad et al. (1955). The agreement of the two strain gages on the compression side was within 10 percent over the full range of loading; the agreement between the gages on the sides was 20 to 30 percent. The test duration was ranging from 30 to 45 minutes which corresponded to a rate of 1.1 to 1.7 microstrains per second at the compression face. While testing concrete cylinders, the testing equipment could not break the cylinders because the strength exceeded the upper load limit. After the maximum load was reached, the cylinders failed within 12 seconds to 1.5 minutes. Therefore the values are not the true short term ones. The testing ages were 57 and 63 days.

Overall, it was hard to understand how the k_3 values derived from the f'_c values derived from out-of-standard tests. It was recommended that, 0.7 would be a suitable β_1 value for concrete compressive strengths higher than 7 ksi. Another recommendation was to further investigate a second degree parabola with a maximum intensity of $0.88 f'_c$ at its apex which coincided with the edge of the specimen. This provided an almost exact correlation with the actual stress distribution observed in the tests. The numerical data is presented in Appendix A.

2.2.2.5 Kaar et al. (1978a)

Kaar et al. (1978a) used the same method and specimens similar in dimensions developed by Hognestad et al. (1955) (Figure 2-7) to investigate the flexural behavior of HSC. The test variables were concrete compressive strength and type of aggregate. For this purpose, 19 bracket specimens made from normal weight aggregate with cylinder compressive strengths ranging from 6.5 to 14.85 ksi and 15 bracket specimens made from light weight aggregate with cylinder compressive strengths ranging from 3.56 to 12.49 ksi were tested under combined axial load and bending. The specimens were cast horizontally and tested vertically.

The major load, P_1 , was applied using a 1000 kip capacity testing machine. The force was applied through a system of bearing plates and rollers that accommodated rotation of the specimen during the test. The minor load, P_2 , was applied using a hydraulic ram through a system of rods, crossheads and rollers. During the test, the major load was increased at a constant rate. The strain at the neutral face was kept at zero by controlling the value of the minor load manually. The opposite face of the cross-section was subjected to a monotonically increasing compressive strain. Strains were measured at mid-height of the section using two 2.5 in. electrical resistance strain gages on the neutral surface, two gages on the compressive face and one gage at mid-depth of each side of the two side faces. A direct current differential transformer was used to monitor the deflection of the test section relative to the ends. Three 6×12 in. concrete cylinders were tested with each specimen.

The stress-strain relationships and numerical values were obtained which characterized the properties of the stress block. The numerical data is presented in Appendix A. The

results were compatible with the previous researches for normal weight concrete. The data for light weight aggregate concrete indicated that β_1 should be reduced for concrete compressive strengths less than 8 ksi.

2.2.2.6 Kaar et al. (1978b)

Kaar et al. (1978b) performed a testing program to evaluate the effects of the rectangular hoops as confinement reinforcement on the behavior of reinforced concrete columns. The results were used to determine the effective stress-strain relationship of confined concrete. The main test variables were concrete compressive strength, hoop size, hoop spacing, amount of longitudinal reinforcement and specimen size. The specimens used in this research were similar to the ones used by Hognestad et al. (1955). For this purpose, 6 specimens with 5×8 in. cross-section and 11 specimens with 10×16 in. cross-section were tested under combined axial load and bending. The lengths of the testing regions were 16 and 32 in, respectively. Cylinder compressive strengths were ranging from 2.78 to 6.47 ksi. The details of the test set-up are presented in Figure 2-10. Three of these specimens were unreinforced in the test region. All others had longitudinal and transverse reinforcement in the test region. The reinforcement in the arms of the specimen was design to resist the stresses in the end region during testing.

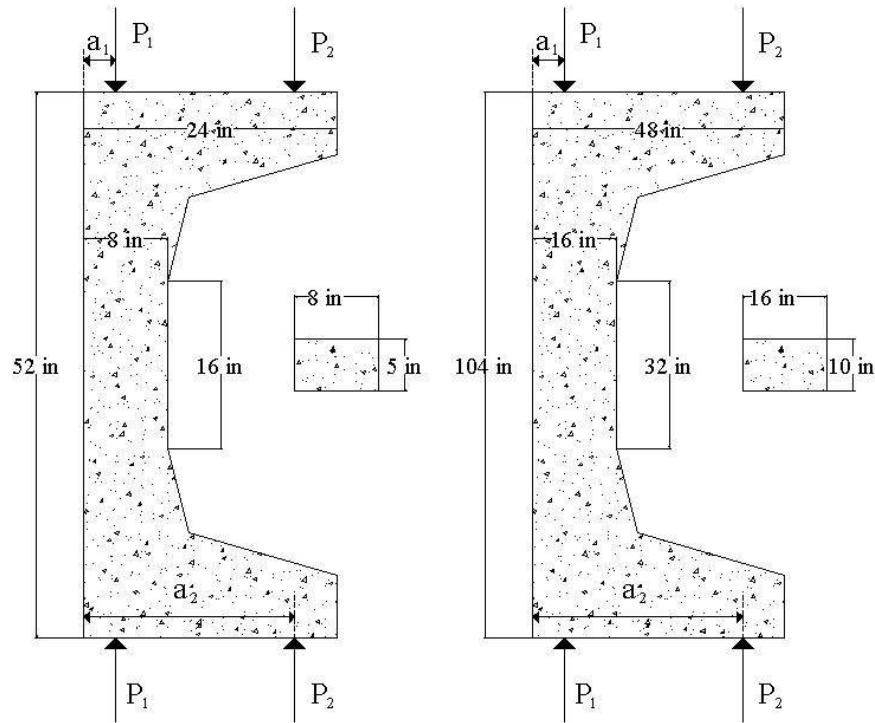


Figure 2-10 – Test Specimens by Kaar et al. (1978b)

The same test method and testing machine used in Kaar et al. (1978a) was applied on the specimens. The instrumentation on the specimens was also same. Three 6×12 in. concrete cylinders were tested with each specimen.

The stress-strain relationships for confined and unconfined concrete were obtained. The numerical data is presented in Appendix A. The results showed that the specimens with hoop reinforcement had significantly greater strains than could be obtained with plain concrete specimens. It was concluded that the amount of reinforcement was the primary variable affecting the stress-strain relationship. For other variables, effects were not distinguishable within the normal scatter of experimental results. An analytical stress-strain relationship proposed earlier was updated and suggested.

2.2.2.7 Swartz et al. (1985)

Swartz et al. (1985) performed an experimental investigation of the flexural properties of higher strength concrete. The test method developed by Hognestad et al. (1955) was used to test specimens with different dimensions. The only test variable was concrete compressive strength. For this purpose a total of 10 bracket specimens with cylinder

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compressive strengths ranging from 8.4 to 11.18 ksi were tested under combined axial load and bending. The details of the specimens are presented in Figure 2-11. The central unreinforced test region had a cross-section of 5×8 in. for one of the specimens and 5×5 in. for all others and was 16 in. long. The brackets were heavily reinforced to obtain a failure in the central unreinforced test region. The bracket specimens were cast horizontally and tested vertically. In addition to these, four reinforced rectangular beams with cylinder compressive strengths ranging from 11.5 ksi to 12.31 ksi were constructed and tested under four point bending. These beams were reinforced longitudinally with $0.5\rho_b$ and $1.5\rho_b$ and one had no stirrups, one had 50 percent of the required stirrup area and two had full required stirrup area based on the ACI 318 requirements. The beam specimens were cast horizontally. Only the bracket specimens are reported here.

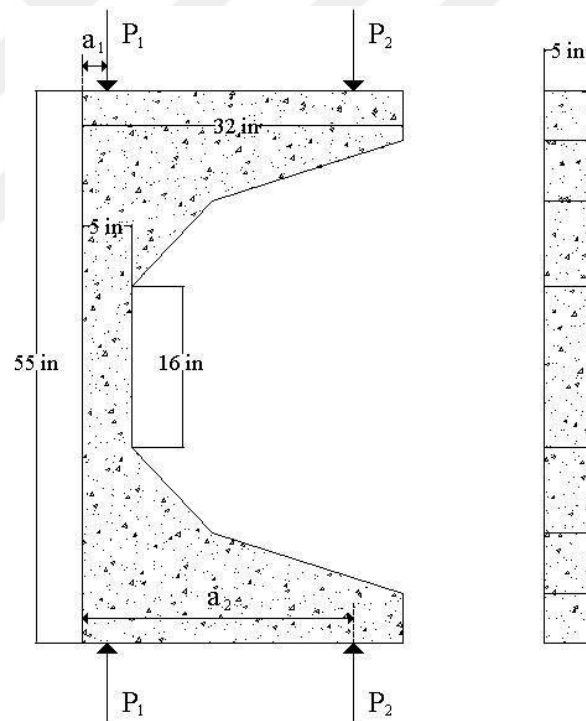


Figure 2-11 – Test Specimen by Swartz et al. (1985)

For the bracket specimens, the primary load, P_1 , was supplied using a 300 kip, load controlled Emery-Tatnall hydraulic testing machine. The secondary load, P_2 , was applied using a hand operated hydraulic jack. The testing procedure was to apply an incremental primary load, maintain this and apply the secondary load while monitoring the two strain gages on the neutral face. When they read zero, the secondary load was maintained while

the other strain readings were taken. For the beam specimens, the loads were applied incrementally to failure with crack growth being monitored. Strains of the bracket tests were measured using 2.4 in. electrical resistance wire gages and 0.7 in. electrical resistance foil gages. Two of these gages were applied at the neutral surface, two at the compression surface and three at each of the two side faces. The test duration was 15 minutes which corresponded to a rate from 3.1 to 4.2 microstrains per second. Concrete cylinders with 3×6 in. dimensions were tested with each specimen.

It was concluded that:

1. The compressive stress block in the beam at failure was curved and may be presented by a parabola with a maximum fiber stress.
2. The strain at ultimate load in the extreme fiber in the beam might be less than 3000 $\mu\epsilon$. A recommended value for design was 2500 microstrains.
3. The secant modulus elasticity might be estimated reasonably well by the ACI formula.
4. The uniaxial stress-strain relationship was almost linear up to stress values about 0.50 of maximum fiber stress.
5. The beam and cylinder compressive stress-strain relationships were similar.

The numerical data is presented in Appendix A.

2.2.2.8 Pastor (1986)

Pastor (1986) investigated bending properties of concrete by using eccentric bracket specimens. The only test variable was concrete compressive strength. For this purpose a total of 10 out of 13 bracket specimens with cylinder compressive strengths ranging from 2.596 to 11.574 ksi were tested under combined axial load and bending. Eccentric bracket specimens, similar to the ones originally used by Hognestad et al. (1955) (Figure 2-7) were tested. 3 specimens could not be tested due to handling problems and a severe honeycomb in the lower end of the section. The bracket specimens were cast and tested vertically.

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The major load, P_1 , was applied through a system of bearing plates 5×8×3 in. and rollers 1 in. diameter. The minor load, P_2 , had basically the same bearing plate and roller system only with smaller dimensions. These loading systems accommodated rotation of the specimen during test. Hydrostone was used between the bearing plates and the specimen to avoid bearing problems. The major load was applied in increments. After each increment, the minor load was adjusted with the hand-pump so that strain on the neutral face would be zero. Strain gages were distributed with three gages on the neutral face, two gages on the compression face and two gages on roughly the 1/3 point and 2/3 point of the sides. The test duration was ranging from 15 to 20 minutes which corresponded to a rate of 2.5 to 3.33 microstrains per second. Concrete cylinders were tested with each specimen.

The stress-strain relationships and numerical values were obtained which characterized the properties of the stress block. Also it was indicated that, the initial modulus of elasticity of the flexural tests were 1.08 times that of compression tests. The numerical data is presented in Appendix A.

2.2.2.9 Schade (1992)

Schade conducted an experimental investigation to confirm the previous findings in terms of similarity of the bending properties and stress-strain distribution of concrete. A new approach for bracket specimens was adapted to minimize the experimental cost. Instead of a bracket specimen, a rectangular column was tested under axial load and bending by attaching two steel cantilever arms (Hollow Structural Steel – 3×2×0.19 in) at the ends to apply the secondary load, P_2 . Two transverse pieces of the same steel sections were placed on the cantilevers in the location of the hydraulic jack of the secondary load. The test variable was strength of concrete. For this purpose, 6 unreinforced and 6 reinforced concrete column specimens with cylinder compressive strengths 15.36 ksi for unreinforced specimens to 15.9 ksi for reinforced specimens were tested under combined axial load and bending. The details of the specimens are presented in Figure 2-12. The columns had a cross-section of 6×6 in. and were 24 in. long. The column specimens were cast and tested vertically. The testing age was 10 days.

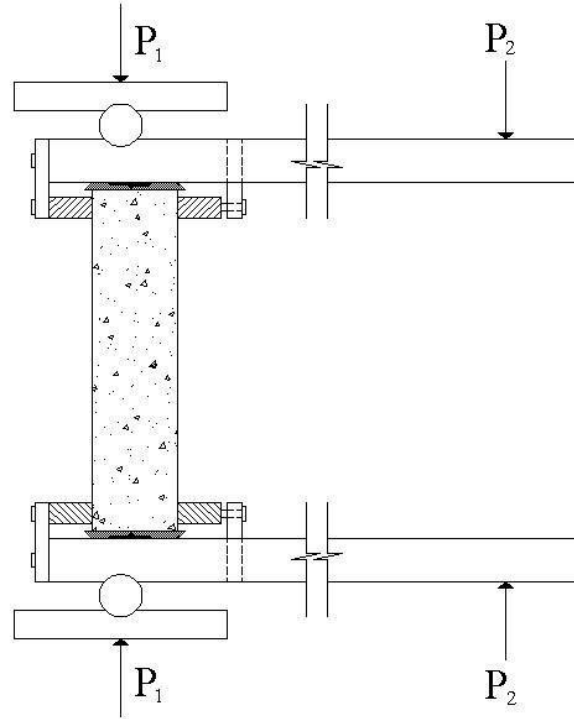


Figure 2-12 – Test Specimen by Schade (1992)

The primary load, P_1 , was applied using a servo-controlled closed-loop MTS hydraulic testing machine with 450 kip capacity. This load was transferred from the machine to the column through a ball and socket arrangement. The secondary load, P_2 , was applied using a hand operated hydraulic jack measured with a load cell. Strains in concrete were measured using 2 in. electrical strain gages. One gage was located on the centerline of each face of the column. Brass target points and a mechanical strain indicator and strain gage based displacement transducers were also used to correlate gage readings. Strains in longitudinal steel were measured using 0.2 in. electrical strain gages. Testing was performed using stroke control. The average test duration was 10 minutes which corresponded to a rate from 5 to 6 microstrains per second at the compression face. Concrete cylinders with 4×8 in. dimensions were tested with each specimen. The loading rate for the concrete cylinders was 2.8 microstrains per second.

The stress-strain relationships and numerical values were obtained which characterized the properties of the stress block. It was concluded that:

1. Plane sections remain plane after deformation.

2. Maximum failure strains were measured far above 3000 microstrains.

The numerical data is presented in Appendix A.

2.2.2.10 Ibrahim (1994)

Ibrahim (1994) performed an experimental program to investigate the use of rectangular stress block for HSC and ultra HSC sections. The main test variables were concrete compressive strength, shape of the section and the confinement steel (diameter, spacing and volumetric ratio). For this purpose, 14 bracket specimens with rectangular cross-section in the test region and 6 bracket specimens with triangular cross-section in the test region were tested under combined axial load and bending. Cylinder compressive strengths were ranging from 8.6 to 18.75 ksi for specimens with rectangular cross-section and from 11.75 to 18.94 ksi for specimens with triangular cross-section in the test region. The details of both specimens are presented in Figure 2-13 and Figure 2-14. The central test region had 8×12 in. cross-section and was 47 in. long for the rectangular specimens. This region had a triangular section with 9.5 in. height and 17.7 in. base side length for the triangular specimens. The section of the test region had a flat edge at the apex of the triangular giving a width of 0.8 in. in order to mount strain gages in the most highly strained face. Three of the rectangular and two of the triangular specimens didn't have any reinforcement in the test region; 11 of the rectangular and 4 of the triangular had vertical and horizontal reinforcement. The brackets were heavily reinforced to obtain a failure in the central unreinforced test region. The specimens were cast and tested vertically. Testing age was ranging from 42 to 186 days.

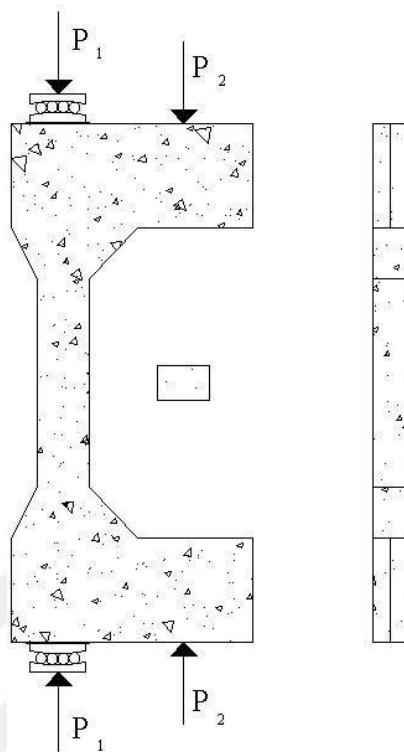


Figure 2-13 – Rectangular Test Specimen by Ibrahim (1994)

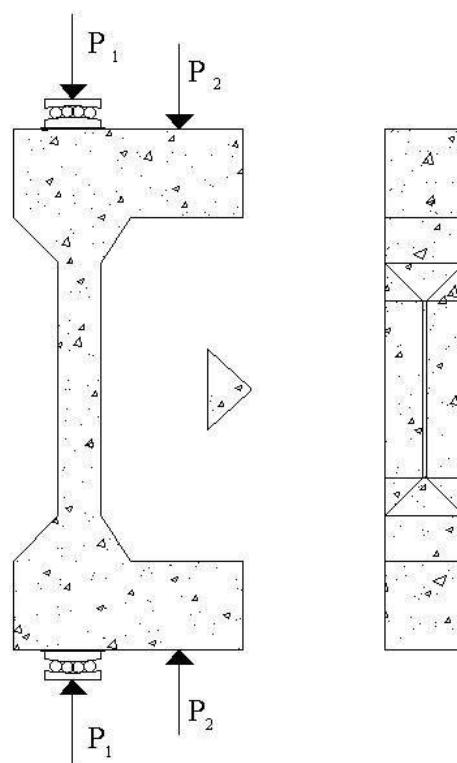


Figure 2-14 – Triangular Test Specimen by Ibrahim (1994)

The primary load, P_1 , was applied using a 1500 kip MTS machine. This load was transferred to the specimen through a system of curved plates and rollers that had a well defined center of rotation. The secondary load, P_2 , was applied using a 120 kip hydraulic jack through a system of steel plates, pins anchor rods and bolts attached at the top and the bottom arms of the specimen. During each test, as the primary load changed, the secondary load was adjusted accordingly to obtain zero strain at the neutral face. The deformations of the specimens were measured using both 1 and 4 in. strain gages and linear variable displacement transducers (LVDT). The total number of strain gages on all the sides of the specimens was varying between 53 and 77. Testing was performed using stroke control. The average test duration was ranging from 180 to 420 minutes which corresponded to a rate from 0.33 microstrains per second at the compression face. This slow rate was adapted to reduce the fluctuations in the primary and secondary loads. Both 4×8 in. and 6×12 in. concrete cylinders were tested with each specimen.

The test program was successful in providing data relating to the flexural behavior of confined and unconfined HSC sections. It was concluded that:

1. The failure of plain concrete and lightly reinforced concrete sections is very brittle.
2. Columns with tie spacing equal to the least column dimension failed suddenly when the column spalled.
3. A well confined HSC section can show a ductile behavior, maintaining the applied loads through large deformations.
4. The triangular compression zones exhibit more ductile behavior than the rectangular compression zones.

The numerical data is presented in Appendix A.

2.2.2.11 Tan and Nguyen (2005)

Tan and Nguyen (2005) conducted an experimental program to investigate the effects of confinement of lateral reinforcement on the flexural capacity of HSC and the applicability of equivalent rectangular stress block of the ACI code to HSC sections. The main test variables were strength of concrete and the confinement reinforcement. For this purpose,

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5 specimens with square cross-section were tested under concentric compression, 17 specimens under eccentric compression with a fixed neutral axis position and 8 specimens under eccentric compression with a fixed eccentricity. Only the specimens under eccentric compression with a fixed neutral axis position are reported here. Cylinder compressive strengths were ranging from 6.6 to 14.59 ksi. The details of the test set-up are presented in Figure 2-15. The specimens had 8×8 in. cross-section and were 47 in. long. Three of the specimens were unreinforced in the test region. All others had longitudinal and transverse reinforcement in the test region. The ends of the specimens were heavily reinforced with longitudinal bars welded to steel plates. Each steel plate facilitated the fixing of the specimen to the loading rig. The specimens were cast horizontally and tested vertically.

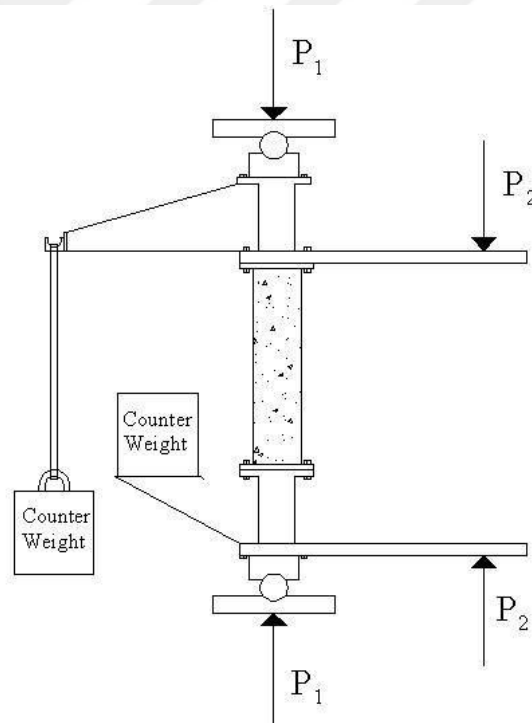


Figure 2-15 – Test Set-Up by Tan and Nguyen (2005)

The primary load, P_1 , was applied using 1125 kip servo-hydraulic testing machine. This load was transferred to the loading rig through a system of roller and plate combination. The secondary load, P_2 , was applied using a manually operated hydraulic jack. During each test, the strain at one face of the specimen was monitored and maintained at zero by manually adjusting the load from the hydraulic jack. The longitudinal deformations of the

specimens were measured using two pairs of LVDTs on opposite side of the specimen in the bending plane. Two 0.4 in. stroke LVDTs were used to measure the most compressive fiber deformation, while for the opposite face, a pair of 0.2 in. stroke LVDTs were employed. Five 1 in. LVDTs were used to measure the lateral displacements along the height of the specimen. Testing was performed using stroke control. The average test duration was ranging from 24 to 39 minutes which corresponded to a rate from 2.2 microstrains per second at the compression face. Three 6×12 in. concrete cylinders were tested with each specimen.

A new one parameter stress block was proposed and it was concluded that:

1. The confinement from lateral reinforcement improved the flexural strength and ductility of the reinforced concrete columns.
2. The effect of configuration of lateral reinforcement was significant to the flexural behavior of confined concrete.
3. Confinement reinforcement made of high-yield-strength steel was more effective than that made of lower-yield strength steel in confining HSC columns.
4. The strength of concrete has the most significant effect on the flexural stress block parameters.
5. The allowable compressive strain of concrete under flexure, as specified in the ACI Code, was a lower-bound value for the measured strains of reinforced concrete specimens.

The numerical data is presented in Appendix A.

2.2.2.12 Summary of the Eccentric Bracket Tests to Date

This section provides a comprehensive summary of eccentric bracket tests constructed from normal and HSC. The outline of the tests performed by different researchers is presented in Table 2-1. The generalized and rectangular stress block parameters obtained from these tests are shown in graphs from Figure 2-16 to Figure 2-22. The ultimate compressive strain of concrete obtained from eccentric bracket tests are also shown in Figure 2-23. Since not all of the parameters of stress blocks were reported in some papers, some of the graphs do not include some of the researchers. The presented research data will be used to compare the equations for stress block parameters in the next sections.

Table 2-1 – Summary of the Eccentric Bracket Tests

Testing By	# of Specimens	Concrete Compressive Strength (ksi)	Cross-Section of Testing Region (in. ×in.)	Length of Testing Region (in.)	Type of Testing Region	Test Duration (min)	Testing Rate (µε/sec)
Hognestad et al. (1955)	20	0.775 – 7.61	5×8 Rectangular	16	Unreinforced	15	3.1 – 4.2
Soliman et al. (1967)	16	–	1 – 3×6, 14 – 4×6, 1 – 5×6 Rectangular	20	Reinforced	More than 120	Not Specified
Sargin et al. (1971)	14	2.988 – 4.684	5×5 Rectangular	10	4 – Unreinforced 10 – Reinforced	45 - 90	0.9 – 2.2
Nedderman (1973)	13 (9 successful)	11.49 – 14.14	5×8 Rectangular	16	Unreinforced	30 – 45	1.1 – 1.7
Kaar et al. (1978a)	19	3.56 – 14.85	5×8 Rectangular	16	Unreinforced	Not Specified	Not Specified
Kaar et al. (1978b)	17	2.78 – 6.47	6 – 5×8, 11 – 10×16 Rectangular	16 and 32	3 – Unreinforced 14 – Reinforced	Not Specified	Not Specified
Swartz et al. (1985)	10	8.4 – 11.18	1 – 5×8 Rectangular 9 – 5×5 Rectangular	16	Unreinforced	15	3.1 – 4.2
Pastor (1986)	13 (10 successful)	2.596 – 11.574	5×8 Rectangular	16	Unreinforced	15 – 20	2.5 – 3.33
Schade (1992)	12	15.36 – 15.9	6×6 Rectangular	24 (Column Length)	6 – Unreinforced 6 – Reinforced	10	5 – 6
Ibrahim (1996)	14	8.6 – 18.75	8×12 Rectangular	47 (Column Length)	3 – Unreinforced 11 – Reinforced	180 – 420	0.33
	6	11.75 – 18.94	h=9.5, a=17.7 Triangular		2 – Unreinforced 4 – Reinforced		
Tan and Nguyen (2005)	17	6.6 – 14.59	8×8 Rectangular	47 (Column Length)	3 – Unreinforced 14 – Reinforced	24 – 39	2.2

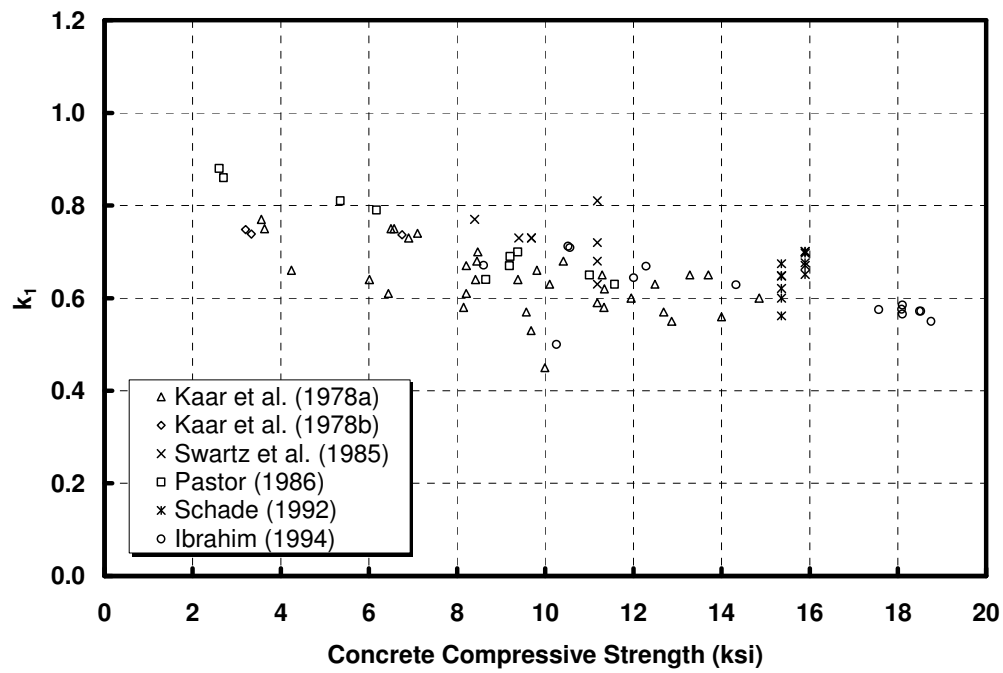


Figure 2-16 – k_1 Parameter from Eccentric Bracket Tests

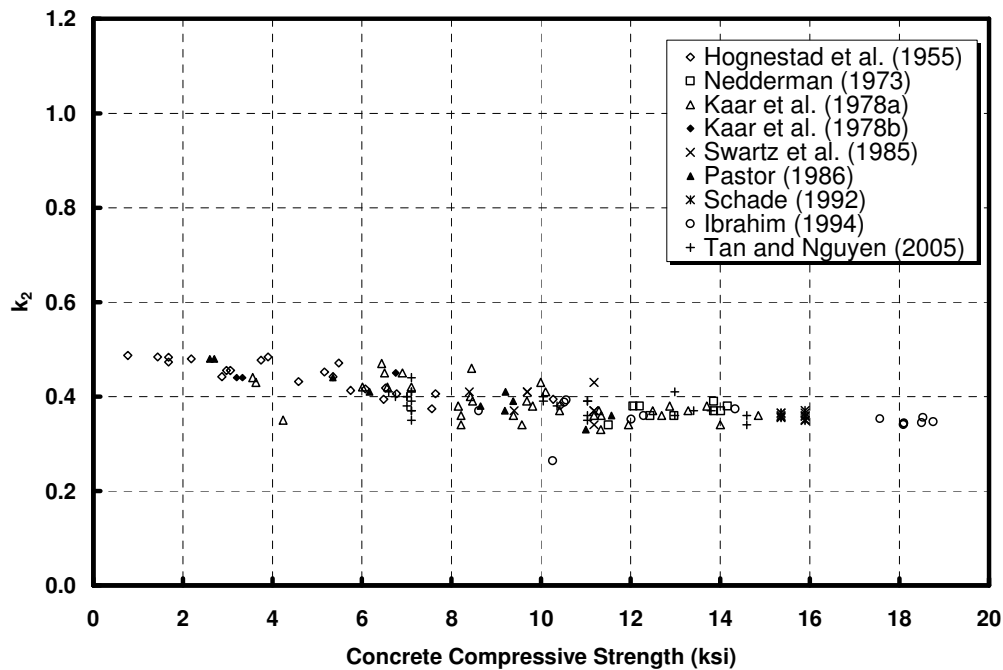


Figure 2-17 – k_2 Parameter from Eccentric Bracket Tests

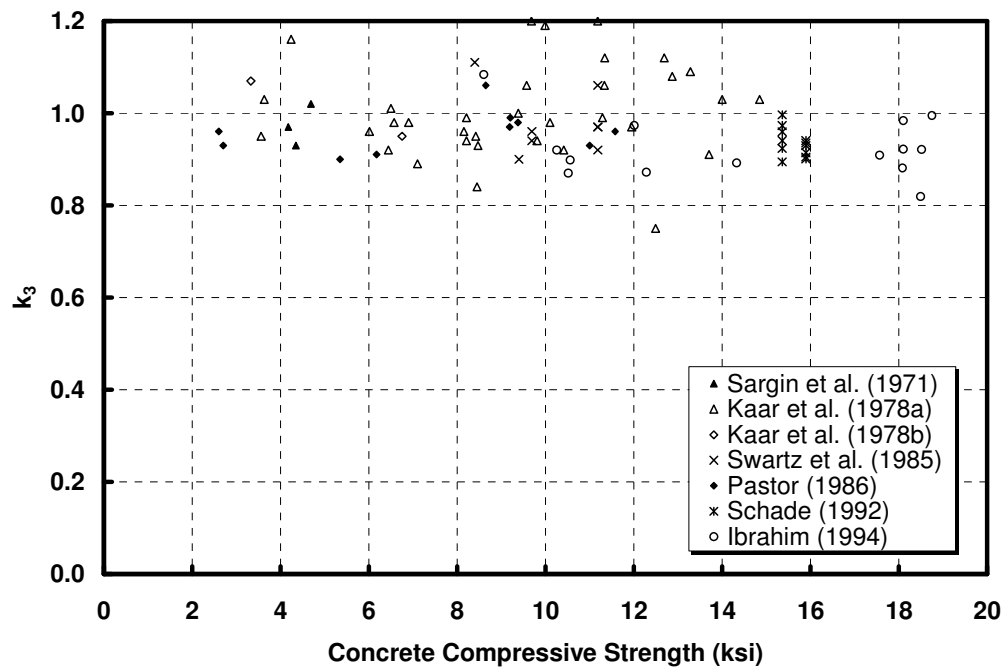


Figure 2-18 – k_3 Parameter from Eccentric Bracket Tests

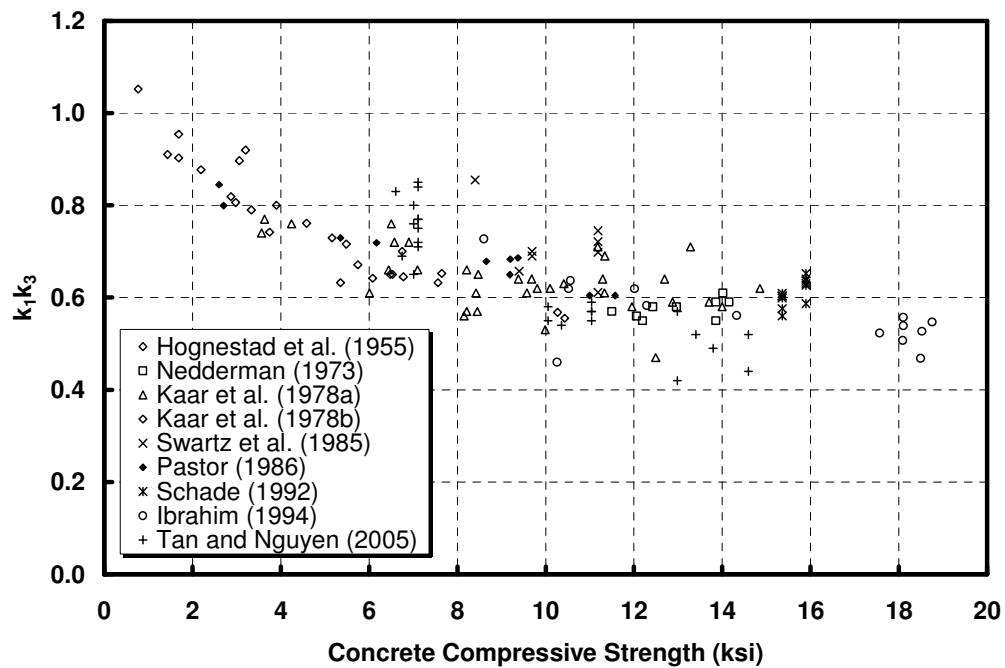


Figure 2-19 – Product of k_1 and k_3 Parameters from Eccentric Bracket Tests

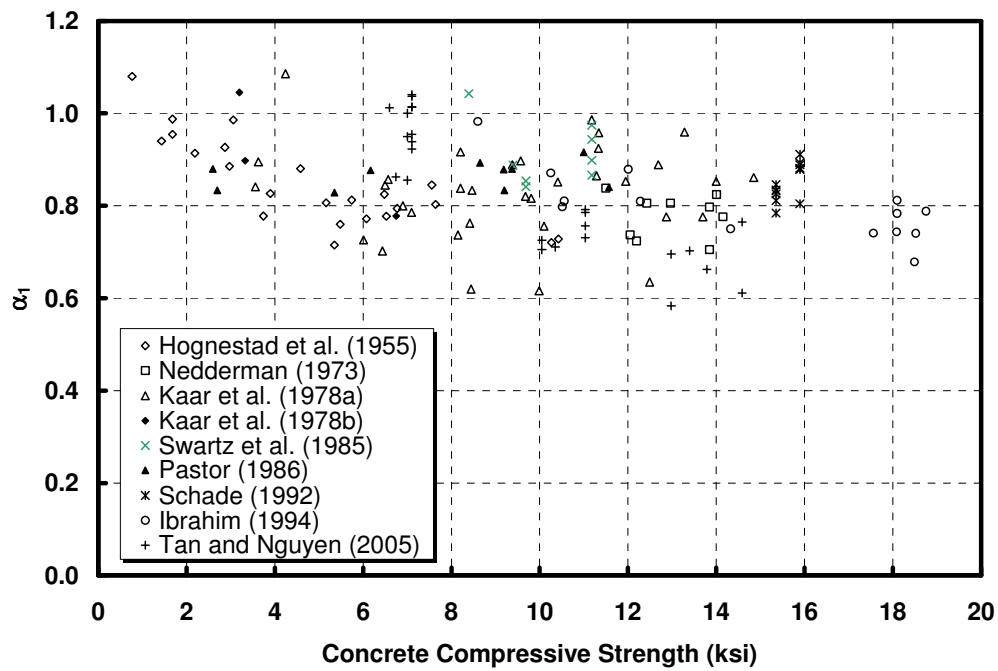


Figure 2-20 – α_1 Parameter from Eccentric Bracket Tests

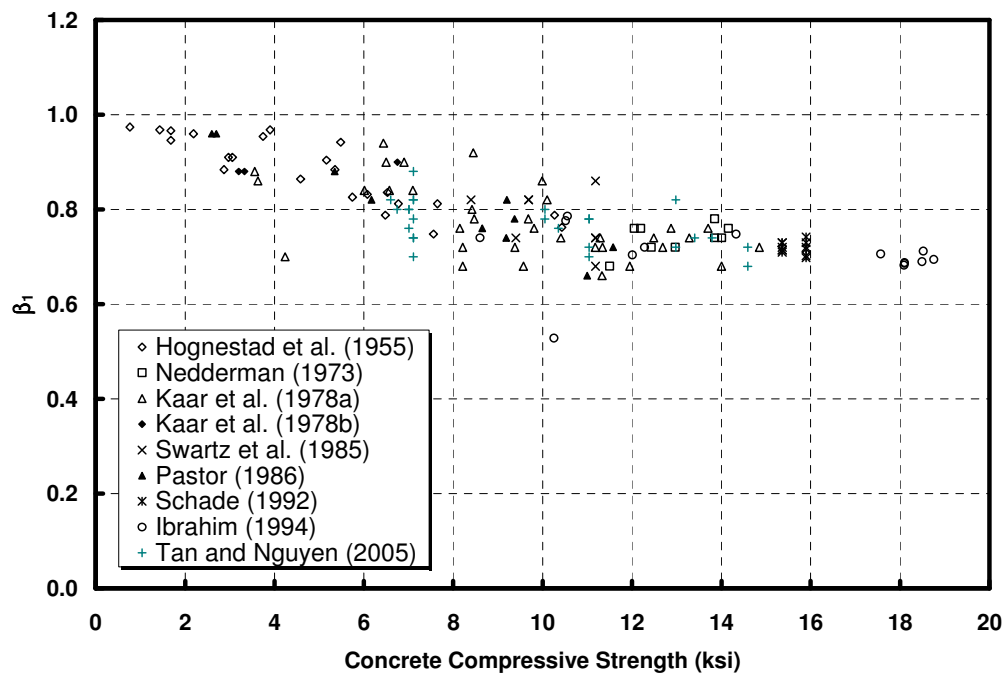


Figure 2-21 – β_1 Parameter from Eccentric Bracket Tests

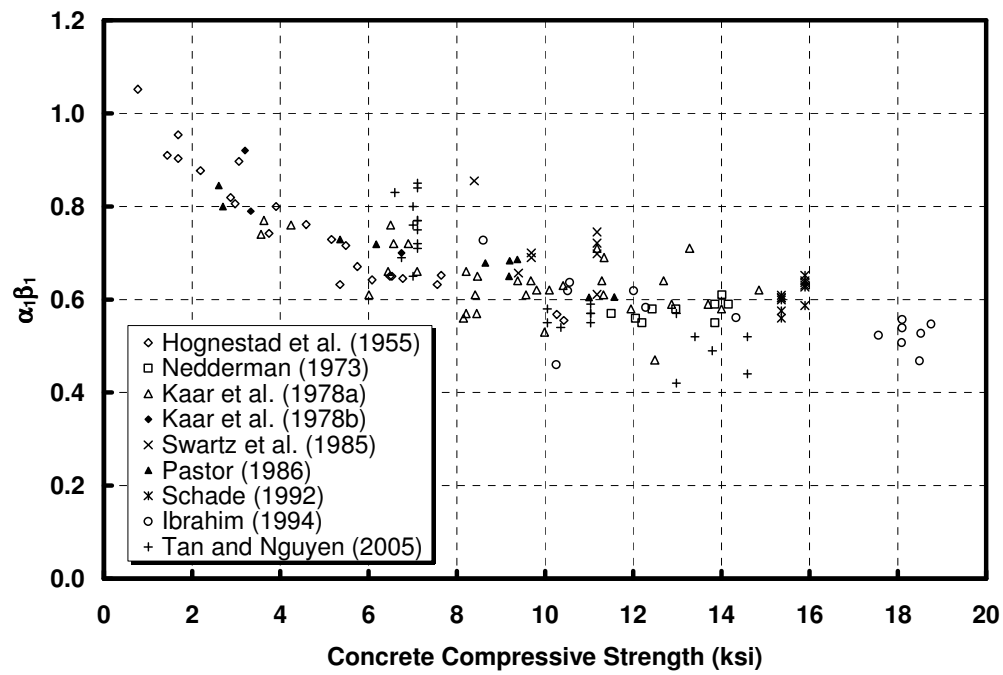


Figure 2-22 – Product of α_l and β_l Parameters from Eccentric Bracket Tests

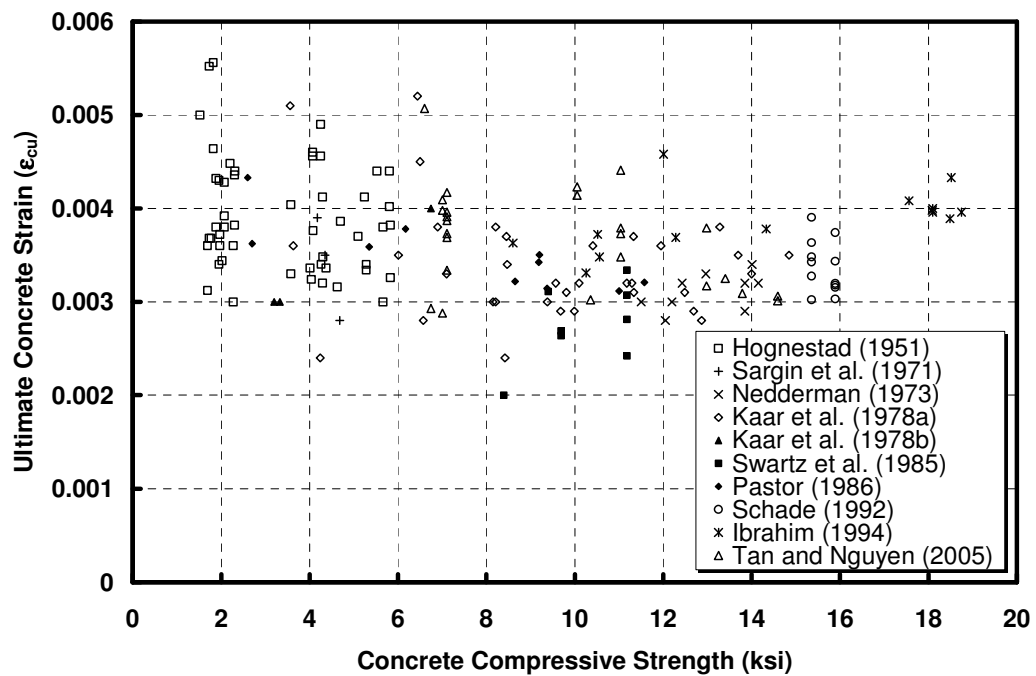


Figure 2-23 – Ultimate compressive strain of concrete from Eccentric Bracket Tests

2.2.3 Proposed Stress-Strain Models for Compression Zone of Flexural Members

The stress-strain relationships presented in this section are mainly developed using concrete cylinders. To use these relationships in either analysis or design, the factor that converts stress-strain relationship of cylinder to the compressive stress distribution in the compression zone of flexural members must be applied. This factor, known as k_3 , should be applied to the stress-strain relationships explained in this section unless it is present in the equation.

2.2.3.1 Jensen (1943)

Jensen (1943) performed flexural tests to evaluate the ultimate strength of beams. He proposed a trapezoidal stress-strain relationship which consists of two linear parts, one presenting elastic behavior and the other representing the plastic behavior. This relationship was also justified by other beam tests in the literature. The following equations were proposed for the stress-strain relationship:

$$f_c = \frac{f'_c \epsilon_c}{\epsilon_o} \text{ for } \epsilon_c < \epsilon_o \quad \text{Equation 2-8}$$

$$f_c = f'_c \text{ for } \epsilon_o < \epsilon_c < \epsilon_{cu} \quad \text{Equation 2-9}$$

where f_c is the stress in general, ϵ_c is the strain in general, f'_c is the cylinder compressive strength of concrete. The strain corresponding to the peak stress, ϵ_o , and the ultimate strain ϵ_{cu} was expressed as:

$$\epsilon_o = \frac{f'_c}{E_c} \quad \text{Equation 2-10}$$

$$\epsilon_{cu} = \frac{f'_c}{(1-\beta)E_c} + \epsilon_o \quad \text{Equation 2-11}$$

where β is the plasticity ratio and E_c is the modulus of elasticity of concrete. These can be calculated as:

$$\beta = \frac{1}{1 + \left(\frac{f'_c (ksi)}{4} \right)} \quad \text{Equation 2-12}$$

$$n = \frac{E_s}{E_c} = 5 + \frac{10}{f'_c (ksi)} \quad \text{Equation 2-13}$$

where n is the modular ratio and E_s is the modulus of elasticity of steel. It was suggested to use the modulus of elasticity as:

$$E_s = 30000 \text{ ksi}.$$

2.2.3.2 Hognestad (1951)

The stress-strain relationship proposed by Hognestad (1951) had a rising parabola and a descending linear branch after the peak stress. It was indicated that the initial curved part of the stress-strain relationship was fairly similar to the relation in direct compression. The slope of the descending branch was chosen in order to give the best statistical agreement between calculated and observed strengths in 120 tests. The relationship was presented as follows:

$$f_c = k_3 f'_c \left[\frac{2\varepsilon}{\varepsilon_o} - \left(\frac{\varepsilon}{\varepsilon_o} \right)^2 \right] \text{ for } \varepsilon_c < \varepsilon_o \quad \text{Equation 2-14}$$

where

$$f_c = k_3 f'_c - \left[(0.1275 f'_c) \left(\frac{\varepsilon - \varepsilon_o}{\varepsilon_u - \varepsilon_o} \right) \right] \text{ for } \varepsilon_o < \varepsilon_c < \varepsilon_u \quad \text{Equation 2-15}$$

where f_c is the stress in general, k_3 is the ratio of the maximum stress to the cylinder strength which is suggested to be taken as 0.85, f'_c is the cylinder compressive strength of concrete and ε is the strain in general. The strain corresponding to the peak stress (ε_o) and the ultimate strain (ε_u) is defined by:

$$\varepsilon_o = \frac{2k_3 f'_c (psi)}{E_c (psi)} \quad \text{Equation 2-16}$$

$$\varepsilon_u = 0.0038 \quad \text{Equation 2-17}$$

where the modulus of elasticity can be calculated as follows:

$$E_c (psi) = 1800000 + 460 f'_c (psi) . \quad \text{Equation 2-18}$$

2.2.3.3 Sargin and Handa (1969)

Sargin and Handa (1969) suggested a second degree rational equation for stress-strain curve of concrete as follows:

$$f_c = k_3 f'_c \frac{Ax + (D-1)x^2}{1 + (A-2)x + Dx^2} \quad \text{Equation 2-19}$$

where f_c is the stress in general, k_3 is the maximum stress ratio, f'_c is the cylinder compressive strength of concrete and D is a parameter which effects mainly the slope of the descending branch. The non-dimensionless strain (x) and the non-dimensional form of initial modulus (A) was defined by:

$$A = E_c \varepsilon_o / k_3 f'_c \quad \text{Equation 2-20}$$

$$x = \varepsilon / \varepsilon_o \quad \text{Equation 2-21}$$

where E_c is the modulus of elasticity of concrete, ε_o is the strain at maximum stress and ε is the stain in general. Influence of important factors such as concrete compressive strength, lateral reinforcement, core-gross area ratio, creep and strain gradient on concrete behavior was considered in the evaluation of these parameters as follows:

$$E_c = 72000 \sqrt{f'_c (psi)} \quad \text{Equation 2-22}$$

$$\varepsilon_o = 0.0021 \quad \text{Equation 2-23}$$

$$D = 0.65 - 5 \times 10^{-5} f'_c (psi) \quad \text{Equation 2-24}$$

The proposed stress-strain relationship was verified with the tests results performed by the authors and the available research data in the literature.

2.2.3.4 Popovics (1973)

Popovics (1973) proposed an expression that presented the shape of the rising branch of stress-strain relationship of a concrete cylinder. It was indicated that this formula was valid only for standard concrete specimens with a height-width ratio not less than two and when uniaxial compressive load is a short term load which was applied at a rate that produces constant rate strain in the specimen. The equation is presented as follows:

$$f_c = f'_c \frac{\epsilon_c}{\epsilon_{co}} \frac{n}{n-1 + (\epsilon_c/\epsilon_{co})^n} \quad \text{Equation 2-25}$$

where f_c is the stress in general, f'_c is the cylinder compressive strength of concrete, ϵ_c is the strain in general, ϵ_{co} is the strain at maximum stress. The coefficient n was expressed as follows:

$$n_{concrete} = 0.4 \times 10^{-3} f'_c (psi) + 1.0 \quad \text{Equation 2-26}$$

2.2.3.5 Wang et al. (1978a, b)

Wang et al. (1978a, b) used the same form of equations proposed by Sargin and Handa (1969). However, instead of using one set of coefficients to generate the complete curve, two sets of coefficients were used, one for the ascending branch and other one for the descending branch. These coefficients were obtained from the relevant boundary conditions assigned to each part of the curve. The stress-strain relationship used was:

$$f_c = f'_c \left[\frac{A \left(\frac{\epsilon}{\epsilon_o} \right) + B \left(\frac{\epsilon}{\epsilon_o} \right)^2}{1 + C \left(\frac{\epsilon}{\epsilon_o} \right) + D \left(\frac{\epsilon}{\epsilon_o} \right)^2} \right] \quad \text{Equation 2-27}$$

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where A , B , C and D are the constants to be determined, f_c is the stress in general, f'_c is the cylinder strength of concrete, ε is the strain in general and ε_o is the strain at maximum stress. For different concrete compressive strengths, these constants shown in Table 2-2 are derived by using the experimental stress-strain relationship.

Table 2-2 – Stress-Strain Relationship Constants by Wang et al. (1978a,b)

f'_c (ksi)	ε_o	Ascending				Descending			
		A	B	C	D	A	B	C	D
3	0.00267	1.5970	-0.3520	-0.4030	0.6480	-1.2758	1.4795	-3.2758	2.4795
4	0.0028	1.4434	-0.6425	-0.5566	0.3574	0.1153	0.3064	-1.8847	1.3064
5	0.00292	1.3648	-0.7590	-0.6352	0.2420	0.3264	0.0480	-1.6736	1.0480
6	0.00305	1.3237	-0.8095	-0.6763	0.1905	0.3659	-0.0507	-1.6341	0.9493
7	0.00317	1.3040	-0.8320	-0.6960	0.1680	0.3576	-0.0956	-1.6424	0.9415
8	0.00330	1.2977	-0.8388	-0.7023	0.1612	0.3326	-0.1163	-1.6674	0.8837
9	0.00342	1.3004	-0.8360	-0.6996	0.1640	0.3019	-0.1244	-1.6981	0.8756
10	0.00355	1.3092	-0.8261	-0.6908	0.1739	0.2697	-0.1253	-1.7303	0.8747
11	0.00367	1.3227	-0.8107	-0.6773	0.1893	0.2380	-0.1218	-1.7620	0.8782
12	0.00380	1.3395	-0.7904	-0.6605	0.2096	0.2078	-0.1154	-1.7922	0.8846
13	0.00392	1.3590	-0.7657	-0.6410	0.2343	0.1796	-0.1071	-1.8204	0.8929

2.2.3.6 Carreira and Chu (1985)

The model proposed by Carreira and Chu (1985) was a general form of serpentine curve as presented by the following:

$$f_c = f'_c \left[\frac{\beta \left(\frac{\varepsilon}{\varepsilon_o} \right)}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_o} \right)^\beta} \right] \quad \text{Equation 2-28}$$

in which f_c is the stress in general, f'_c is the cylinder compressive strength of concrete, ε is the strain in general, ε_o is the strain at maximum stress, β is a material parameter that depended on the shape of the stress-strain diagram. ε_o and β were given by:

$$\varepsilon_o = (4.88 f'_c (\text{ksi}) + 168) \times 10^{-5} \quad \text{Equation 2-29}$$

$$\beta = \frac{1}{1 - (f'_c / \epsilon_o E_c)} \quad \text{Equation 2-30}$$

where modulus of elasticity of concrete (E_c) can be calculated by using the following equation:

$$E_c = 33w^{1.5} \sqrt{f'_c \text{ (psi)}} \quad \text{Equation 2-31}$$

where w is the unit weight of concrete in lb/ft^3 .

2.2.3.7 Thorenfeldt et al. (1987)

Thorenfeldt et al. (1987) reported that while expression by Popovics (1973) describes well the rising branch of the stress-strain curve, it did not drop fast enough after the peak for HSC. To increase the post-peak decay, they suggested the addition of the factor, k . The proposed equation was as follows:

$$f_c = f'_c \frac{\epsilon_c}{\epsilon_{co}} \frac{n}{n-1 + (\epsilon_c / \epsilon_{co})^{nk}} \quad \text{Equation 2-32}$$

where f_c is the stress in general, f'_c is the cylinder compressive strength of concrete, ϵ_c is the strain in general, ϵ_{co} is the strain at maximum stress and n is the coefficient for concrete behavior. The factor k was defined by:

$$\begin{aligned} k &= 1 \quad \text{when} \quad \epsilon_c / \epsilon_{co} \leq 1 \\ k &> 1 \quad \text{when} \quad \epsilon_c / \epsilon_{co} > 1 \end{aligned} \quad \text{Equation 2-33}$$

Best correlation with the test results was observed when $1 < k < f_c / 20$.

Other parameters can be calculated as follows:

$$E_{cn} = \left(\frac{f'_c}{0.145} \right)^{0.3} \times 10^4 \quad \text{Equation 2-34}$$

$$m = \frac{\varepsilon_o}{\varepsilon_n} \quad \text{Equation 2-35}$$

$$\varepsilon_n = \frac{f'_c}{E_{cn}} \quad \text{Equation 2-36}$$

$$\varepsilon_o = (0.004f'_c + 1.9) \times 10^{-3} \geq (0.015f'_c + 1.17) \times 10^{-3} \quad \text{Equation 2-37}$$

$$\varepsilon_{cu} = (2.5m - 1.5)\varepsilon_n \quad \text{Equation 2-38}$$

2.2.3.8 CEB-FIB Model Code (1990)

The stress-strain relationship of concrete in compression for short-term loading was approximated as follows:

$$f_c = f'_c \left[\frac{\left(\frac{E_{it}}{E_o} \right) \left(\frac{\varepsilon}{0.0022} \right) - \left(\frac{\varepsilon}{0.0022} \right)^2}{1 + \left(\frac{E_{it}}{E_o} - 2 \right) \left(\frac{\varepsilon}{0.0022} \right)} \right] \quad \text{Equation 2-39}$$

where f_c is the concrete stress, f'_c is the cylinder compressive strength of concrete, ε is the concrete strain, E_{it} is the initial tangent elastic modulus, E_o is the secant elastic modulus from the origin to the peak compressive stress. This approximation is valid for concrete strain ranges between 0 and $\varepsilon_{\max}$ where $\varepsilon_{\max}$ is the concrete strain when concrete stress is equal to $0.5f'_c$ on the descending part of the stress-strain curve. The elastic modulus and $\varepsilon_{\max}$ can be calculated from:

$$E_{it} = 335549(f'_c \text{ (psi)})^{0.3} \quad \text{Equation 2-40}$$

$$E_o = \frac{f'_c}{0.0022} \quad \text{Equation 2-41}$$

and

$$\varepsilon_{\max} = \varepsilon_o \left[\frac{1}{2} \left(\frac{1}{2} \frac{E_{it}}{E_o} + 1 \right) + \sqrt{\frac{1}{4} \left(\frac{1}{2} \frac{E_{it}}{E_o} + 1 \right)^2 - \frac{1}{2}} \right] \quad \text{Equation 2-42}$$

Note that E_{it} defined by **Dahl (1992)** was used in the equations which enabled the stress-strain model to be used for concrete compressive strengths up to 16.8 ksi.

For strains $\epsilon > \epsilon_{\max}$, the descending branch of the stress-strain relationship can be described as:

$$f_c = \frac{f'_c}{\left[\frac{1}{(\epsilon_{\max}/\epsilon_o)^2} \xi - \frac{2}{(\epsilon_{\max}/\epsilon_o)^2} \right]^{(e/\epsilon_o)}} + \left[\frac{4}{(\epsilon_{\max}/\epsilon_o)^2} - \xi \right] \frac{\epsilon}{\epsilon_o} \quad \text{Equation 2-43}$$

where

$$\xi = \frac{4 \left[\left(\frac{\epsilon_{\max}}{\epsilon_o} \right)^2 \left(\frac{E_{it}}{E_o} - 2 \right) + 2 \left(\frac{\epsilon_{\max}}{\epsilon_o} \right) - \frac{E_{it}}{E_o} \right]}{\left[\left(\frac{\epsilon_{\max}}{\epsilon_o} \right) \left(\frac{E_{it}}{E_o} - 2 \right) + 1 \right]^2} \quad \text{Equation 2-44}$$

2.2.3.9 Muguruma et al. (1991)

Based on their experimental work, Muguruma et al. (1991) proposed a two part stress-strain curve for unconfined concrete which could be applied to concrete with a wide range of concrete compressive strengths. The first part of the curve consisted of the commonly used second degree parabola that had a zero slope at the point (f'_c, ϵ_{co}) where f'_c was the ultimate concrete compressive strength and ϵ_{co} was the concrete compressive strain at f'_c . The second part of the curve was linear till the maximum compressive strain which was 0.004.

$$f_c = E_i \epsilon_c + \frac{(f'_c - E_i \epsilon_m)}{\epsilon_m^2} \epsilon_c^2 \quad \text{when } 0 < \epsilon_c < \epsilon_m \quad \text{Equation 2-45}$$

$$f_c = \frac{f'_c}{(\epsilon_m - 0.004)} (\epsilon_c - 0.004) \quad \text{when } \epsilon_m < \epsilon_c < 0.004 \quad \text{Equation 2-46}$$

where f_c is the stress in general, ε_c is the strain in general, ε_m is the strain at maximum stress and E_i is the initial elastic modulus. E_i and ε_m are given by the following equations based on empirical testing results.

$$\varepsilon_m = 0.0013 \left(1 + \frac{f'_c (MPa)}{98.6} \right) \quad \text{Equation 2-47}$$

$$E_i (MPa) = 22700 \sqrt{f'_c (MPa)} / 19.6 \quad \text{Equation 2-48}$$

2.2.3.10 Collins and Porasz (1989)

Collins and Porasz (1989) proposed new k and n factors for the equation proposed by Thorenfeldt et al. (1987) as follows:

$$k = 0.64 + \frac{f'_c (psi)}{9000} \quad \text{Equation 2-49}$$

$$n = 0.8 + \frac{f'_c (psi)}{2500} \quad \text{Equation 2-50}$$

2.2.3.11 Hsu and Hsu (1994)

Hsu and Hsu (1994) proposed a modified form of the stress-strain relationship proposed by Carreira and Chu (1985): n was added to the original equation. Also the parameters were modified and corrected for the presented unconfined concrete. The complete stress-strain curve was presented by:

$$\frac{f_c}{f'_c} = \frac{n\beta \left(\frac{\varepsilon}{\varepsilon_o} \right)}{n\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_o} \right)^{n\beta}} \quad \text{for } 0 \leq \varepsilon < \varepsilon_{\max} \quad \text{Equation 2-51}$$

and

$$\frac{f_c}{f'_c} = \eta_d \exp \left[-k_d \left(\frac{\varepsilon}{\varepsilon_o} - \frac{\varepsilon_{\max}}{\varepsilon_o} \right)^a \right] \quad \text{for } \varepsilon_{\max} \leq \varepsilon \quad \text{Equation 2-52}$$

$$\beta = \frac{1}{1 - [f'_c / (\epsilon_o E_{it})]} \text{ for } \beta \geq 1.0 \quad \text{Equation 2-53}$$

where n is the parameter depended on the material strength, β is the parameter depended on the shape of the stress-strain diagram, f'_c is the stress in general, ϵ is the strain in general, f'_c is the peak stress of concrete, ϵ_o is the strain corresponding to the peak stress, $\epsilon_{\max}$ is the strain at $0.3f'_c$ on the descending portion of the stress-strain diagram, η_d is equal to 0.3, corresponding to $0.3f'_c$ on the descending portion of the stress-strain curve, $k_d = 0.8$, $a = 0.5$ and E_{it} is the initial tangential modulus. The parameters f'_c , ϵ_o , n , β and E_{it} can be directly obtained from the uniaxial compression tests under constant strain. The slope of the descending branch is affected by the β value, a flatter curve would yield a reduced β . A best fitting statistical analysis was performed using the experimental data to obtain the following equation for β .

$$\beta = \left(\frac{f'_c \text{ (psi)}}{9460} \right)^3 + 2.59 \quad \text{Equation 2-54}$$

The following equation for n values was proposed as a function of the compressive strength.

$$n = \begin{cases} 1 & \text{for } 0 \leq \epsilon < \epsilon_o \\ 2 & \text{for } \epsilon_o \leq \epsilon < \epsilon_{\max} \text{ if } 0 \leq f'_c < 9 \text{ ksi} \\ 3 & \text{for } \epsilon_o \leq \epsilon < \epsilon_{\max} \text{ if } 9 \text{ ksi} \leq f'_c < 11 \text{ ksi} \\ 5 & \text{for } \epsilon_o \leq \epsilon < \epsilon_{\max} \text{ if } 11 \text{ ksi} \leq f'_c < 13 \text{ ksi} \\ & \text{if } 13 \text{ ksi} \leq f'_c \end{cases} \quad \text{Equation 2-55}$$

A regression analysis was performed to obtain the equations for ϵ_o and E_{it} .

$$\epsilon_o = 8.9 \times 10^{-8} f'_c \text{ (psi)} + 2.114 \times 10^{-3} \quad \text{Equation 2-56}$$

$$E_{it} = 1.2431 \times 10^{-1} f'_c \text{ (psi)} + 3.28312 \times 10^3 \quad \text{Equation 2-57}$$

2.2.3.12 Wee et al. (1996)

A research was conducted by Wee et al. (1996) to find a simple, more accurate model to represent the complete stress-strain curve, particularly for HSC. The equation proposed by Carreira and Chu (1985) was chosen because of its simplicity. For the ascending branch of the stress-strain curve:

$$f_c = f'_c \left[\frac{\beta \left(\frac{\epsilon}{\epsilon_o} \right)}{\beta - 1 + \left(\frac{\epsilon}{\epsilon_o} \right)^\beta} \right] \quad \text{Equation 2-58}$$

and

$$\beta = \frac{1}{1 - (f'_c / \epsilon_o E_{it})} \quad \text{Equation 2-59}$$

were used. But for modeling the descending branch of the stress-strain curve, same equations were used with two correction factors, k_1 and k_2 . The factor k_1 was applied to the β values in the numerator and in the first term of the demoninator, while k_2 was applied to β in the exponent of the last term of the denominator. The resulting expression for the descending portion of the stress-strain curve becomes:

$$f_c = f'_c \left[\frac{k_1 \beta \left(\frac{\epsilon}{\epsilon_o} \right)}{k_1 \beta - 1 + \left(\frac{\epsilon}{\epsilon_o} \right)^{k_2 \beta}} \right] \quad \text{Equation 2-60}$$

An analysis carried out using the set of data that gathered for concrete compressive strengths ranging from 7.25 ksi to 17.4 yielded:

$$k_1 = \left(\frac{7252}{f'_c (psi)} \right)^{3.0} \quad \text{Equation 2-61}$$

and

$$k_2 = \left(\frac{7252}{f'_c (psi)} \right)^{1.3} \quad \text{Equation 2-62}$$

For concrete compressive strengths below 7.25 ksi., k_1 and k_2 should be taken as unity. The equation for E_{it} and ϵ_o were obtained by regression analysis of the experimental data and were presented as follows:

$$E_{it} = 281571 (f'_c (psi))^{(1/3)} \quad \text{Equation 2-63}$$

$$\epsilon_o = 0.000225(f'_c (psi))^{(1/4)} \quad \text{Equation 2-64}$$

2.2.3.13 Van Gysel and Taerwe (1996)

Van Gysel and Taerwe (1996) proposed a relationship that simulates the behavior of HSC. The increase in the peak strain and the steepness of the softening branch at higher strength levels were also included in the formulation. Although the proposed relationship was largely based on the CEB Model (1990), the descending branch of the stress-strain relationship of HSC was predicted better. For the ascending branch, the same stress-strain relationship specified by CEB Model (1990) was used by changing the strain at peak stress with the following equation:

$$\frac{f_c}{f'_c} = \frac{\frac{E_{co}}{E_{c1}} \frac{\epsilon_c}{\epsilon_{c1}} - \left(\frac{\epsilon_c}{\epsilon_{c1}} \right)^2}{1 + \left(\frac{E_{co}}{E_{c1}} - 2 \right) \frac{\epsilon_c}{\epsilon_{c1}}} \quad \text{for } \epsilon_c < \epsilon_{c1}$$

with

$$\epsilon_{c1} = 0.00015 f'_c{}^{0.31} (psi) \quad \text{for } f'_c > 5831 \text{ psi}$$

$$\epsilon_{c1} = 0.0022 \quad \text{for } f'_c \leq 5831 \text{ psi}$$

where f_c is the concrete stress, ϵ_c is the concrete strain, f'_c is the cylinder compressive strength of concrete, E_{co} is the initial tangent modulus of elasticity, E_{cl} is the secant elastic modulus from the origin to the peak compressive stress f'_c and ϵ_{cl} is the concrete strain at peak stress.

For the descending branch, the following relationship was proposed which is based on the formulation originally proposed by Sargin and Handa (1969)

$$\frac{f_c}{f'_c} = \frac{1}{1 + \left(\frac{\frac{\epsilon_c}{\epsilon_{cl}} - 1}{\epsilon_{cl} + \frac{t}{\epsilon_{cl}}} - 2 \right)}$$

with corresponding t values are presented in Table 2-3.

Table 2-3 – Values of the Parameter t for Different Concrete Compressive Strengths

f'_c (ksi)	7252	8702	10153	11603	13053	14504
t (%)	0.807	0.579	0.388	0.221	0.07	0.015

2.2.3.14 Attard and Setunge (1996)

Attard and Setunge (1996) used the non-dimensional formulation suggested by Sargin and Handa (1969) to define the stress-strain curve of concrete for the unconfined uniaxial case for concrete compressive strengths between 2.9 to 18.855 ksi. It was indicated that in uniaxial compression, the failure load was a combination of shear failure and tensile splitting. To define the descending curve in uniaxial compression, one point was needed on the descending curve. The point of inflection suggested by Wang et al. was used. On this basis, the full uniaxial stress-strain relationship was established and the following equation was derived:

$$\frac{f_c}{f'_c} = \frac{\left(\frac{f_{ic}}{\epsilon_{co}\epsilon_{ic}} \frac{(\epsilon_{ic} - \epsilon_{co})^2}{f'_c - f_{ic}} \right) \left(\frac{\epsilon_c}{\epsilon_{co}} \right)}{1 + \left(\frac{f_{ic}}{\epsilon_{co}\epsilon_{ic}} \frac{(\epsilon_{ic} - \epsilon_{co})^2}{f'_c - f_{ic}} - 2 \right) \left(\frac{\epsilon_c}{\epsilon_{co}} \right) + \left(\frac{\epsilon_c}{\epsilon_{co}} \right)^2} \quad \text{Equation 2-65}$$

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where f_c is the stress at strain ϵ_c , f'_c is the cylinder compressive strength of concrete, f_{ic} is the stress at point of inflection, ϵ_{ic} is the strain at point of inflection and ϵ_{co} is the strain at peak.

To define the modulus of elasticity, E_c , the formula used in ACI 318 was used for NSC and the formula used in ACI 363 was used for HSC which is valid up to 12 ksi.

$$E_c = w_c^{1.5} 33 \sqrt{f'_c \text{ (psi)}} \text{ for NSC} \quad \text{Equation 2-66}$$

$$E_c = \left(40000 \sqrt{f'_c \text{ (psi)}} + 1.0 \times 10^6 \right) \left(\frac{w_c}{145} \right)^{1.5} \text{ for HSC} \quad \text{Equation 2-67}$$

where w_c is the unit weight of concrete in lbs/ft³.

The equation proposed by Setunge was used to define the strain at peak stress.

$$\epsilon_{co} = \frac{f'_c}{E_c} \frac{14.78}{\sqrt[4]{f'_c \text{ (psi)}}} \text{ for crushed aggregates} \quad \text{Equation 2-68}$$

$$\epsilon_{co} = \frac{f'_c}{E_c} \frac{13.12}{\sqrt[4]{f'_c \text{ (psi)}}} \text{ for gravel aggregates} \quad \text{Equation 2-69}$$

Based on the analytical expressions in Mander et al. (1988) and Collins (1992) and the experimental results of Dahl (1992), the following approximate expressions for the uniaxial inflection point were proposed:

$$\frac{\epsilon_{ic}}{\epsilon_{co}} = 4 - 0.3 \ln(f'_c \text{ (psi)}) \quad \text{Equation 2-70}$$

$$\frac{f_{ic}}{f'_c} = 2.256 - 0.17 \ln(f'_c \text{ (psi)}) \quad \text{Equation 2-71}$$

2.2.3.15 Oztekin et al. (2003)

Oztekin et al. (2003) used the stress-strain relationship by Hognestad et al (1951) to obtain a modified model which was applicable to HSC. It was indicated that, it was very difficult to simulate the falling branch of HSC due to its sudden failure. Ultimate strain,

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ε_{cu} , was assumed to be equal to the strain in the maximum stress, ε_{co} . On this basis, they proposed the following equation:

$$f_c = f'_c \left(k \frac{\varepsilon_c}{\varepsilon_{cu}} - (k-1) \left(\frac{\varepsilon_c}{\varepsilon_{cu}} \right)^2 \right) \quad \text{Equation 2-72}$$

where k was a modification parameter described as follows:

$$k = 2 - \frac{f'_c \text{ (psi)} - 5800}{10153} \text{ for } 8700 \text{ psi} \leq f'_c \leq 13633 \text{ psi} \quad \text{Equation 2-73}$$

ε_{cu} , proposed in **CEB Model (1990)**, was modified according to experimental results as follows:

$$\varepsilon_{cu} = [2.2 + 1.034 \times 10^{-4} (f'_c - 5800)] \times 10^{-3} \text{ for } 8700 \text{ psi} \leq f'_c \leq 13633 \text{ psi} \quad \text{Equation 2-74}$$

2.2.4 Proposed Rectangular Stress Block Parameters

The definitions for rectangular stress block parameters for concrete is presented in Section 2.2.1. In this section, the proposed relationships and equations by many researchers are illustrated, starting from the best known one, Charles Whitney.

2.2.4.1 Whitney (1937)

Whitney (1937) proposed a rectangular stress block to replace the existing allowable stress design theory in design codes. This stress block was verified by using the experimental data on beams tested by other researchers at that time. The width of the proposed rectangular stress block was $0.85f'_c$. This assumption was based on the tests on actual strength of columns performed by Richart et al. (1931a, 1931b, 1931c and 1932) and permitted a consistent transition in the moment-axial load diagram from pure axial to pure moment resistance. The height of the rectangular stress block (a) was changing according to the reinforcement ratio of the beam. For over-reinforced sections, a/d was 0.537 and for under-reinforced sections a/d was changing from 0 to 0.537 as a quadratic curve for no reinforcement and balanced ratio, respectively.

2.2.4.2 Mattock et al. (1961)

The proposed parameters by Mattock et al. (1961) for rectangular stress block are currently being used in ACI 318 and AASHTO LRFD Bridge Design Specifications (2004) for concrete compressive strengths less than 10 ksi. The depth of the stress block was 0.85 for all the concrete compressive strengths and the height was equal to 0.85 for concrete compressive strengths up to 4 ksi, and thereafter was reduced by 0.05 for each 1 ksi of strength in excess of 4 ksi but not less than 0.65. The ultimate strain of concrete was assumed to be 0.003 for all target strengths. No upper concrete compressive strength limit was specified which later caused some inconsistencies between the predicted and experimental values.

$$\alpha_1 = 0.85 \quad \text{Equation 2-75}$$

$$\beta_1 = \begin{cases} 0.85 & \text{for } f'_c \leq 4 \\ 0.85 - 0.05(f'_c - 4) & \text{for } f'_c > 4 \end{cases} \quad \text{Equation 2-76}$$

$$\epsilon_{cu} = 0.003$$

Equation 2-77

where f'_c is in ksi.

2.2.4.3 Zia (1983)

Zia (1983) used the idealized trapezoidal stress-strain relationship involving the concept of plasticity proposed by Jensen (1943) to provide the rectangular stress block parameters for HSC.

$$\alpha_1 = \frac{3(1+\beta)^2}{4(1+\beta+\beta^2)}$$

Equation 2-78

$$\beta_1 = \frac{2(1+\beta+\beta^2)}{3(1+\beta)}$$

Equation 2-79

where

$$\beta = 1 - \frac{f'_c}{E_c \epsilon_{cu}}$$

Equation 2-80

It was suggested to use the current ACI Building Code expressions at that time for modulus of elasticity of concrete and the ultimate strain expressed as;

$$E_c = 57000\sqrt{f'_c}$$

Equation 2-81

$$\epsilon_{cu} = 0.003$$

Equation 2-82

The rectangular stress block parameters shown in Table 2-4 were derived based on the proposed equations.

Table 2-4 – Rectangular Stress Block Parameters Proposed by Zia (1983)

f'_c (ksi)	α_1	β_1	ϵ_{cu}
5	0.98	0.81	0.003
10	0.95	0.74	
15	0.91	0.71	

2.2.4.4 Li (1993)

For HSC, Li (1993) stated that the actual shape of the compressive stress distribution for unconfined concrete was similar to the shape of a triangle with peak stress occurring at a strain of about 0.003. Li (1993) assumed conservatively that the maximum stress in the actual triangular stress block of HSC is f'_c (that is, $k_3 = 1.0$), for the triangular stress block and the equivalent rectangular stress block to have the same magnitude and position of resultant concrete compressive force requires $\alpha_1 f'_c a = f'_c c/2$ and $a/2 = c/3$, from which $\alpha_1 = 0.75$. Hence for HSC the equivalent rectangular stress block had a mean stress about $0.75 f'_c$ and $\beta_1 = a/c = 0.67$. On this basis, the following equivalent rectangular compressive stress block for unconfined concrete was suggested.

$$\alpha_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 8 \\ 0.85 - 0.02758(f'_c - 8) \geq 0.75 \text{ for } f'_c > 8 \end{cases} \quad \text{Equation 2-83}$$

$$\beta_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 4.35 \\ 0.85 - 0.05516(f'_c - 4.35) \geq 0.65 \text{ for } f'_c > 4.35 \end{cases} \quad \text{Equation 2-84}$$

$$\epsilon_{cu} = 0.003 \quad \text{Equation 2-85}$$

where f'_c is in ksi.

2.2.4.5 Azizinamini et al. (1994)

Azizimanini et al. (1994) stated that the typical stress-strain relationships in compression for HSC could be characterized by an ascending portion that is primarily linear with a maximum strength achieved at an axial strain between approximately 0.0024 and 0.003. Considering this, the researchers concluded that using a triangular compressive stress block was more appropriate in calculating the flexural capacity of the columns with concrete compressive strength exceeding 10 ksi. The maximum compressive stress was assumed to be $0.85 f'_c$ at axial compressive strain of 0.003. Considering the equilibrium of horizontal forces and moment equilibrium, it was shown that the equivalent rectangular compressive block would have the following properties: intensity of the compressive stress would be $0.63 f'_c$ rather than $0.85 f'_c$; and the depth of the rectangular

compression block would be 0.67 times the depth of the neutral axis. Considering these, the following rectangular stress block parameters were recommended.

$$\alpha_1 = \begin{cases} 0.85 & \text{for } f'_c \leq 10 \\ 0.85 - 0.05(f'_c - 10) \geq 0.60 & \text{for } f'_c > 10 \end{cases} \quad \text{Equation 2-86}$$

$$\beta_1 = \begin{cases} 0.85 & \text{for } f'_c \leq 4.35 \text{ KSI} \\ 0.85 - 0.05516(f'_c - 4.35) \geq 0.65 & \text{for } f'_c > 4.35 \text{ KSI} \end{cases} \quad \text{Equation 2-87}$$

$$\varepsilon_{cu} = 0.003 \quad \text{Equation 2-88}$$

where f'_c is in ksi.

2.2.4.6 Ibrahim (1994)

Ibrahim (1994) proposed new rectangular stress block parameters for both NSC and HSC. He compared the experimental data to date with the ACI code. It was found that the ACI value for β_1 fell below the experimental showing that it was too small. This indicated that the internal lever arm is too big and the moment capacity would be overestimated. A new equation was proposed for β_1 which was passing through the center of the experimental data. On the other hand, α_1 parameter was derived to provide a conservative lower bound for the experimental data of k_3 obtained from concentrically loaded columns. It was also indicated that the proposed equation should provide conservative design for eccentric sections when combined with the parameter β_1 . Based on these, the following rectangular stress block parameters were proposed. For concrete compressive strength greater than 14.5 ksi, α_1 and β_1 were both taken constant and equal to 0.725 and 0.7 respectively which represent a stress distribution very close to triangular.

$$\alpha_1 = 0.85 - \frac{f'_c}{116} \geq 0.725 \quad \text{Equation 2-89}$$

$$\beta_1 = 0.95 - \frac{f'_c}{58} \geq 0.70 \quad \text{Equation 2-90}$$

$$\varepsilon_{cu} = 0.003 \quad \text{Equation 2-91}$$

where f'_c is in ksi.

2.2.4.7 Pendyala and Mendis (1998)

Pendyala and Mendis (1998) stated that at 11.6 ksi the stress-strain relationship for HSC was still curvilinear and tending to be linear at 14.5 ksi. Hence the assumption of linearity at 11.6 ksi was found to be unduly conservative. Pendyala and Mendis (1998) concluded that none of the rectangular stress blocks developed to date was applicable to extend the validity of design code to 14.5 ksi and did not provide continuity of values for NSC and HSC. On this basis, new stress block parameters were proposed below for concretes with $8.267 \text{ ksi} < f'_c < 14.503 \text{ ksi}$.

$$\alpha_1 = 0.85 - 0.01724(f'_c - 8.267) \text{ for } 8.267 \leq f'_c \leq 14.503 \quad \text{Equation 2-92}$$

$$\beta_1 = 0.65 - 0.00862(f'_c - 8.267) \text{ for } 8.267 \leq f'_c \leq 14.503 \quad \text{Equation 2-93}$$

$$\epsilon_{cu} = 0.003 \quad \text{Equation 2-94}$$

where f'_c is in ksi.

2.2.4.8 Attard and Stewart (1998)

Attard and Stewart (1998) proposed new rectangular stress block parameters based on mean results of probabilistic analysis of the tests available in the literature using a stress-strain relationship for HSC. The proposed parameters included estimates of variability and distribution of the input properties. A sensitivity analysis was also carried out to ascertain the effect of parameter uncertainty. It was indicated that for a ductile singly reinforced rectangular section, the ultimate moment capacity was relatively insensitive to the stress block model. However, estimates of the ductility level at both ultimate and column capacity in primary compression failure was significantly affected by the choice of the stress block parameters. Based on these, the following parameters were proposed for concrete with $2.9 \text{ ksi} < f'_c < 17.4 \text{ ksi}$.

$$\alpha_1 = \left\{ \begin{array}{l} 1.2932 \left(\frac{f'_c}{0.145} \right)^{-0.0998} \geq 0.71 \text{ for Dogbone Tests} \\ 0.6470 \left(\frac{f'_c}{0.145} \right)^{0.0324} \geq 0.58 \text{ for Sustained Load Tests} \end{array} \right\} \quad \text{Equation 2-95}$$

$$\beta_1 = 1.0948 \left(\frac{f'_c}{0.145} \right)^{-0.091} \geq 0.67 \quad \text{Equation 2-96}$$

$$\varepsilon_{cu} = 0.003 \quad \text{Equation 2-97}$$

where f'_c is in ksi.

2.2.4.9 Rangan (1999)

Rangan (1999) proposed to use the following equivalent rectangular stress block parameters for grades of concrete with compressive strength in the range of 2.9 to 14.5 ksi.

$$\alpha_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 8 \\ 0.85 - 0.02758(f'_c - 8) \geq 0.75 \text{ for } f'_c > 8 \end{cases} \quad \text{Equation 2-98}$$

$$\beta_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 4.35 \\ 0.85 - 0.05516(f'_c - 4.35) \geq 0.65 \text{ for } f'_c > 4.35 \end{cases} \quad \text{Equation 2-99}$$

$$\varepsilon_{cu} = 0.003 \quad \text{Equation 2-100}$$

where f'_c is in ksi.

The author specified that equivalent stress block parameters were valid up to the point when the depth of neutral axis was equal to the depth of extreme layer of tensile steel measured from the compression face.

2.2.4.10 Bae and Bayrak (2003)

Bae and Bayrak (2003) modeled the stress-strain response of unconfined HSC using the suggestion of Popovics (1973) which was then modified by Thornfeldt et al. (1987) and then by Collins and Porasz (1989). Bae and Bayrak (2003) stated that the model proposed was applicable to a wide range of concrete compressive strengths. For concrete compressive strengths up to 16.3 ksi, the proposed model offered a very good approximation of the experimentally measured behavior of HSC cylinders tested by the author. In deriving the stress block parameters, the maximum reliable strain was assumed as the assumed cover spalling strain of 0.0025. Based on these, the following stress block parameters were proposed.

$$\alpha_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 10.2 \\ 0.85 - 0.02758(f'_c - 10.2) \geq 0.67 \text{ for } f'_c > 10.2 \end{cases} \quad \text{Equation 2-101}$$

$$\beta_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 4.35 \\ 0.85 - 0.02758(f'_c - 4.35) \geq 0.67 \text{ for } f'_c > 4.35 \end{cases} \quad \text{Equation 2-102}$$

$$\epsilon_{cu} = \begin{cases} 0.003 \text{ for } f'_c \leq 8 \\ 0.0025 \text{ for } f'_c > 8 \end{cases} \quad \text{Equation 2-103}$$

where f'_c is in ksi.

2.2.4.11 Sun et al. (2003)

Sun et al. (2003) proposed a rectangular stress block whose results were significantly closer to that of Popovics (1973) equation on stress-strain curve for concrete. When maximum steel reinforcement was calculated with a strain of 0.005 for the proposed model, it was seen that the proposed model was more conservative than the design codes. On this basis, the proposed model is described as follows:

$$\alpha_1 = 0.85 \quad \text{Equation 2-104}$$

$$\beta_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 2.94 \\ \frac{28}{30 + f'_c} \geq 0.70 \text{ for } f'_c > 2.94 \end{cases} \quad \text{Equation 2-105}$$

$$\epsilon_{cu} = \begin{cases} 0.003 \text{ for } f'_c \leq 4 \\ \frac{0.033}{7 + f'_c} \geq 0.0022 \text{ for } f'_c > 4 \end{cases} \quad \text{Equation 2-106}$$

where f'_c is in ksi.

2.2.4.12 Oztekin et al. (2003)

Oztekin et al. (2003) proposed stress block parameters obtained from the compression test results of 6×12 in. cylinders composed of eight series of high-performance concretes (HPC). Regression analysis was performed between the stress block parameters and the

concrete compressive strength. ε_{cu} proposed by CEB Model (1990) was modified according to the experimental tests results. The following parameters were derived:

$$\alpha_1 = 0.964 - 0.01379 f'_c \quad \text{Equation 2-107}$$

$$\beta_1 = 0.805 - 0.00827 f'_c \quad \text{Equation 2-108}$$

$$\varepsilon_{cu} = [2.2 + 0.1034(f'_c - 5.8)] \times 10^{-3} \text{ for } 8.7 \leq f'_c \leq 13.6 \quad \text{Equation 2-109}$$

where f'_c is in ksi.

2.2.4.13 Ozbakkaloglu and Saatcioglu (2003)

Ozbakkaloglu and Saatcioglu (2003) developed a rectangular stress block for a wide range of concrete compressive strengths up to 18.9 ksi. The stress-strain relationship proposed by Popovics (1973) was selected to derive the generalized stress-strain parameters. This relationship was applicable to eccentrically loaded members for HSC, as well as NSC. The descending portion of the curve was simplified to be linear. The curve was integrated numerically up to the ultimate compressive strain to find the area and the centroid of area for different concrete compressive strengths. This analytical model was verified by a large volume of column tests. The proposed rectangular stress block parameters were as follows:

$$\alpha_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 4 \\ 0.85 - 0.01(f'_c - 4) \geq 0.72 \text{ for } f'_c > 4 \end{cases} \quad \text{Equation 2-110}$$

$$\beta_1 = \begin{cases} 0.85 \text{ for } f'_c \leq 4 \\ 0.85 - 0.014(f'_c - 4) \geq 0.67 \text{ for } f'_c > 4 \end{cases} \quad \text{Equation 2-111}$$

$$\varepsilon_{cu} = 0.003 \quad \text{Equation 2-112}$$

where f'_c is in ksi.

2.2.4.14 Tan and Nguyen (2005)

Tan and Nguyen (2005) proposed a new equivalent rectangular stress block for concrete in flexure calibrated with the published test data on stress block parameters. It was stated

that β_1 values of the ACI code were lower than the experimental data, which overestimated the distance from the resultant force to the neutral axis position. The parameter β_1 was kept close to experimental values, while the product of $\alpha_1 \times \beta_1$ was chosen smaller than the test values. For simplicity, linear expressions for β_1 and $\alpha_1 \times \beta_1$ were chosen for regression analysis of the test data in the range of concrete compressive strengths up to 14.8 ksi because data beyond this range showed a different trend. On this basis, one linear equation was found to be enough for both parameters as follows:

$$\alpha_1 = \beta_1 = \begin{cases} 0.9 - 0.01379 f'_c & \text{for } f'_c \leq 14.5 \\ 0.7 & \text{for } f'_c > 14.5 \end{cases} \quad \text{Equation 2-113}$$

$$\epsilon_{cu} = 0.003 \quad \text{Equation 2-114}$$

where f'_c is in ksi.

2.2.4.15 Summary of the Proposed Rectangular Stress Block

The equations proposed for rectangular stress block parameters by different researchers are presented in Table 2-5. The comparisons of these equations are shown in Figure 2-24 and Figure 2-25. These equations are used to compare the test data obtained from this research and other researches in literature in Chapter 5.

Table 2-5 – Summary of the Proposed Rectangular Stress Block Parameters

Reference	α_1	β_1	ϵ_{cu}
Mattock et al. (1961)	0.85	0.85 for $f'_c \leq 4$ 0.85 – 0.05($f'_c - 4$) for $f'_c > 4$	0.003
Li (1993)	0.85 for $f'_c \leq 8$ 0.85 – 0.02758($f'_c - 8$) ≥ 0.75 for $f'_c > 8$	0.85 for $f'_c \leq 4.35$ 0.85 – 0.05516($f'_c - 4.35$) ≥ 0.65 for $f'_c > 4.35$	0.003
Azizinamini et al. (1994)	0.85 for $f'_c \leq 10$ 0.85 – 0.05($f'_c - 10$) ≥ 0.60 for $f'_c > 10$	0.85 for $f'_c \leq 4.35$ 0.85 – 0.05516($f'_c - 4.35$) ≥ 0.65 for $f'_c > 4.35$	0.003
Ibrahim (1994)	$0.85 - \frac{f'_c}{116} \geq 0.725$	$0.95 - \frac{f'_c}{58} \geq 0.70$	0.003
Pendyala and Mendis (1998)	0.85 – 0.01724($f'_c - 8.3$) for $8.3 \leq f'_c \leq 14.5$	0.65 – 0.00862($f'_c - 8.3$) for $8.3 \leq f'_c \leq 14.5$	0.003
Attard and Stewart (1998)	$1.2932 \left(\frac{f'_c}{0.145} \right)^{-0.0998} \geq 0.71$ for Dogbone Tests $0.6470 \left(\frac{f'_c}{0.145} \right)^{0.0324} \geq 0.58$ for Sustained Load Tests	$1.0948 \left(\frac{f'_c}{0.145} \right)^{-0.091} \geq 0.67$	0.003
Rangan (1999)	0.85 for $f'_c \leq 8$ 0.85 – 0.02758($f'_c - 8$) ≥ 0.75 for $f'_c > 8$	0.85 for $f'_c \leq 4.35$ 0.85 – 0.05516($f'_c - 4.35$) ≥ 0.65 for $f'_c > 4.35$	0.003
Bae and Bayrak (2003)	0.85 for $f'_c \leq 10.2$ 0.85 – 0.02758($f'_c - 10.2$) ≥ 0.67 for $f'_c > 10.2$	0.85 for $f'_c \leq 4.35$ 0.85 – 0.02758($f'_c - 4.35$) ≥ 0.67 for $f'_c > 4.35$	0.003 for $f'_c \leq 8$ 0.0025 for $f'_c \geq 8$
Sun et al. (2003)	0.85	0.85 for $f'_c \leq 2.94$ $\frac{28}{30 + f'_c} \geq 0.70$ for $f'_c > 2.94$	0.003 for $f'_c \leq 4$ $\frac{0.033}{7 + f'_c} \geq 0.0022$ for $f'_c > 4$
Oztekin et al. (2003)	$0.964 - 0.01379 f'_c$	$0.805 - 0.00827 f'_c$	$[2.2 + 0.1034(f'_c - 5.8)] \times 10^{-3}$ for $8.7 \leq f'_c \leq 13.6$
Ozbakkaloglu and Saatcioglu (2003)	0.85 for $f'_c \leq 4$ 0.85 – 0.01($f'_c - 4$) ≥ 0.72 for $f'_c > 4$	0.85 for $f'_c \leq 4$ 0.85 – 0.014($f'_c - 4$) ≥ 0.67 for $f'_c > 4$	0.003
Tan and Nguyen (2005)	0.9 – 0.01379 f'_c for $f'_c \leq 14.5$ 0.7 for $f'_c > 14.5$	0.9 – 0.01379 f'_c for $f'_c \leq 14.5$ 0.7 for $f'_c > 14.5$	0.003

where f'_c is in ksi.

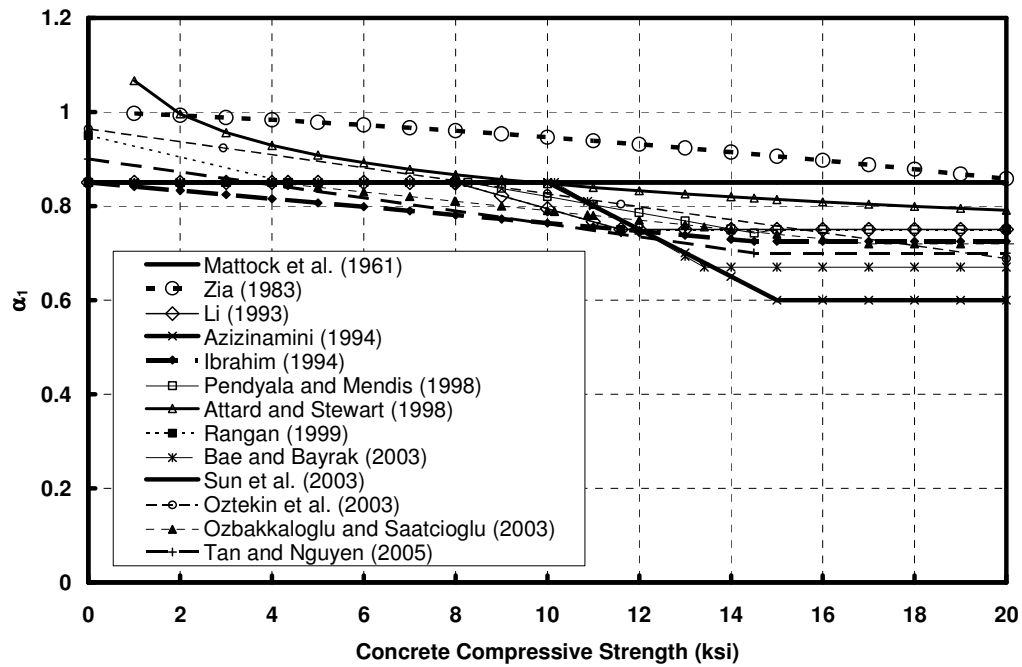


Figure 2-24 – Proposed Equations for α_l by Researchers

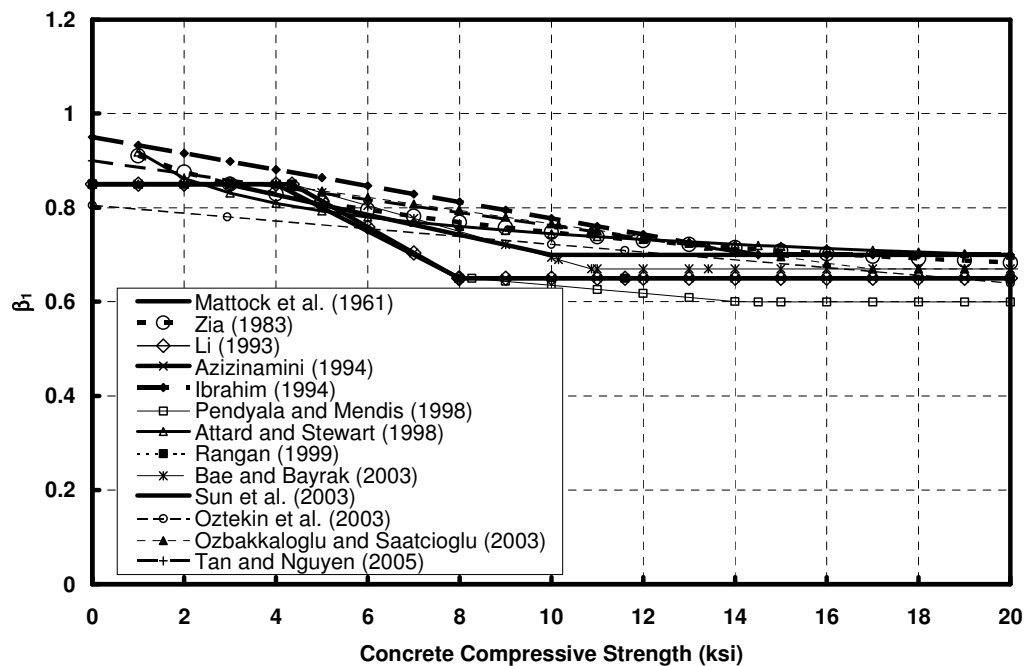


Figure 2-25 – Proposed Equations for β_l by Researchers

2.2.5 Rectangular Stress Block Parameters in Design Codes

2.2.5.1 ACI 318 (2005) and AASHTO LRFD Bridge Design Specifications (2004)

American Concrete Institute 318 (2005), “Building Code Requirements for Structural Concrete” and American Association of State Highway and Transportation Officials Load and Resistance Factor Design Bridge Design Specifications (2004) provide same rectangular stress block parameters for flexural design which were originally proposed by Mattock et al. (1961). Although no upper limit on concrete compressive strength is specified by ACI 318 (2005), the upper limit for AASHTO LRFD Bridge Design Specifications (2004) is 10 ksi. The maximum usable strain at the extreme concrete compression fiber is assumed to be 0.003. The equivalent stress block parameters are specified as follows:

$$\alpha_1 = 0.85 \quad \text{Equation 2-115}$$

$$\beta_1 = \begin{cases} 0.85 & \text{for } f'_c \leq 4 \\ 0.85 - 0.05(f'_c - 4) \geq 0.65 & \text{for } f'_c > 4 \end{cases} \quad \text{Equation 2-116}$$

$$\epsilon_{cu} = 0.003 \quad \text{Equation 2-117}$$

where f'_c is in ksi.

2.2.5.2 CSA A23.3 (1994) and CSA S6 (2001)

Similar rectangular stress block parameters are introduced by Canadian Standards Association, Standard A23.3 (1994), “Design of Concrete Structures” and Canadian Standards Association, Standard S6 (2001), “Canadian Highway Bridge Design Code”. This model was mainly the modified version of the equations proposed by Ibrahim and MacGregor (1997). These equations consider the more nearly triangular distribution of the higher strengths of concrete. The provisions specified by CSA A23.3 (1994) are applicable to concrete with a compressive cylinder strength ranging from 2.9 ksi to 11.6 ksi whereas the provisions specified by CSA S6 (2001) are applicable to concrete with a compressive strength ranging from 4.4 ksi for non-prestressed and 5.1 ksi for prestressed concrete to 12.3 ksi. The upper limits in both codes are established due to the lack of knowledge in behavior of structural elements with HSC. In the calculation of the

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rectangular stress block, the maximum usable strain at the extreme concrete compression fiber is assumed to be 0.0035. The rectangular stress block parameters used in these codes are as follows:

$$\alpha_1 = 0.85 - 0.01034 f'_c \geq 0.67 \quad \text{Equation 2-118}$$

$$\beta_1 = 0.97 - 0.01724 f'_c \geq 0.67 \quad \text{Equation 2-119}$$

$$\varepsilon_{cu} = 0.0035 \quad \text{Equation 2-120}$$

where f'_c is in ksi.

2.2.5.3 NZS 3101 (1995)

The stress block parameters specified by New Zealand Concrete Structures Code 3101 (1995) were based on the work performed by Li (1993). The model has a tri-linear shape for both α_1 and β_1 as follows:

$$\alpha_1 = \begin{cases} 0.85 & \text{for } f'_c \leq 8 \\ 0.85 - 0.02758(f'_c - 8) \geq 0.75 & \text{for } f'_c > 8 \end{cases} \quad \text{Equation 2-121}$$

$$\beta_1 = \begin{cases} 0.85 & \text{for } f'_c \leq 4.35 \\ 0.85 - 0.05516(f'_c - 4.35) \geq 0.65 & \text{for } f'_c > 4.35 \end{cases} \quad \text{Equation 2-122}$$

$$\varepsilon_{cu} = 0.003 \quad \text{Equation 2-123}$$

where f'_c is in ksi.

2.2.5.4 EC2 (2004)

The rectangular stress block specified in Eurocode 2 (2004) is applicable for concrete compressive strengths up to 13 ksi. Constant stress block parameters and ultimate compressive strain of concrete up to 7.25 ksi are modified to be used by higher concrete compressive strengths. The concrete compressive strength in EC 2 (2004) is based on characteristic cylinder strength. The following stress block parameters and ultimate compressive strain of concrete are proposed:

$$\alpha_1 = \begin{cases} \alpha_{cc} & \text{for } f'_{ck} \leq 7.25 \\ \alpha_{cc} \left(1 - \frac{f'_{ck} - 7.25}{29} \right) & \text{for } 7.25 \leq f'_{ck} \leq 13.0 \end{cases} \quad \text{Equation 2-124}$$

$$\beta_1 = \begin{cases} 0.80 & \text{for } f'_{ck} \leq 7.25 \\ 0.80 - \frac{f'_{ck} - 7.25}{68} & \text{for } 7.25 \leq f'_{ck} \leq 13.0 \end{cases} \quad \text{Equation 2-125}$$

$$\epsilon_{cu} = \begin{cases} 0.0035 & \text{for } f'_{ck} \leq 7.25 \\ 0.0026 + 0.035 \left(\frac{13.05 - f'_{ck}}{14.5} \right)^4 & \text{for } 7.25 \leq f'_{ck} \leq 13.0 \end{cases} \quad \text{Equation 2-126}$$

where f'_c is in ksi.

The α_{cc} factor which accounts for long term effects on compressive strength and unfavorable effects resulting the way load is applied is recommended to be *1.0* by EC 2 (2004). However, this factor may be changed by amendments in a National Annex when adopted by an individual country. Many countries in Europe have adopted the value of α_{cc} as 0.85.

2.2.5.5 NS 3473 (1995)

The Norwegian Concrete Structures Code 3473 (1995) provides discrete values for various concrete compressive strength as presented in Table 2-6.

Table 2-6 – Stress Block Parameters for the Norwegian Code NS 3473 (1995)

Concrete Compressive Strength (ksi)	3.6	5.1	6.5	8.0	9.4	10.9	12.3
ϵ_{cu}	0.0035	0.0035	0.0035	0.0035	0.0032	0.00305	0.0029
β_1	0.80	0.80	0.80	0.80	0.80	0.78	0.76
α_1	1	1	1	1	0.97	0.96	0.95

2.2.5.6 CEB-FIB Model Code (1990)

The rectangular stress block specified by CEB-FIB Model Code (1990) has the following parameters:

$$\alpha_1 = 0.85 \left(1 - \frac{f'_c}{36.3} \right) \quad \text{Equation 2-127}$$

$$\beta_1 = 1.0 \quad \text{Equation 2-128}$$

$$\varepsilon_{cu} = 0.004 - 0.002 \frac{f'_c}{14.5} \quad \text{Equation 2-129}$$

where f'_c is in ksi.

2.2.5.7 ACI 441-R96 (1996)

ACI Committee 441 (1996) on HSC columns suggested the following rectangular stress block parameters based on the research data in the literature on HSC.

$$\alpha_1 = 0.85 - 0.05(f'_c - 10) \geq 0.60 \text{ for } f'_c > 10 \quad \text{Equation 2-130}$$

$$\beta_1 = 0.67 \text{ for } f'_c \geq 10 \quad \text{Equation 2-131}$$

$$\varepsilon_{cu} = 0.003 \quad \text{Equation 2-132}$$

where f'_c is in ksi.

2.2.5.8 Summary of the Rectangular Stress Block Parameters in Design Codes

The equations for rectangular stress block parameters specified by different design codes worldwide are given in Table 2-7. The comparisons of these equations are shown in Figure 2-26 and Figure 2-27. These equations are used to compare the test data obtained from this research and other researches in literature in Chapter 5.

Table 2-7 – Summary of the Rectangular Stress Block Parameters in Design Codes

Reference	α_1	β_1	ϵ_{cu}
LRFD (2004) and ACI 318 (2005)	0.85	$0.85 \text{ for } f'_c \leq 4$ $0.85 - 0.05(f'_c - 4) \geq 0.65 \text{ for } f'_c > 4$	0.003
NZS 3101 (1995)	$0.85 \text{ for } f'_c \leq 8$ $0.85 - 0.02758(f'_c - 8) \geq 0.75 \text{ for } f'_c > 8$	$0.85 \text{ for } f'_c \leq 4.35$ $0.85 - 0.05516(f'_c - 4.35) \geq 0.65 \text{ for } f'_c > 4.35$	0.003
CSA A23.3 (1994)	$0.85 - 0.01034f'_c \geq 0.67$	$0.97 - 0.01724f'_c \geq 0.67$	0.0035
EC2 (2004)	$\alpha_{cc} \text{ for } f'_{ck} \leq 7.25$ $\alpha_{cc} \left(1 - \frac{f'_{ck} - 7.25}{29} \right) \text{ for } 7.25 \leq f'_{ck} \leq 13.05$	$0.80 \text{ for } f'_{ck} \leq 7.25$ $0.80 - \frac{f'_{ck} - 7.25}{68} \text{ for } 7.25 \leq f'_{ck} \leq 13.05$	$0.0035 \text{ for } f'_{ck} \leq 7.25$ $0.0026 + 0.035 \left(\frac{13.05 - f'_{ck}}{14.5} \right)^4$ $\text{for } 7.25 \leq f'_{ck} \leq 13.05$
NS 3473 (1995)			$0.0035 \text{ for } f'_{ck} \leq 8$
CEB-FIB Model Code (1990)	$0.85 \left(1 - \frac{f'_c}{36.3} \right)$	1	$0.004 - 0.002 \frac{f'_c}{14.5}$
ACI 441-R96 (1996)	$0.85 - 0.05(f'_c - 10) \geq 0.60 \text{ for } f'_c > 10$	$0.67 \text{ for } f'_c \geq 10$	0.003

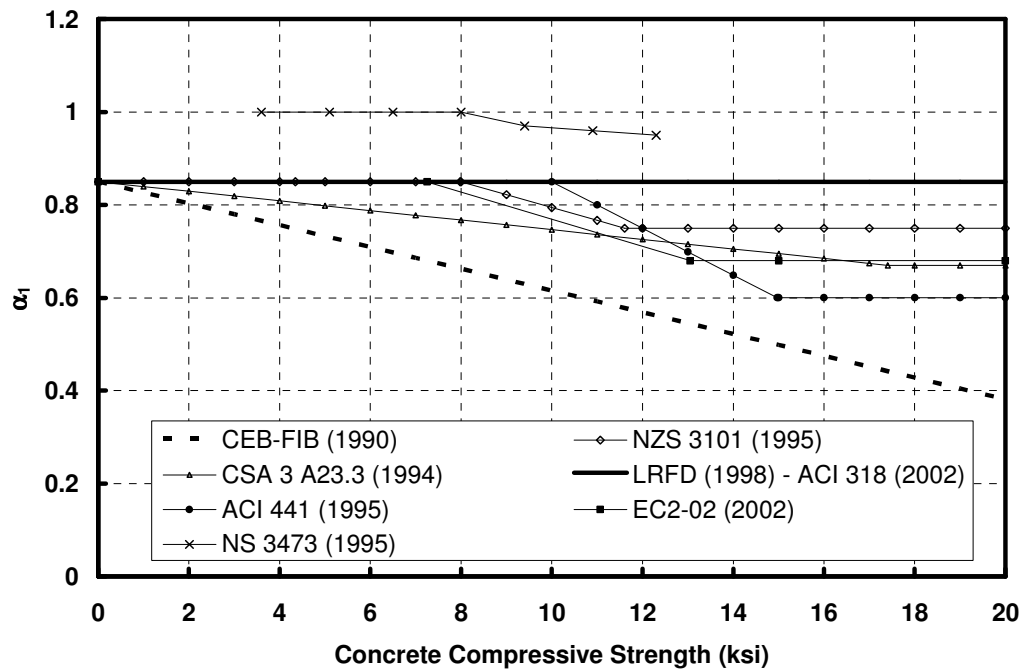


Figure 2-26 – α_1 in Design Codes

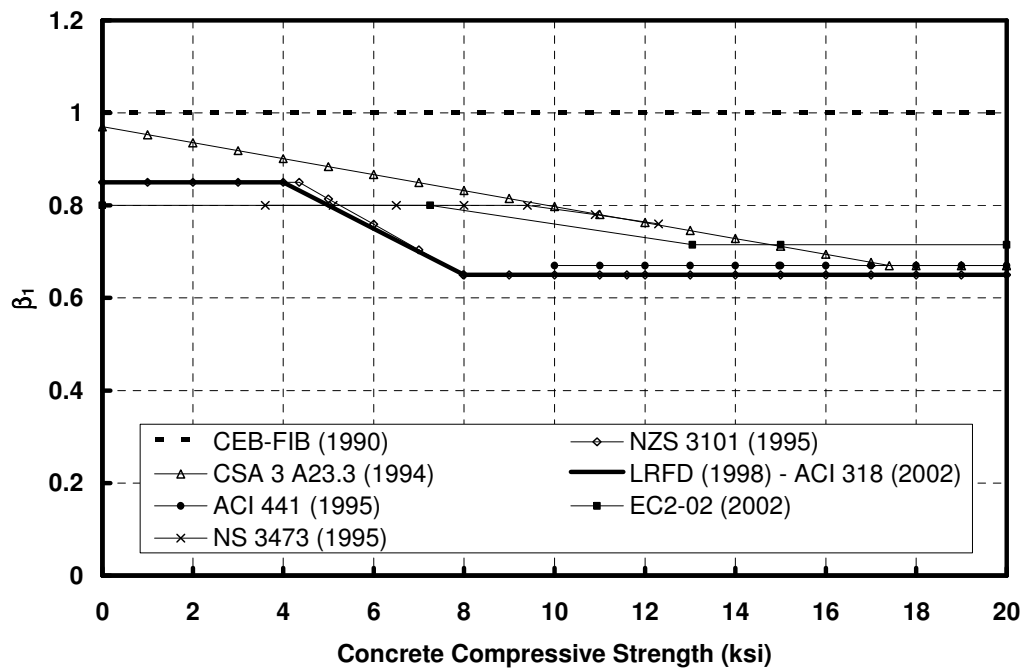


Figure 2-27 – β_1 in Design Codes

2.3 Poisson's Ratio of HSC

Poisson's ratio is defined as the ratio of the longitudinal compressive strain of concrete in the direction of the applied load divided by the transverse tensile strain of concrete normal to the applied load. For a perfectly incompressible material, Poisson's ratio is exactly 0.5. For NSC in the linearly elastic range, Poisson's ratio is assumed to be 0.2 for design purposes by AASHTO LRFD Bridge Design Specifications (2004).

2.3.1 Komendant et al. (1978)

Komendant et al. (1978) used 6×16 in. cylinders to evaluate the creep behavior of concrete under various combinations of temperatures varying from 73 to 160°F, stress levels varying from 30 to 60 percent of concrete compressive strength and age of loading varying from 28 to 270 days. The concrete compressive strengths at 60 days was 7.5 ksi for moist cured specimens and 7.0 ksi for sealed specimens. As a byproduct of the testing program, Poisson's ratio of concrete was also determined. Tests results for Poisson's ratio are given in Appendix D. It was concluded that Poisson's ratio, which averaged 0.22, was not affected by thermal variations.

2.3.2 Perenchio and Klieger (1978)

Perenchio and Klieger (1978) tested air-entrained and non-air-entrained concretes with three different aggregates at water-cement ratios of 0.30, 0.35 and 0.40. Concrete compressive strengths varied from 7.2 to 11.6 ksi. The behaviors of the 6×12 in. cylinders were evaluated for concrete compressive strength, modulus of elasticity, creep and Poisson's ratio. Tests results for Poisson's ratio are given in Appendix D. It was concluded that Poisson's ratio did not show a good correlation, but tended to decrease with increasing water-cement ratios.

2.3.3 Carrasquillo et al. (1981)

Carrasquillo et al. (1981) conducted an experimental program to evaluate the behavior of concretes with water-cement ratios varying from 0.70 to 0.32 and concrete compressive strengths ranging from 3.0 to 11.0 ksi. Two different aggregates were used in the concrete mixture. The test results were evaluated for concrete compressive strength, stress-strain

relationship, strength gain with age, specimen size effect, effects of drying, modulus of elasticity, Poisson's ratio, modulus of rupture and split cylinder strength. Tests performed on 4×8 in. cylinders indicated that Poisson's ratio is essentially 0.2 regardless of the concrete compressive strength for both gravel and limestone concretes. Tests results for Poisson's ratio are given in Appendix D.

2.3.4 Swartz et al. (1985)

Swartz et al. (1985) evaluated Poisson's ratio by testing eccentric bracket specimens with concrete compressive strengths ranging from 8.4 to 12.3 ksi. The central unreinforced test region had a cross-section of 5×5 in. and was 16 in. long. Tests results for Poisson's ratio are given in Appendix D. It was concluded that within a given mixture design, the values were quite consistent. However, from mixture to mixture, the values varied from 0.14 to 0.23. No explanation was offered for this behavior.

2.3.5 Jerath and Yamane (1987)

Jerath and Yamane (1987) performed a study to evaluate the effects of superplasticizers on concrete compressive strength, stress-strain relationship, modulus of elasticity, Poisson's ratio, modulus of rupture and split cylinder strength. Six by twelve inches concrete cylinders with water-cement ratios ranging from 0.28 to 0.55 and with concrete compressive strengths ranging from 3.7 to 9.5 ksi were tested to determine characteristics of twelve different mixture designs. Tests results for Poisson's ratio are given in Appendix D. It was concluded that Poisson's ratio was about 0.2 for various strengths of concrete for mixtures both with and without superplasticizers.

2.3.6 Radain et al. (1993)

Radain et al. tested 6×12 in. cylinders to evaluate Poisson's ratio of concrete with compressive strengths ranging from 5.77 to 13.0 ksi. Poisson's ratio obtained varied from 0.15 to 0.23 with an average value close to 0.19. Test results were compatible with the available data in the literature.

2.3.7 Ibrahim (1994)

The average values of Poisson's ratio were determined based on 14 eccentric bracket specimens tested under combined axial load and bending by Ibrahim (1994). Concrete compressive strengths ranged from 8.6 to 18.8 ksi. The central test region had a cross-section of 8×12 in. and was 47 in. long. It was concluded that the average values of Poisson's ratio in the elastic range varied between 0.16 and 0.22.

2.3.8 Iravani (1996)

Iravani (1996) tested 4×8 in. cylinders to determine the mechanical properties of HPC with compressive strengths ranging from 9.4 to 17.4 ksi. Tests results were evaluated based on concrete compressive strength gain with time, effect of type of cement, effect of drying, modulus of elasticity, Poisson's ratio and splitting tensile strength. Tests results for Poisson's ratio are given in Appendix D. Test results indicated that Poisson's ratio was about 0.18 on average at 40 percent of the ultimate stress.

2.3.9 Persson (1999)

Persson (1999) performed an experimental and numerical study on air-cured and sealed specimens to evaluate Poisson's ratio of HPC. The parameters considered in the testing program were influence of maturity, type of aggregate, curing type and moisture. The concrete compressive strength at 28 days ranged from 7.3 to 16.5 ksi. Cylinders with 2.2×11.8 in. dimensions and 3.9 in. cubes were tested. Test results indicated that Poisson's ratio of HPC is slightly smaller than that of NSC. It was also concluded that Poisson's ratio was not affected by the variations in relative humidity.

2.3.10 Rashid et al. (2002)

Rashid et al. (2002) performed an extensive literature survey to collect approximately 153 experimental values of Poisson's ratio of concretes in the elastic range. Based on these experimental values, it was concluded that Poisson's ratio of HSC was comparable to the expected range of values for NSC. Test results indicated that Poisson's ratio was varying from 0.15 to 0.25 and an average value of 0.2 was a reasonable value for concrete compressive strengths ranging from 2.9 to 17.4 ksi.

2.3.11 Logan (2005)

Poisson's ratio was determined using 4×8 in. cylinders with concrete compressive strengths ranging from 5.1 ksi to 16.8 ksi by Logan (2005). Tests results are given in Appendix D. The measured Poisson's ratio of concrete showed large amounts of variations and did not show apparent correlation with the measured concrete compressive strength. In addition, it was observed that curing procedures and age had little or no effect on Poisson's ratio. The average Poisson's ratio for all of the tested cylinders was 0.17 with a standard deviation of 0.07. It was suggested that same Poisson's ratio of NSC can be assumed for also HSC.

2.3.12 Summary of the Tests on Poisson's Ratio

The summary of the tests performed by different researchers to evaluate Poisson's ratio of concrete is shown in Table 2-8. Poisson's ratio's obtained by different researchers are also shown in Figure 2-28.

Table 2-8 – Summary of the Tests on Poisson's Ratio

Reference	Concrete Compressive Strength (ksi)	Specimen Size and Type (in.)	Conclusion
Komendant et al. (1978)	7.0 – 7.5	6×16 Cylinder	Average ν is 0.22.
Perenchio and Klieger (1978)	7.2 – 11.6	6×12 Cylinder	ν tends to decrease with increasing water-cement ratios.
Carrasquillo et al. (1981)	3.0 – 11.0	4×8 Cylinder	ν is 0.2 regardless of the concrete compressive strength.
Swartz et al. (1985)	8.4 – 12.3	Cross-Section: 5×5 Length: 16 Eccentric Bracket	ν varies from 0.14 to 0.23.
Jerath and Yamane (1987)	3.7 – 9.5	6×12 Cylinder	ν is 0.2 for various concrete compressive strengths.
Radain et al. (1993)	5.77 – 13.0	6×12 Cylinder	ν varies from 0.15 to 0.23 with an average value close to 0.19.
Ibrahim (1994)	8.6 – 18.8	Cross-Section: 8×12 Length: 47 Eccentric Bracket	ν varies from 0.16 and 0.22.
Iravani (1996)	9.4 – 17.4	4×8 Cylinder	Average ν is 0.18.
Persson (1999)	7.3 – 16.5	2.2×11.8 Cylinder 3.9 Cube	ν of HPC is slightly smaller than that of NSC.
Logan (2005)	5.1 – 16.8	4×8 Cylinder	Average ν is 0.17.

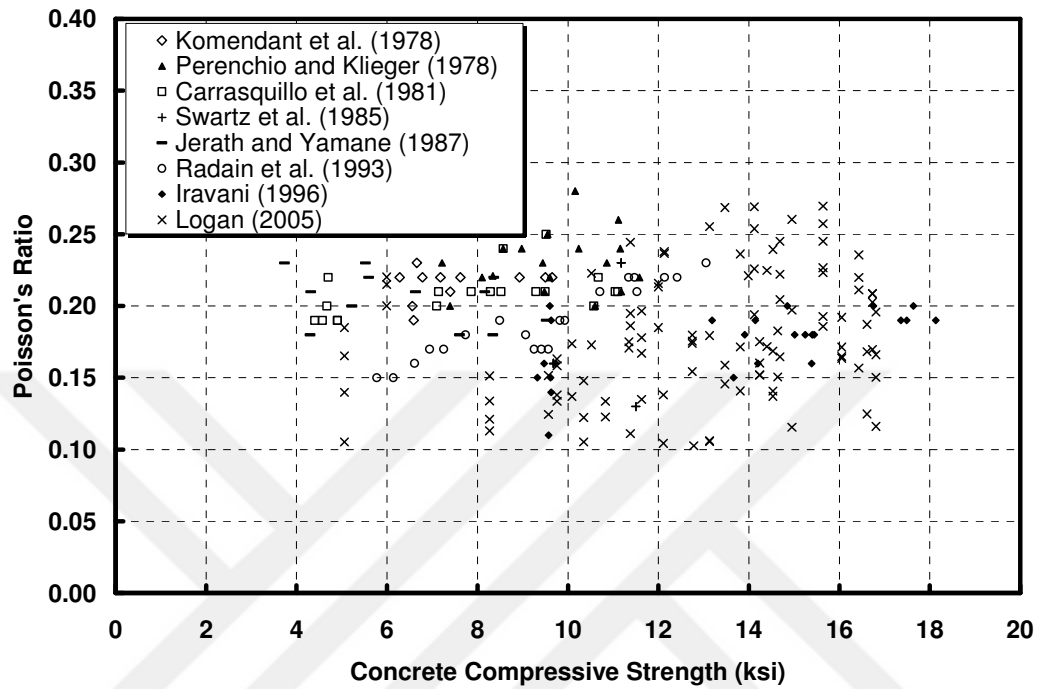


Figure 2-28 – Poisson's Ratios by Different Researchers

2.4 Creep and Shrinkage of HSC

After the application of load (either tensile or compressive) on hardened concrete, initially, it deforms elastically due to the applied stresses. At this stage, the microstructure of concrete does not have any kinds of significant cracks since the applied load is not greater than the proportional limit. If this load is maintained on concrete, additional deformation will develop in concrete in time which is known as creep deformation. Under the sustained load, some micro cracking develops in concrete, faster at early ages and gets stable later. There are both recoverable and irrecoverable components of this process. After removal of the sustained load, nearly entire elastic deformation is recovered. Although some of the creep deformation is recovered, entire creep deformation is not fully compensated due to the changes in microstructure of concrete. The creep process starting from the application of sustained load and ending some time after the removal of the sustained load is shown in Figure 2-29.

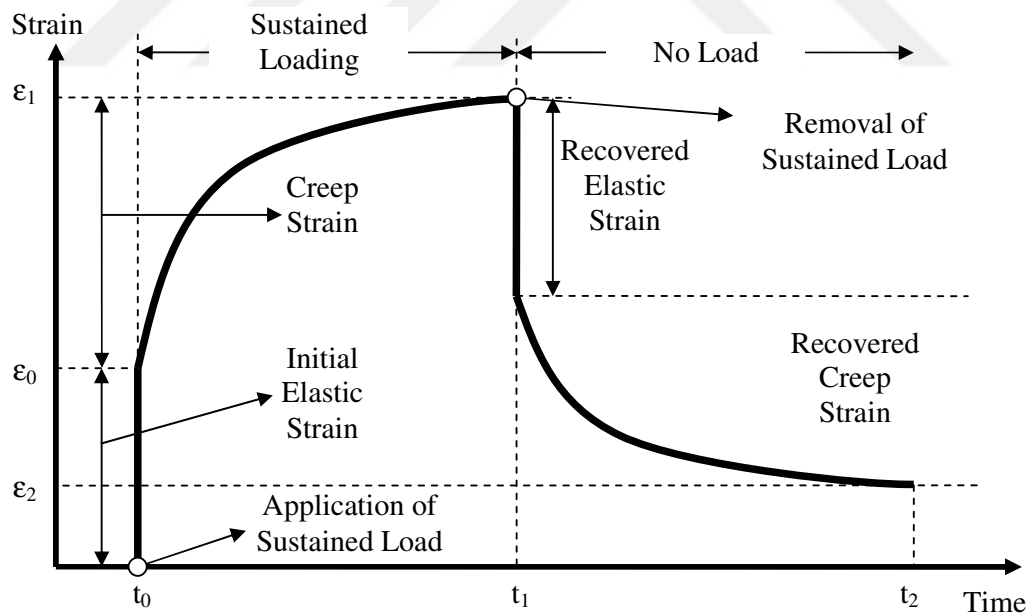


Figure 2-29 – Strain History of Concrete under Sustained Load

There are two types of creep of concrete: Basic creep occurs under constant moisture conditions. Drying creep is the additional creep in excess of basic creep that occurs due to a

moisture loss from the ambient conditions. Drying creep is a combined effect of shrinkage and creep whereas basic creep is an independent process.

After the concrete hardens, its volume reduces in time due to the loss of moisture content known as shrinkage. There are three main types of shrinkage of concrete: Drying shrinkage occurs due to the loss of moisture content from hardened concrete under drying conditions. This process is partially irreversible, even though concrete is placed in a fully moist environment, not all the drying shrinkage will be recovered. Autogenous (chemical) shrinkage occurs due to removal of internal water as a result of hydration of cement. Carbonation shrinkage occurs due to the carbonation of the hydration products in the presence of CO in a low relative humidity environment.

Reinforced concrete structures must be carefully designed to avoid problems associated with creep and shrinkage. Creep and shrinkage of concrete might be a significant problem since they may cause:

- Deformations significantly larger than elastic deformations
- Redistribution of internal forces which may create additional forces on unexpected members
- Cracking and degradation of structures which may result in corrosion of reinforcement, spalling of concrete, loss of serviceability and possibly collapse.
- Increased prestress losses in prestressed members
- A slow long time growth of buckling deflections in slender and thin structures.

More information and case studies about the problems associated with creep and shrinkage can be found in Neville (2002a, 2002b).

Both creep and shrinkage is dependent on many factors such as, age of concrete, water to cementitious materials ratio, aggregate, composition, unit weight of concrete, modulus of elasticity, relative humidity, temperature, volume to surface ratio. Additionally, creep also depends on the strength of concrete at the time of application of the sustained load and

magnitude of the applied stress. The effects of these parameters on creep and shrinkage of concrete can be found in details in ACI 209R-92 (1992).

2.4.1 Tests on Creep and Shrinkage of HSC

The tests performed to evaluate the creep and shrinkage behavior of concrete is presented in this section. The conclusions of these tests will be used to evaluate the test results of this research.

2.4.1.1 Ngab et al. (1981)

Ngab et al. (1981) conducted an experimental program to evaluate the time dependent deformation and sustained load strength of HSC. Specimens with 3.5×3.5×10.5 dimensions were monitored for a period up to 90 to determine drying shrinkage, basic creep and drying creep of HSC. The main variables investigated included concrete compressive strength (varying from 4.2 to 10.2 ksi), conditions of drying (sealed and unsealed), age at first loading (2 and 28 days) and, intensity of load (varying from 0.45 to 0.85 f'_c), and duration of load (60 and 90 days). Test results indicated that creep might be much less for HSC than that of NSC, especially when specimens are permitted to dry under sustained load. It was observed that shrinkage of HSC was greater than that of NSC. The sustained compressive stress over one-half of the concrete compressive strength was found to have detrimental effects on both normal- and HSC. Regardless of stress intensity, the ratio of sustained load strength to compressive strength was observed to be higher for HSC than for NSC.

2.4.1.2 Collins (1989)

An experimental program was conducted by Collins (1989) on creep and shrinkage of HSC. Variations in cement paste content and maximum aggregate size was the main parameters investigated in this study. For this purpose, five mixture designs were tested with concrete compressive strengths ranging from 8.05 to 9.28 ksi at 28 days. Creep and shrinkage testing were performed in accordance with ASTM C 512 in an environmental chamber maintained at 70°F temperature and 50 percent relative humidity. Shrinkage testing was performed on concrete specimens after moist curing for 7 and 28 days, whereas creep testing was performed on concrete specimens after moist cured for 28 days. Test results indicated that creep and shrinkage deformations are somewhat less for concrete mixtures with lower cement paste contents and larger aggregate size. It was observed that the shrinkage deformations

were inversely proportional to the moist curing time and the creep deformations increased directly with an increase in the applied stress level. It was concluded that although the creep and shrinkage might be reduced by careful mixture proportioning and aggregate selection, the applied stress level has significantly important effect on column shortening.

2.4.1.3 Paulsen et al. (1991)

Paulsen et al. (1991) evaluated the long term deflection of HSC using nine under-reinforced beams with concrete compressive strengths ranging from 6 to 12 ksi, loaded over a 12 months period. The beams had also varying amounts of compressive steel. Companion axially loaded 4×16 in. concrete cylinders were used to determine creep coefficients. It was observed that the creep coefficient for HSC was about half of that of NSC. Based on the test results, modifications to the ACI Building Code method of predicting long term deflections were suggested.

2.4.1.4 Giaccio et al. (1993)

Giaccio et al. (1993) performed an extensive research program to evaluate the creep and shrinkage properties of 6×12 in. cylinders with concrete compressive strengths ranging from 3.4 to 12.3 ksi. Three mixture designs were used with water to cement ratios varying from 0.65 to 0.30. The creep specimens were loaded under 20 percent of the compressive strength at the age of loading where the specimens were loaded in the 1st and the 28th day. It was observed that drying shrinkage of concrete slightly decreases as strength level increases. The specific creep of NSC was twice to three times greater than that of HSC. The test results indicated that the specific creep of a HSC loaded at early ages would be smaller than that of NSC which achieved the same strength level at later ages.

2.4.1.5 Khan et al. (1997)

Khan et al. conducted an experimental study on early-age shrinkage and thermal and creep strains of 4×8 in. cylinders with concrete compressive strengths ranging from 4.4 to 14.5 ksi subjected to sealed and air-dried conditions. It was observed that demolding at early ages resulted in greater shrinkage and thermal strains in HSC than in NSC. The test results indicated that creep of HSC is much more sensitive to the age of loading than that of NSC.

2.4.1.6 Mokhtarzadeh and French (2000)

Mokhtarzadeh and French tested 268 specimens to investigate the effect of mixture variations on creep, shrinkage and water absorption properties of HSC. Four by eleven inches cylinders were used in the evaluation of creep and shrinkage properties with concrete compressive strengths ranging from 8.0 to 18.6. The water to cementitious material ratio was 0.3 in all the mixtures. Both accelerated heat curing and 7-day moist curing were investigated.

All specimens were monitored for a period of one year. It was observed that the specific creep of concrete decreased as concrete compressive strength increased. It was concluded that concrete compressive strength and composition of cementitious material had no significant effect on drying shrinkage of HSC. Modifications were proposed to the ACI 209 creep and shrinkage prediction relationships to provide better estimation for HSC.

2.4.1.7 Huo et al. (2001)

Experiments on creep, shrinkage and modulus of elasticity of HSC with concrete compressive strengths ranging from 9.0 to 12.4 ksi were conducted and continued for more than two years. The creep and shrinkage specimens were 4×4×24 in. prisms. All the specimens were moist cured for either 7 or 28 days before the initial readings were recorded. Analysis and comparison of the test results showed that the shrinkage strains and creep coefficients of HSC were lower than those of NSC. Creep and shrinkage prediction relationships were proposed for concrete compressive strengths up to 15 ksi. These relationships were adapted by AASHTO LRFD Bridge Design Specifications (2004) after being proposed by Tadros et al. (2003).

2.4.1.8 Jianyong and Yan (2001)

Creep and drying shrinkage properties of HSC were investigated using three concrete mixtures with concrete compressive strengths ranging from 11.7 to 15.1 ksi cube strength at 28 days. The specimens for creep and shrinkage tests were prisms with 4×4×12 in. and 4×4×20 in. dimensions, respectively. The creep specimens were loaded up to 40 percent of the 28-day strength and monitored for 180 days. The shrinkage tests were performed at the same time under same humidity and temperature conditions. It was concluded that creep and shrinkage would greatly be reduced using mineral admixtures such as granulated blast furnace slag and silica fume.

2.4.1.9 Suksawang et al. (2005)

Suksawang et al. conducted a study to identify the shrinkage and creep of concrete with compressive strengths ranging from 10.4 to 12.7 ksi at 28 days. Water to binder ratio of the mixture was 0.27. Shrinkage and creep tests were performed using 3×3×11 in. prisms and 6×12 in. cylinders, respectively. The applied load on the creep specimens ranged from 30 to 35 percent of the ultimate load. Test results are given in Appendix E. Test results showed that shrinkage of silica fume concretes was greater than that of fly ash concretes. It was observed that adding fly ash reduced the drying shrinkage of silica fume concrete. Creep of silica fume concretes was greater than that of fly ash concretes.

2.4.1.10 Summary of the Tests on Creep and Shrinkage

The summary of the tests performed by different researchers to evaluate the creep and shrinkage behavior of concrete is shown in Table 2-9.

Table 2-9 - Summary of the Tests on Creep and Shrinkage

Reference	Concrete Compressive Strength (ksi)	Specimen Size and Type (in.)	Conclusion
Ngab et al. (1981)	4.2 – 10.2	3.5×3.5×10.5 Prism	Creep of HSC is much less than that of NSC. Shrinkage of HSC is greater than that of NSC
Collins (1989)	8.1 – 9.3	6×12 Cylinder	Creep and shrinkage deformations are less for concrete mixtures with lower cement paste contents and larger aggregate size.
Paulsen et al. (1991)	6.0 – 12.0	6×16 Cylinder	The creep coefficient for HSC is about half of that of NSC.
Giaccio et al (1993)	3.4 – 12.3	6×12 Cylinder	The specific creep of NSC was twice to three times greater than that of HSC.
Khan et al. (1997)	4.4 – 14.5	4×8 Cylinder	Demolding at early ages resulted in greater shrinkage and thermal strains in HSC than in NSC. Creep of HSC is much more sensitive to the age of loading than that of NSC.
Mokhtarzadeh and French (2000)	8.0 – 18.6	4×11 Cylinder	The specific creep of concrete decreased as compressive strength increased. Concrete compressive strength and composition of cementitious material had no significant effect on drying shrinkage of HSC.
Huo et al. (2001)	9.0 – 12.4	4×4×24 Prism	The shrinkage strains and creep coefficients of HSC were lower than those of NSC.
Jianyong and Yan (2001)	11.7 – 15.1	Creep: 4×4×12 Prism Shrinkage: 4×4×20 Prism	Creep and shrinkage would greatly be reduced using mineral admixtures.
Suksawang et al. (2005)	10.4 – 12.7	Creep: 6×12 Cylinder Shrinkage: 3×3×11 Prism	Creep and shrinkage of silica fume concretes were greater than that of fly ash concretes.

2.4.2 Creep and Shrinkage Predictions Models

Creep and shrinkage prediction models are investigated in this section in details. Necessary relationships and equations are given to calculate and the predicted values. These relationships will be used to compare the test results of this research and if needed a new relationship will be proposed in the light of these relationships.

2.4.2.1 ACI 209R-92 (1992)

ACI Committee 209 (ACI 209R-92 1992) suggests the following equation to predict the creep coefficient of concrete at any time (ν_t):

$$\nu_t = \left[t^\psi / (d + t^\psi) \right] \times \nu_u \quad \text{Equation 2-133}$$

where t is the time in days, ψ is a constant between 0.40 and 0.80, d is a constant between 6 and 30 days, ν_u is the ultimate creep coefficient which is between 1.30 and 4.15.

For NSC cured under standard conditions ACI Committee 209 (ACI 209R-92 1992) recommends the following creep coefficient for a loading age at 7 days for moist-cured concrete, and at 1-3 days, for steam-cured concrete:

$$\nu_t = \left[t^{0.60} / (10 + t^{0.60}) \right] \times 2.35 \quad \text{Equation 2-134}$$

ACI Committee 209 (ACI 209R-92 1992) suggests the following equation to predict shrinkage strain of concrete at any time ($(\epsilon_{sh})_t$):

$$(\epsilon_{sh})_t = \left[t^\alpha / (f + t^\alpha) \right] \times (\epsilon_{sh})_u \quad \text{Equation 2-135}$$

where t is the time in days, α is a constant between 0.90 and 1.10, f is a constant between 20 and 130 days, $(\epsilon_{sh})_u$ is the ultimate shrinkage strain which is between 415×10^{-6} and 1070×10^{-6} .

For normal weight, sand lightweight and all lightweight concrete under standard conditions ACI Committee 209 (ACI 209R-92 1992) recommends the following shrinkage strain:

$$(\varepsilon_{sh})_t = \left[t / (35 + t) \right] \times 780 \times 10^{-6} \text{ for 7-day moist-cured concrete} \quad \text{Equation 2-136}$$

$$(\varepsilon_{sh})_t = \left[t / (55 + t) \right] \times 780 \times 10^{-6} \text{ for steam-cured concrete} \quad \text{Equation 2-137}$$

2.4.2.2 CEB-FIB Model Code (1990)

The creep coefficient ($\phi(t, t_0)$) for service stressed less than $0.4f_{cm}(t_0)$ specified in CEB-FIB Model Code (1990) is given as:

$$\phi(t, t_0) = \phi_0 \beta_c(t - t_0) \quad \text{Equation 2-138}$$

where ϕ_0 is the notional creep coefficient, β_c is the coefficient to describe the development of creep with time after loading, t is the age of concrete (days) at the moment considered, t_0 is the age of concrete at loading (days). The relationship for the coefficients are shown as:

$$\phi_0 = \phi_{RH} \beta(f_{cm}) \beta(t_0) \quad \text{Equation 2-139}$$

with

$$\phi_{RH} = 1 + \frac{1 - RH / RH_0}{0.46(h/h_0)^{1/3}} \quad \text{Equation 2-140}$$

$$\beta(f_{cm}) = \frac{5.3}{(f_{cm} / f_{cmo})^{0.5}} \quad \text{Equation 2-141}$$

$$\beta(t_0) = \frac{1}{0.1 + (t_0 / t_1)^{0.2}} \quad \text{Equation 2-142}$$

$$h = 2A_c / u \quad \text{Equation 2-143}$$

where f_{cm} is the mean concrete compressive strength at the age of 28 days (MPa), f_{cmo} is equal to 10 MPa, RH is the relative humidity of ambient environment (%), RH_0 is equal to

100%, h is the notational size of member, A_c is the cross-section, u is the perimeter of the member in contact with the atmosphere, h_o is equal to 100 mm and t_1 is equal to 1 day. The development of creep with time is given by

$$\beta_c(t-t_0) = \left[\frac{(t-t_0)/t_1}{\beta_H + (t-t_0)/t_1} \right]^{0.3} \quad \text{Equation 2-144}$$

with

$$\beta_H = 150 \left\{ 1 + \left(1.2 \frac{RH}{RH_o} \right)^{18} \right\} \frac{h}{h_o} + 250 \leq 1500 \quad \text{Equation 2-145}$$

where t_1 is equal to 1 day, RH_o is equal to 100% and h_o is equal to 100 mm.

The total shrinkage $\varepsilon_{cs}(t, t_s)$ specified in CEB-FIB Model Code (1990) is given as:

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cso} \beta_s(t-t_s) \quad \text{Equation 2-146}$$

where ε_{cso} is the notional shrinkage coefficient, β_s is the coefficient to describe the development of shrinkage with time, t is the age of concrete in days and t_s is the age of concrete in days at the beginning of shrinkage. The notional shrinkage coefficient may be obtained from:

$$\varepsilon_{cso} = \varepsilon_s(f_{cm}) \beta_{RH} \quad \text{Equation 2-147}$$

with

$$\varepsilon_s(f_{cm}) = [160 + 10\beta_{sc}(9 - f_{cm}/f_{cmo})] \times 10^{-6} \quad \text{Equation 2-148}$$

where f_{cm} is the mean concrete compressive strength at the age of 28 days in MPa, f_{cmo} is equal to 10 MPa, β_{sc} is a coefficient which depends on the type of cement: β_{sc} is equal to 4 for slowly hardening cements SL, β_{sc} is equal to 5 for normal or rapid hardening cements N and R, and β_{sc} is equal to 8 for rapid hardening high strength cements RS,

$$\beta_{RH} = -1.55\beta_{sRH} \text{ for } 40\% \leq RH \leq 99\% \quad \text{Equation 2-149}$$

$$\beta_{RH} = +0.25 \text{ for } RH \geq 99\% \quad \text{Equation 2-150}$$

where

$$\beta_{sRH} = 1 - \left(\frac{RH}{RH_o} \right)^3 \quad \text{Equation 2-151}$$

with RH is the relative humidity of the ambient atmosphere in % and RH_o is equal to 100%. The development of shrinkage with time is given by

$$\beta_s(t - t_s) = \left[\frac{(t - t_s)/t_1}{350(h/h_o)^2 + (t - t_s)/t_1} \right]^{0.5} \quad \text{Equation 2-152}$$

where h is defined previously, t_1 is equal to 1 day and h_o is equal to 100 mm.

2.4.2.3 Tadros et al. (2003) and AASHTO LRFD Bridge Design Specifications (2004)

The creep coefficient (ψ) specified by Tadros et al. (2003) and AASHTO LRFD Bridge Design Specifications (2004) is given by:

$$\Psi(t, t_i) = 1.90k_{td}k_{la}k_s k_{hc}k_f \quad \text{Equation 2-153}$$

where t is the age of concrete after loading in days, t_i is the age of concrete when load is initially applied for accelerated curing or the age minus 6 days for moist curing in days, k_{td} is

the time development correction factor, k_{td} is the loading age correction factor, k_s is the member size correction factor, k_{hc} is the relative humidity correction factor and k_f is the concrete compressive strength factor. These factors are expressed as:

$$k_{td} = \frac{t}{61 - 4f'_{ci} + t} \quad \text{Equation 2-154}$$

$$k_{la} = t_i^{-0.118} \quad \text{Equation 2-155}$$

$$k_s = \frac{1064 - 94V/S}{735} \quad \text{Equation 2-156}$$

$$k_{hc} = 1.56 - 0.008H \quad \text{Equation 2-157}$$

$$k_f = \frac{5}{1 + f'_{ci}} \quad \text{Equation 2-158}$$

where f'_{ci} is the specified compressive strength in ksi at prestress transfer for prestressed members or 80 percent of the strength at service for non-prestressed members, V/S is the volume to surface ratio in inches and H is the relative humidity of the ambient air.

The shrinkage strain (ϵ_{sh}) specified by Tadros et al. (2003) and AASHTO LRFD Bridge Design Specifications is given by:

$$\epsilon_{sh} = 480 \times 10^{-6} k_{td} k_s k_{hs} k_f \quad \text{Equation 2-159}$$

where same previously defined correction factors are used except the member size correction factor, k_s , which is defined as:

$$k_{hs} = 2.00 - 0.0143H. \quad \text{Equation 2-160}$$

2.4.2.4 Australian Standard for Concrete Structures AS3600 (2006)

The model specified by AS3600 (2006) for predicting the creep coefficient (ϕ_{cc}) for concrete compressive strengths between 20 MPa and 100 MPa is given as:

$$\varphi_{cc} = k_2 k_3 k_4 k_5 \varphi_{cc,b} \quad \text{Equation 2-161}$$

where k_2 is the coefficient for development of creep in time, k_3 is the coefficient for loading age, k_4 is the coefficient for environment and k_5 is the coefficient for the influence of humidity and specimen size. These coefficients can be calculated as:

$$k_2 = \frac{\alpha_2 t^{0.8}}{t^{0.8} + 0.15 t_h} \quad \text{Equation 2-162}$$

where t is the time in days since first loading, t_h is the hypothetical thickness and α_2 can be calculated as:

$$\alpha_2 = 1.0 + 1.12 e^{-0.008 t_h} \quad \text{Equation 2-163}$$

$$k_3 = \begin{cases} 1.97 - 0.031\tau & \text{for } \tau \leq 28 \\ 1.119 - 0.0006\tau & \text{for } 28 < \tau < 365 \\ 0.9 & \text{for } \tau \geq 365 \end{cases} \quad \text{Equation 2-164}$$

where τ is the age of concrete at the time of loading in days.

$$k_4 = \begin{cases} 0.70 & \text{for an arid environment} \\ 0.65 & \text{for an interior environment} \\ 0.60 & \text{for a temperature environment} \\ 0.50 & \text{for a tropical/coastal environment} \end{cases} \quad \text{Equation 2-165}$$

$$k_5 = \begin{cases} 1.0 & \text{when } f'_c \leq 50 \text{ MPa} \\ (2.0 - \alpha_3) - 0.02(1.0 - \alpha_3) f'_c & \text{when } 50 \text{ MPa} < f'_c \leq 100 \text{ MPa} \end{cases} \quad \text{Equation 2-166}$$

where

$$\alpha_3 = \frac{0.7}{k_2 \alpha_2} \quad \text{Equation 2-167}$$

The model specified by AS3600 (2006) for predicting the drying shrinkage strain (ϵ_{csd}) is given as:

$$\epsilon_{csd} = k_1 k_4 \epsilon_{csd.b} \quad \text{Equation 2-168}$$

where k_1 is the coefficient for development of creep in time, k_4 is the coefficient for environment as defined previously and $\epsilon_{csd.b}$ is the basic drying shrinkage. The coefficient k_1 and can be calculated as:

$$k_1 = \frac{\alpha_1 t_d^{0.8}}{t_d^{0.8} + 0.15 t_h} \quad \text{Equation 2-169}$$

$$\epsilon_{csd.b} = (1.0 - 0.008 f'_c) \times \epsilon_{csd.b*} \quad \text{Equation 2-170}$$

where t_d is the time in days after the commencement of drying, t_h is the hypothetical thickness and $\epsilon_{csd.b*}$ is the coefficient for quality of the local concrete, including the type and the quantity of aggregates, cement, cement replacement materials and admixtures. α_1 and $\epsilon_{csd.b*}$ can be calculated from:

$$\alpha_1 = 0.8 + 1.2 e^{-0.005 t_h} \quad \text{Equation 2-171}$$

$$\epsilon_{csd.b*} = \left\{ \begin{array}{ll} 800 \times 10^{-6} & \text{for Sydney and Brisbane} \\ 900 \times 10^{-6} & \text{for Melbourne} \\ 800 \times 10^{-6} & \text{for elsewhere in Australia} \end{array} \right\}. \quad \text{Equation 2-172}$$

3 EXPERIMENTAL PROGRAM

3.1 General

This chapter describes the components of the experimental program undertaken to evaluate the stress-strain relationship, creep and shrinkage behavior of HSC. Test specimens, material properties, test set-ups, instrumentations and test procedures are summarized in the following sections.

3.2 Eccentric Bracket Tests

The purpose of eccentric bracket tests is to determine the stress-strain distribution of concrete based on linear strain profile within the compression zone and evaluate various parameters of the rectangular stress block. Generally, eccentric bracket specimens have two arms, each connected at the top and the bottom ends of the specimen to transfer the moment from the applied load to the arm to the mid-section of the specimen. As observed in the literature review, these arms may either be concrete which could be cast monolithically with the concrete specimen or steel which are connected to the concrete specimen by specially designed connections. In this research the second configuration was selected, using the steel arms to transfer the moment to the concrete.

3.2.1 Design of Test Specimens

The main objective in designing the test specimens is to have a specimen that can represent the compression zone of flexural member. The test specimens were designed according to the limitations of the testing machine and the testing equipment. The selected specimen was a square column with 9×9×40 in. dimensions. When this specimen is loaded under flexure and axial compression to obtain zero strain on one side and maximum strain on the other extreme opposite side, the cross-sectional area of the specimen represents mainly a compression zone of a beam member with 9×26 in. dimensions. To calculate the dimensions of the prototype member, the following assumptions were made:

1. $\varepsilon_t = 0.005$, $c/d_t = 0.375$
2. cover concrete = 2 in. from the center of the extreme tension reinforcement

where ε_t is the tensile strain at the centroid of the extreme tension reinforcement, c is the distance from the extreme compression fiber to the neutral axis and d_t is the distance from the extreme compression fiber to the centroid of the extreme tension reinforcement.

The prototype beam member is presented in Figure 3-1.

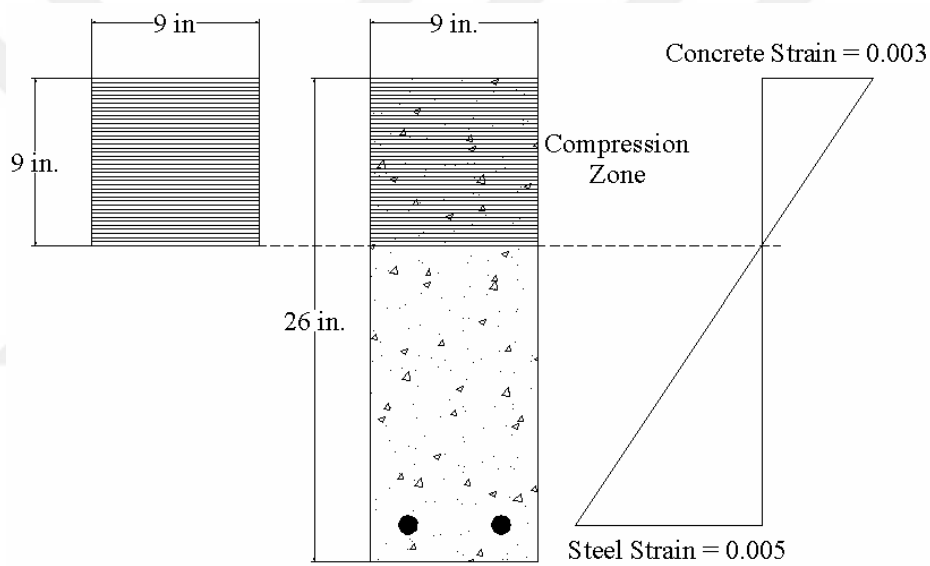


Figure 3-1 – Prototype Beam Member

Two pilot specimens were tested to evaluate the distribution of the forces from the steel arms to the concrete mid-section. The Pilot Specimen 1 was a 9×9×40 in. rectangular specimen with a total of twelve holes at the top and the bottom of the specimen. These holes facilitated the connection of the steel arms. A picture of the test set-up for Pilot Specimen 1 is presented in Figure 3-2.



Figure 3-2 – Test Set-Up for Pilot Specimen 1

After testing the Pilot Specimen 1, it was observed that the ends of the specimen were not strong enough to transfer both the main axial load and the moment applied by the arms. The lower end of the specimen failed after applying $1/8$ of the expected load. The failure mode at the lower end is shown in Figure 3-3.



Figure 3-3 – Failure Mode at the Lower End of Pilot Specimen 1

Rectangular steel tubes were used in the Pilot Specimen 2 at the ends. These tubes were both confining the concrete at the ends and transferring the moment in a more distributed way on the concrete surfaces. A picture for the test set-up for the Pilot Specimen 2 is shown in Figure 3-4.



Figure 3-4 – Test Set-Up for Pilot Specimen 2

The testing Pilot Specimen 2 was very successful in transferring the moment from the arms to the concrete specimen. Simultaneously, the application of the main axial load was achieved through the ends of the specimen. The failure occurred at the mid-height at the expected load level. Using the steel tubes at the ends was the decision that finalized the design of the test specimens.

3.2.2 Test Specimens

Test series consisted of 21 square column specimens with 9×9×40 in. dimensions. A general view of the concrete specimen is shown in Figure 3-5. The end sections of the specimens were heavily reinforced, while the test region in the middle of the specimens was plain concrete. The main parameter was the concrete compressive strength. Three different HSC mixture designs were used to cast the specimens. The target concrete compressive strengths of these mixtures at 28 days were 10, 14 and 18 ksi. Five, six and ten eccentric bracket specimens were cast for 10, 14 and 18 ksi target concrete compressive strength, respectively. Details of the test specimens are illustrated in Table 3-1.

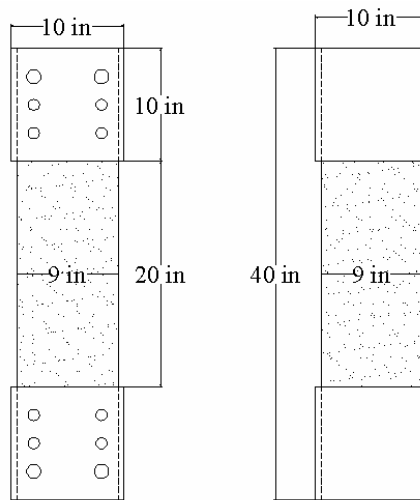


Figure 3-5 – General View of the Specimen

Table 3-1 – Details of the Test Specimens

Specimen No	Target Concrete Compressive Strength at 28 Days (ksi)
10EB1	10.0
10EB2	
10EB3	
10EB4	
10EB5	
10EB6	
14EB1	14.0
14EB2	
14EB3	
14EB4	
14EB5	
14EB6	
18EB1	18.0
18EB2	
18EB3	
18EB4	
18EB5	
18EB6	
18EB7	
18EB8	
18EB9	
18EB10	

3 Experimental Program

The end sections of the concrete specimens were reinforced with three #4 U-shaped longitudinal and three #3 transverse reinforcement. Steel reinforcement configuration of the eccentric bracket specimens is shown in Figure 3-6. Furthermore, the ends of the specimens were confined with 1/2 in. thick 10 in. high rectangular steel tubes. The combination of the steel tubes and heavy reinforcement facilitated proper transfer of the axial load and moment transfer and eliminates possible localized failures at the ends of the specimens. The plain concrete test section in the middle of the specimens was 16 in. long.

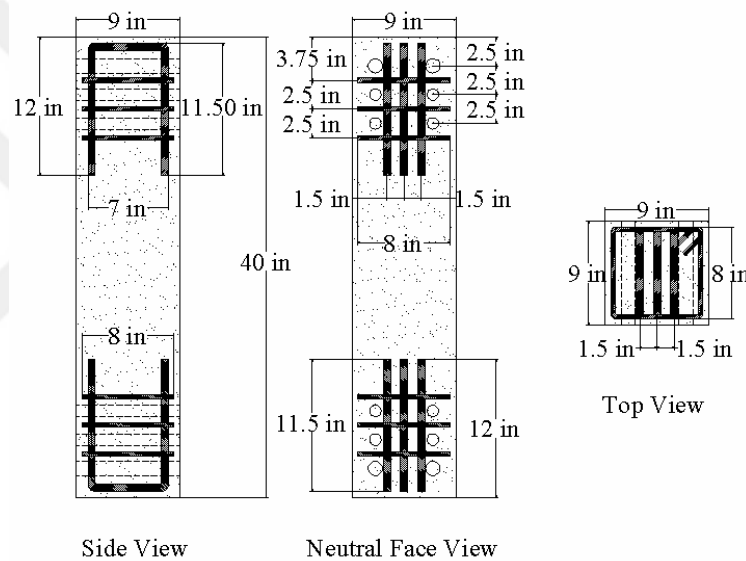


Figure 3-6 – Steel Reinforcement Configuration

3.2.3 Materials

3.2.3.1 Concrete

Three different mixtures were developed for 10 ksi, 14 ksi and 18 ksi target concrete cylinder strengths at 28 days (Logan 2005). In order to obtain these target strength levels, numerous trial batches were prepared in Constructed Facilities Laboratory (www.ce.ncsu.edu/centers/cfl) and in the laboratory of Ready Mixed Concrete Company (www.rmcc.info). After the desired strength levels were achieved from laboratory batches, these mixtures were made in large quantities in mixing trucks due to the large quantity of concrete needed to cast all the specimens.

3.2.3.1.1 Cement

Type I/II cement produced by Roanoke Cement Company (www.roanoke-cement.com) was used in all the mixtures used in this research.

3.2.3.1.2 Fine Aggregate

Two types of fine aggregate were used depending on the target compressive strength. The first type of fine aggregate was natural sand used by the Ready-Mixed Concrete Company (www.rmcc.info) in all of their concrete mixtures. The results of the sieve analysis performed at NCSU Constructed Facilities Laboratory (www.ce.ncsu.edu/centers/cfl) are given in Appendix B.

The second type of fine aggregate, 2MS concrete sand, was a manufactured sand produced by Carolina Sunrock Corporation (www.thesunrockgroup.com) and quarried in Butner, North Carolina. The material datasheet of fine aggregate provided by the company is given in Appendix B.

3.2.3.1.3 Coarse Aggregate

The coarse aggregate, #78M crushed stone, was also provided by Carolina Sunrock Corporation (www.thesunrockgroup.com) and quarried in Butner, North Carolina. The nominal maximum size of the coarse aggregate is 3/8 in. Additional information about the coarse aggregate can be found in the material datasheet provided in Appendix B.

3.2.3.1.4 Silica Fume

The densified Elkem Microsilica® 971 silica fume produced by Elkem Materials Incorporated (www.materials.elkem.com) was used in all the mixtures. Additional information about the silica fume is presented in Appendix B.

3.2.3.1.5 Fly Ash

Class F fly ash produced by Boral Material Technologies (www.boralmti.com) was used in the HSC mixtures. Material datasheet provided by the company is shown in Appendix B.

3.2.3.1.6 Admixtures

Both the high-range water-reducing and the retarding admixtures were manufactured by Degussa Admixtures Incorporated (www.degussa.com). The high-range water-reducing

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admixture (HRWRA) used was Glenium® 3030. The retarding admixture used was DELVO® Stabilizer. Additional information about these admixtures provided by the company is given in Appendix B.

3.2.3.1.7 Concrete Mix Proportions

After numerous trial batches, three mixture designs were selected to be used as 10, 14 and 18 ksi target concrete compressive strengths. These mixture designs are presented in Table 3-2.

Table 3-2 – Three Concrete Mixture Designs

	Target Concrete Compressive Strengths (ksi)		
Material	10.0	14.0	18.0
Cement (lbs/yd ³)	703	703	935
Silica Fume (lbs/yd ³)	75	75	75
Fly Ash (lbs/yd ³)	192	192	50
Fine Aggregate (lbs/yd ³)	1055 (NS*)	1315 (MS*)	1240 (MS*)
Coarse Aggregate (lbs/yd ³)	1830	1830	1830
Water (lbs/yd ³)	292	250	267
HRWRA (oz/cwt)**	17	24	36
Retarding Agent (oz/cwt)**	3	3	3
Water / Cementitious Material	0.30	0.26	0.25
28-Day Compressive Strength (ksi)	11.4	14.4	17.1

* NS: Natural Sand, MS: Manufactured Sand

** Ounces per 100 pounds of cementitious materials

3.2.3.1.8 Mechanical Properties

3.2.3.1.8.1 Cylinder Compression Test

Either three or four of 4×8 in. concrete cylinders were used to obtain the concrete compressive strength at the time of testing of the specimens. The cylinders tested at 28 days were cured in the lime water tanks continuously until the day of testing which was by ASTM standards for quality control testing in the concrete industry. Other cylinders were moist cured for the first 7 days with the test specimens. These test specimens and cylinders were then stored in the laboratory environment where the ambient temperature was approximately 72°F and the relative humidity was approximately 50 percent. The

ends of the cylinders were grinded before testing to failure to smooth the end surfaces. The cylinder test results for each of the specimens are presented in Appendix C.

3.2.3.2 Reinforcement

Grade 60 reinforcing steel provided by Gerdau Ameristeel Incorporated (www.gerdauameristeel.com) was used both for longitudinal and transverse reinforcement in the construction of the specimens.

3.2.4 Specimen Preparation

The steel tube sections were constructed by a local company, Hamilton Machine Works. Two of these steel tubes were used for each specimen to confine and have a better load transfer through the top and the bottom ends. The inner dimensions of the steel tube sections were 9×9×10 in. with a 1/2 in. steel plate thickness. The necessary ASTM A992 steel plates were welded together to construct the tube sections. Holes were drilled on two opposite faces to facilitate the connection of the moment arms. On one drilled side, there were total six holes, two of them for 1 and 1/4 in. threaded rod and four of them for 1 in. threaded rod.

The reinforcement cages at the ends were assembled outside the formwork. The PVC tubes were cut into necessary lengths to fit into the holes of the steel tube section. The steel tube section, reinforcement cage and PVC tubes are shown in Figure 3-7. The assembled reinforcement and PVC tubes were placed into the steel tube section. The assembly of the steel tube section is shown in Figure 3-8.



Figure 3-7 – Steel Tube Section, Reinforcement Cage and PVC Tubes



Figure 3-8 – Assembly of Steel Tube

The formworks were constructed from 3/4 in. plywood to enable repeated usage. Lumber stiffeners were placed at the backside of the plywood to eliminate the bulging effect due

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to the pressure of concrete at the casting stage. The formworks of the specimens were constructed so that the specimens would be cast vertically. Form release compound was applied on the inner surfaces of the formworks for easy stripping. Initially, the top and bottom steel tube sections were located in the formwork. Any voids between the formworks and the steel tube were sealed with silicone. The formwork was closed as a last step in the formwork preparation. These steps in the assembly procedure of the formwork are shown in Figure 3-9.



Figure 3-9 – Assembly of the Formwork

The concrete was prepared in the Ready Mixed Concrete Company's concrete mixing plant. Before mixing the ingredients of concrete, the moisture content of the fine and coarse aggregate was measured and the total amount of water in the mixture was adjusted accordingly. A total of 3 yd³ mixture was prepared to cast the test specimens. The slump, unit weight and air content tests on the concrete mixture were performed and compared with the lab batches. If the results of these tests were not compatible with the corresponding target strength of the lab batch, no specimens were cast with that concrete mixture. The specimens were cast directly from the concrete truck. A picture from the casting day is shown Figure 3-10. Scoops and trowels were used in the casting process. Three or four 4×8 in. companion cylinders were cast with each specimen. The specimens were vibrated by a 1 in. vibrator. The cylinders were cast in two layers and each layer was rodded 25 times with a 3/8 in. rod.



Figure 3-10 – The Casting Day

The surfaces of the concrete specimens exposed to air were covered with wet burlap and plastic sheets 2 hours after casting. The cylinder molds were capped, simultaneously. The specimens and the concrete cylinders were cured in the formworks and plastic molds for one day, respectively. The specimens and cylinders were then demolded and moist cured with wet burlap and plastic sheets until the 7th day. Starting from the seventh day, all the specimens and cylinders were air cured in the laboratory environment until the testing day. The ASTM cured cylinders were placed in the lime-water tank after demolding in the first day. These cylinders were moved out of the lime-water tank for compression tests on the 28th day. The specimens were at least 28 days old at the time of testing.

3.2.5 Test Set-Up

Two steel moment arms produced by Hamilton Machine Works were connected to the ends of the rectangular concrete specimen. Since the ends of the concrete specimen were confined with steel tube sections, the steel arms were directly connected to the steel tube sections. The connection was established using threaded rods through the holes on the specimen and the steel arms. The steel arms were designed to resist the maximum applied loads to fail the strongest specimen by a safety factor of 2 against yielding. ASTM A992

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steel was used in the construction of the arms. Two 9×24×1/2 in. plates were welded to two 24 in. long C 8×11.5 channel sections to obtain the moment arm. Stiffeners were used in every 6 in. to increase the shear strength of the arm. One inch thick end plate was welded to the built-up section to facilitate the connection to the concrete specimen. Stiffeners were welded between the built-up section and the end plate to increase the stiffness of the end plate. The drawing of the moment arm is shown in Figure 3-11. The general view of moment arm is shown in Figure 3-12.

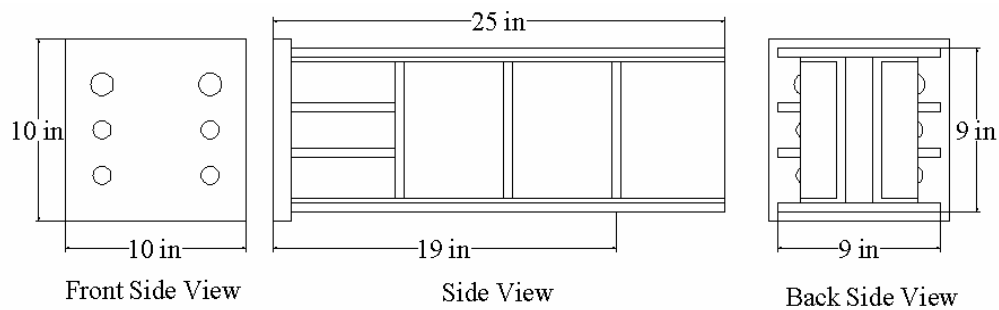


Figure 3-11 – Drawings of the Moment Arm

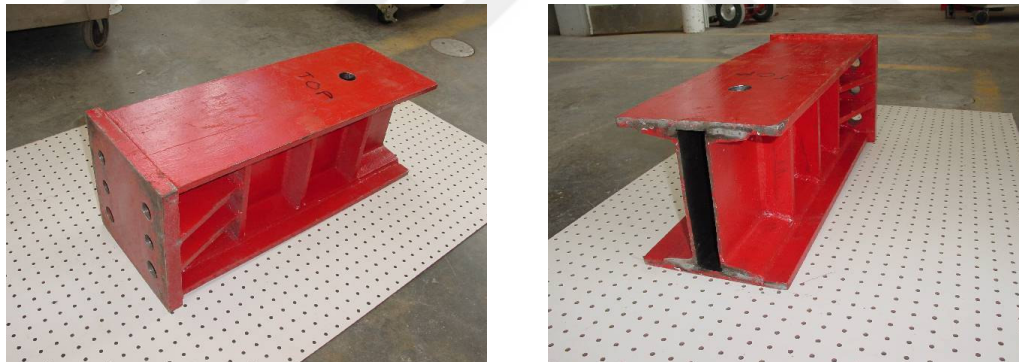


Figure 3-12 – General View of the Moment Arm

A total of six threaded rods are used at each end to connect of the moment arm to the concrete specimen. Two different sizes of Grade B16 alloy high strength steel threaded rods, 1 and 1.25 in, with 105 ksi yield strength and 125 ksi ultimate strengths, respectively, were used in three layers. The threaded rods, washers and nuts were provided by McMaster and Carr (www.mcmaster.com). The design for the threaded rods with a safety factor of 2 against yielding required to use two 1.25 in. threaded rods at the most stressed layer (furthest layer from the mid-section of the concrete specimen) and four 1 in. threaded rod at the other layers. The sizes of the threaded rods and their locations on the specimen are shown in Figure 3-13.

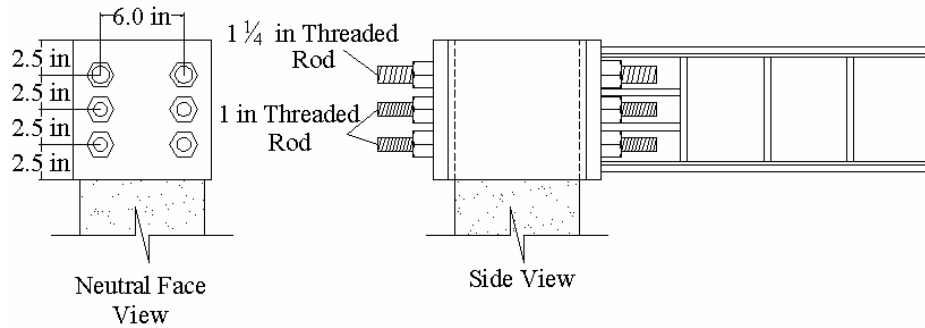


Figure 3-13 – Sizes and Locations of the Threaded Rods

Two roller bearing systems constructed by Hamilton Machine Works were used to eliminate the end restrictions on the specimen. The roller bearing system was designed to resist the maximum applied loads to fail the strongest specimen by a safety factor of two against yielding. ASTM A992 steel was used in the construction of the roller bearing system. One roller bearing system consisted of six rollers and two curved plates. The diameter of the each roller was 1 in. and 9 in. long. The bottom plate was 8×9 in. curved plate with 1 1/2 in. to 3/4 in. of tapering through outside. The radius of the tapering for the bottom curves plate was 11 in. The top plate was 8×9 in. curved plate with 3/4 in. to 1 27/64 in. tapering through inside. The radius of the tapering for the top curved plate was 12 in. According to these radii, the center of the applied axial loads was 9 1/4 in. from the top and the bottom of the specimens. After machining the steel plates and the rollers, they were all hardened. The rollers and the plates were fixed by using side plates and bolts for easy handling and assembly of the specimens in the compression machine. Before the testing the specimens to failure, the bolts of the side plates were removed. The cross-section of the roller connection is shown in Figure 3-14. General view of the roller connection is given in Figure 3-15.

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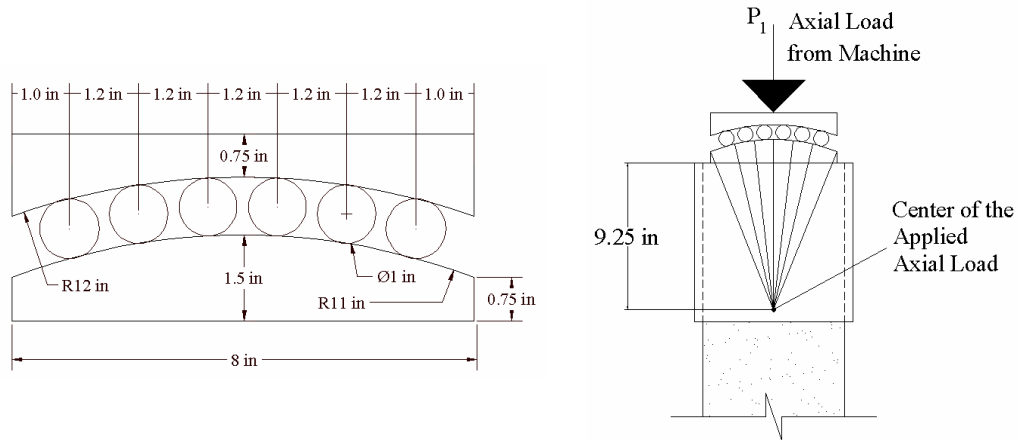


Figure 3-14 – Cross-Section of the Roller Connection

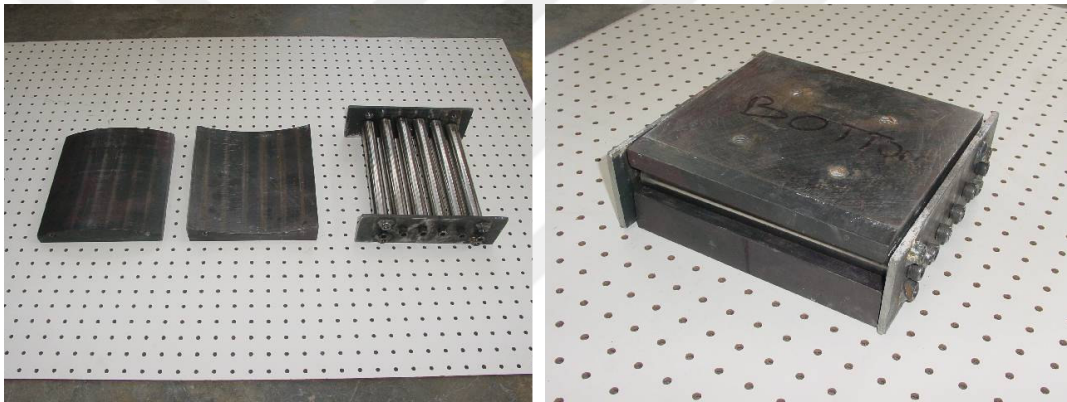


Figure 3-15 – General View of Roller Connection

The main axial load was applied using 2000 kip Baldwin-Lima-Hamilton compression machine. The rectangular concrete specimens with steel tubing sections were placed on the roller bearing system lying on the compression machine (Figure 3-16). At this stage the roller bearing system was fixed to the side plates to provide a stable system. Hydro-stone was used between the specimen and the roller bearing system both at the top and bottom to level the concrete specimen and to eliminate localized concrete failures at the ends. For a safety precaution, the specimen was surrounded with wooden frame to avoid any instability before and after testing the specimen to failure.



Figure 3-16 – Placement of the Specimen into the Compression Machine

Before connecting the moment arms, the hydrostone at the bottom of the specimen was let to set for at least 30 minutes. Initially the bottom moment arm was raised by using the crane up to the necessary location. The threaded rods were placed through the holes of the lower end of the specimen. On the opposite side of the connection, 10×10×1 in. plate was located to distribute the forces applied by the threaded rod more uniformly on the steel tube section. After positioning the bottom moment arm and adjusting its location with the crane, the arm was fixed by tightening the nuts of the threaded rods. At this stage the bolts fixing the roller bearing system was released and the concrete specimen fixed to the bottom lower arm was leveled by using the crane. After performing necessary leveling, lumber was placed under the moment arm to hold everything in position. Then the crane was detached from the bottom moment arm. The top moment arm was positioned in a similar manner. The connected moment arms to the concrete specimens are shown in Figure 3-17. The secondary load which created flexural effects on the specimen was applied using a 60 tons Enerpac hydraulic jack. A 1 in. diameter and 6 ft threaded rod was placed in the holes on the arms. The load cell and the jack were positioned on the top moment arm and both arms were connected together by using threaded rod. The test set-up is shown in Figure 3-18. A general view of the test set-up is illustrated in Figure 3-19.



Figure 3-17 – Connections of the Moment Arms

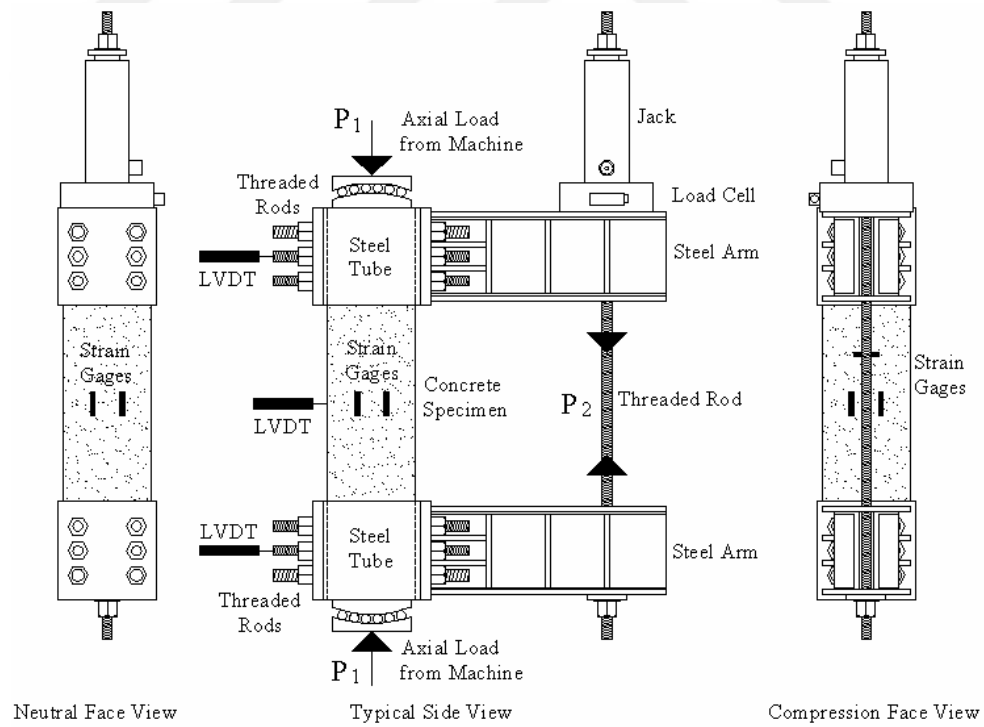


Figure 3-18 – Test Set-Up



Figure 3-19 – General View of the Test Set-Up

3.2.6 Instrumentation

The main axial load P_1 , applied using the 2000 kip Baldwin-Lima-Hamilton load controlled hydraulic compression machine, was measured using an internal load cell which is mounted in the machine. The load P_2 was applied with a 120000 lbs. Enerpac hydraulic jack and was measured using a 100000 lbs. Strainert Universal Flat Load Cell.

Companion 4×8 in. cylinders for the eccentric bracket specimens were tested using a 500-kip MTS compression machine and the corresponding load was measured using the internal load cell of the compression machine. Review of studies related to end treatments for tests of compression cylinders, grinding the ends of cylinders provided the highest

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strength and lowest coefficient of variation (Logan 2005). Therefore, both end surfaces of the companion cylinders were grinded to eliminate irregularities in the surfaces and to ensure that the ends were perpendicular to the sides of the specimen. The tests were performed in accordance with ASTM C 39 with a loading rate of 35 ± 7 psi/sec.

Each specimen was instrumented with PL-60-3L concrete electrical resistance strain gages supplied from Texas Measurements, Inc. (www.straingage.com). The length of the electrical resistance strain gages were 2.4 in. The concrete surface where the electrical resistance strain gages were attached was initially smoothed by using a rubbing-stone. The residual dust was cleaned by using pressurized air and water. After the concrete surface dried, the position of the electrical resistance strain gages were marked on all the sides and a very thin layer of 5 min epoxy was applied on the concrete surface to cover the holes on concrete and to have a smoother surface for the attachment of the electrical resistance strain gages. M-Bond 200 Adhesive, provided by Vishay Instruments Group (www.vishay.com), was used to bond the electrical resistance strain gages to the concrete surface. The application procedure followed to apply the electrical resistance strain gage with M-Bond 200 Adhesive Kit can be found in company's website. After the electrical resistance strain gages were applied on the concrete specimen, the adhesive was let to set for at least 2 minutes.

The number of strain gages attached on the concrete was determined after the pilot tests. In the pilot tests, 18 strain gages were mounted on concrete, four on each typical side face, 6 on compression face and 4 on neutral face. The location of the strain gages for pilot tests is presented in Figure 3-20. The vertical strain gages on the compression and the zero strain side were placed in three layers to find out whether there was any end effects due to confinement. The horizontal strain gages on the compression side were placed in two layers due to the same reason. After the pilot tests were performed, no significant end effects were observed on the strain gage readings. As a result, the number of strain gages was reduced to nine for the remaining test specimens.

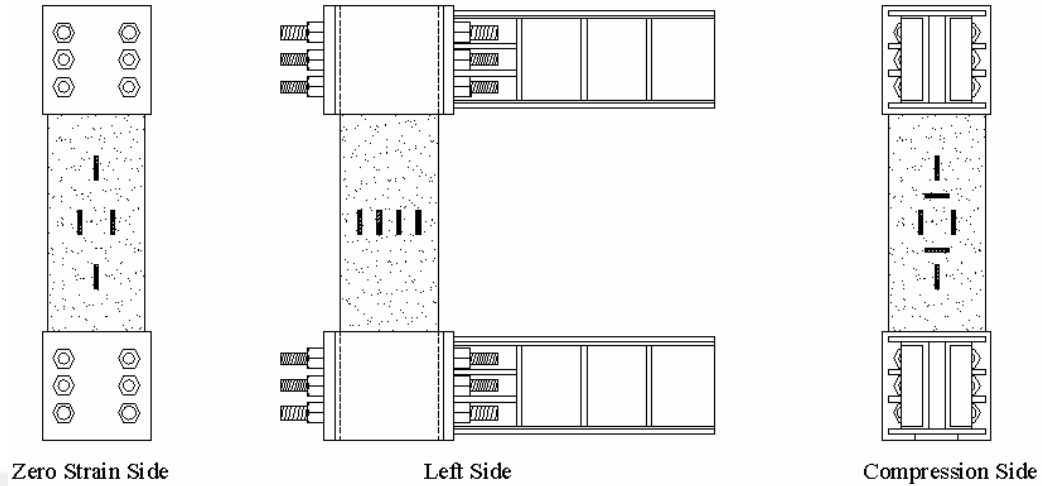


Figure 3-20 – Location of the Strain Gages for Pilot Tests

A total of 9 strain gages were located on each test specimen. Two of them were applied on the zero strain face. Four of them were mounted on the two sides of the specimen. Three of them were located on the compression side of the specimen, one of which was used to measure the transverse strain of concrete. The location of the strain gages for test specimens is presented in Figure 3-21.

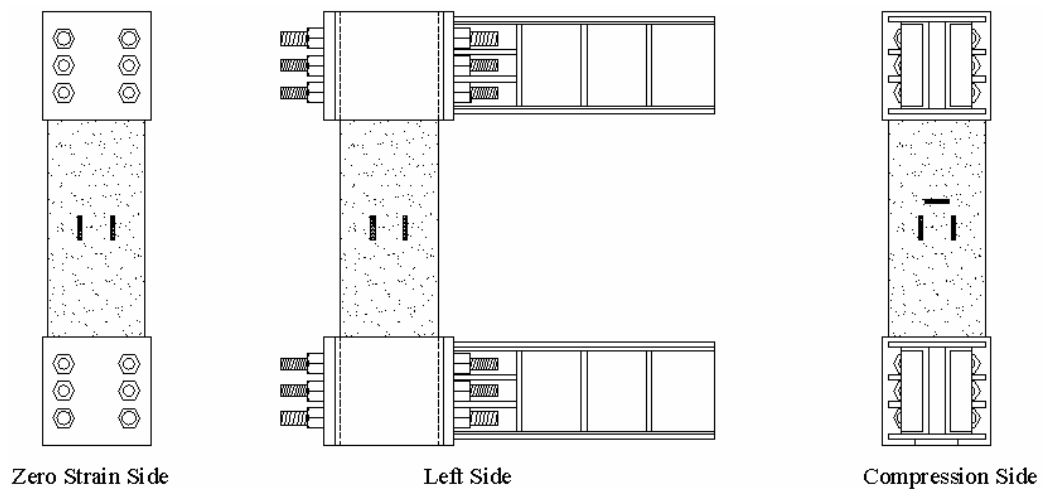


Figure 3-21 – Location of the Strain Gages for Test Specimens

The loads, P_1 and P_2 , compression machine stroke and 9 strain gages were recorded during the tests by a data acquisition system.

3.2.7 Test Procedure

The wires of the strain gages and LVDTs were connected to the data acquisition system. All the LVDTs were calibrated, located and adjusted at the predetermined positions. The load cell and stroke of the compression machine and the load cell on the steel moment arm was connected to the data acquisition system. The crane was attached to the top arm of the concrete specimen to remove the pieces of lumber under the moment arms. The specimen was leveled by using the crane again. The hydrostone and the top roller bearing system were placed at the top of the specimen. The bolts fixing the top roller bearing system was released and the hydrostone was left to set for a few minutes. Before the hydrostone was completely set, a very small amount of load was applied to the specimen. At this stage the level of the specimen was checked again. After the hydrostone was completely set, the readings of the instrumentations were balanced to zero and the compression machine was activated to apply the compressive load.

As the main axial load applied by the compression machine started to increase incrementally, the secondary load was applied by a hydraulic jack using a hand-pump to maintain neutral surface (zero strain) at one face of the specimen. It was intended to apply the load at a rate of approximately 2 microstrains per second at the other extreme compression face of each specimen. Each test was completed in approximately 25 minutes. The tests were stopped after the explosive failure of the concrete in the compression zone. The failed specimen was held by the safety frame after the end of the each test.

After each test, the safety frame was disassembled and the specimen was removed by the forklift. The compression machine, steel arms and the roller bearing system was then cleaned and prepared for the next test.

3.3 Creep Tests

Creep tests were performed to evaluate the creep behavior of HSC under sustained load. The main parameters included in the testing program were concrete compressive strength, curing type, age of concrete at the time of loading and the level of the sustained load. The testing scheme is presented in Table 3-3 and Table 3-4.

Table 3-3 – Testing Scheme for Cylindrical Creep Specimens

Rack ID	Target Concrete Compressive Strength (ksi)	Curing Type	Day of Loading (Days)	Applied Stress Level ($f'_c A_g$)
10Rack 1	10.0	1-Day Heat	1	0.2
10Rack 2		7-Day Moist	14	0.2
10Rack 3			28	0.2
10Rack 4			7	0.4
10Rack 5			14	0.4
10Rack 6			28	0.4
14Rack 1	14.0	1-Day Heat	1	0.2
14Rack 2		7-Day Moist	14	0.2
14Rack 3			28	0.2
14Rack 4			7	0.4
14Rack 5			14	0.4
14Rack 6			28	0.4
18Rack 1	18.0	1-Day Heat	1	0.2
18Rack 2		7-Day Moist	14	0.2
18Rack 3			28	0.2
18Rack 4			7	0.4
18Rack 5			14	0.4
18Rack 6			28	0.4

Table 3-4 – Testing Scheme for Cylindrical Shrinkage Specimens

Specimen ID	Target Concrete Compressive Strength (ksi)	Curing Type	Day of Initial Monitoring
10SC1	10.0	1-Day Heat	1
10SC2		7-Day Moist	7
14SC1	14.0	1-Day Heat	1
14SC2		7-Day Moist	7
18SC1	18.0	1-Day Heat	1
18SC2		7-Day Moist	7

Three different HSC mixture designs were used to cast the cylindrical creep and shrinkage specimens. The target concrete compressive strengths of these mixtures at 28

days were 10.0 ksi, 14.0 ksi and 18.0 ksi. Details about the mixture design are presented in Section 3.2.3.1.

Two different curing methods, 1-day heat and 7-day moist, were used for the creep specimens. The heat cured specimens were placed in an environmental chamber as soon as the specimens were cast. The internal temperature of the concrete was maintained between 150 and 160°F for 24 hours which simulated curing procedures of prestressed concrete plants. After the specimens were removed from the chamber, they were stored in the laboratory where the temperature was maintained at approximately 72°F and 50 percent relative humidity until the time of testing. The moist cured specimens were submerged in water in curing tanks for seven days as soon as they were demolded 24 hours after of casting. The water temperature was maintained at $73.5^{\circ}\text{F} \pm 3.5^{\circ}\text{F}$ using heaters equipped with adjustable thermostats. The water was saturated with calcium hydroxide to prevent leaching of calcium hydroxide from the test specimens. The curing tanks also contained pumps that circulated the water for a homogeneous mixture. The specimens were stored in the laboratory at approximately 72°F temperature and 50 percent relative humidity after 7 days.

The heat cured specimens were loaded on 1st day after casting. The moist cured specimens were loaded on the 7th, 14th and 28th day after casting. Two different load levels were applied on the specimens. $0.2 f'_c A_g$ and $0.4 f'_c A_g$. The load levels were calculated according to the target concrete compressive strengths at 28 days.

3.3.1 Test Specimens

Test series consisted of 36 cylinders with 4×12 in. dimensions loaded in 18 creep racks and 6 cylinders with 4×12 in. dimensions monitored for shrinkage purposes. Two specimens were loaded in each rack. PVC sewer pipe and end caps were used for the molds of the 4×12 cylinder specimens. PVC sewer pipe with 4 in. inner diameter was cut into 12 in. lengths. The end caps were glued at one end of each mold. The performance of these PVC pipes was outstanding even used repeatedly. Three pairs of brass insert were placed onto the molds to measure the creep strains. Before the casting day, form release agent was applied to the inner surface of the PVC molds. The brass insert were positioned afterwards.

3.3.2 Creep Racks

A typical creep rack is shown in Figure 3-22. Each rack consisted of two steel plates which were the upper base plate and the lower jacking plate. An upper jacking plate was also used at the time of loading the specimens which was removed after the specimens were loaded. 38 triangular steel plates, either 13×13×1 or 13×13×2 in, were used as the upper base plate, lower jacking plate and upper jacking plate. 54 rectangular steel plates with 4.5×4.5×0.5 in. dimensions were used as the lower base plate. These plates were constructed by a local company named as Hamilton Machine Works. Three, 1 in. diameter threaded rods were used for each of the 12 creep racks while three, 1.25 in. diameter threaded rods were used for the remaining 6 creep racks. These rods were B7 Alloy Steel, either 1 in. 8 thread or 1.25 in. 7 thread, 72 in. length, ASTM 193, plain finish threaded rods. Also Grade 8 plain steel hex nuts and zinc-plated steel USS high-strength flat washer were used as the necessary load application mechanism. The threaded rods, nuts and washers were supplied from McMaster and Carr (www.mcmaster.com). The configuration of the plates and thread rods in the racks are shown in Table 3-5.

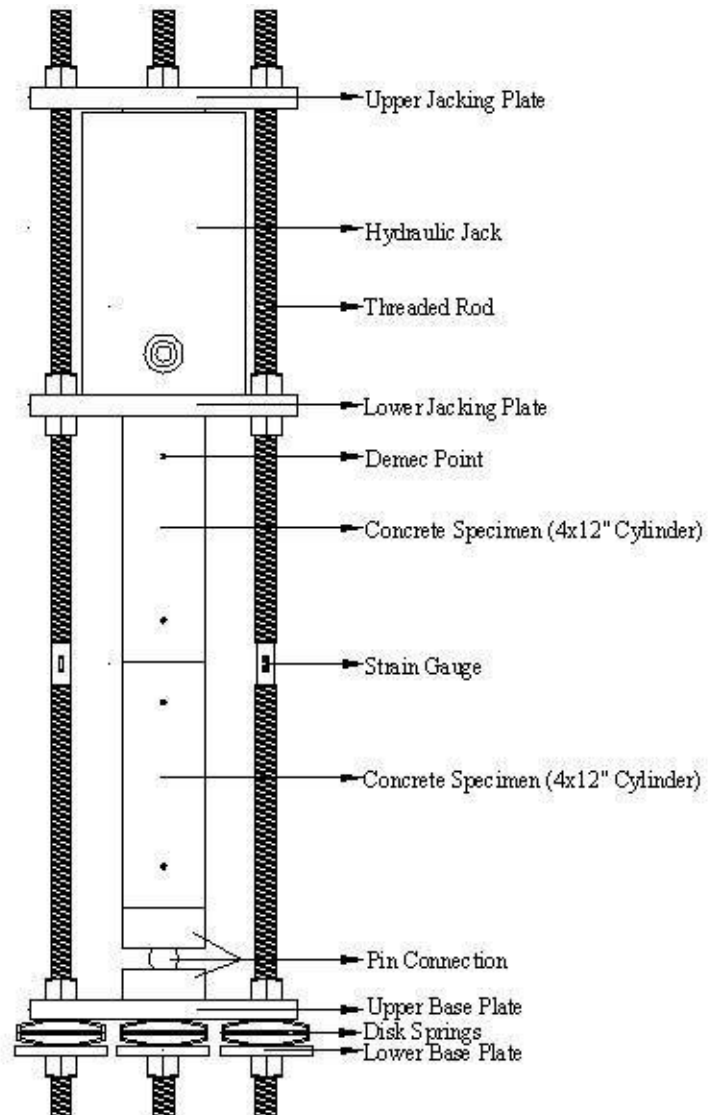


Figure 3-22 – Typical Creep Rack

Table 3-5 – Configuration of Plates and Threaded Rods in the Racks

Rack ID	Dimensions of Triangular Plate(in.)	Diameter of Threaded Rod (in.)
10Rack 1	13×13×1	1
10Rack 2		
10Rack 3		
10Rack 4		
10Rack 5		
10Rack 6		
14Rack 1	13×13×2	1.25
14Rack 2		
14Rack 3		
14Rack 4		
14Rack 5		
14Rack 6		
18Rack 1	13×13×1	1
18Rack 2		
18Rack 3		
18Rack 4	13×13×2	1.25
18Rack 5		
18Rack 6		

A pin connection was placed in each rack to provide concentric loading to the specimens. It consisted of two dipped-at-center steel plates with 4×4×1.5 in. and 4×4×2 in. dimensions. A 1.5 in. diameter ball bearing was placed into the dips. This was also constructed in Hamilton Machine Works. Each rack contained 3 pairs of K4250-M-375 high carbon and stainless steel disk springs supplied from Key Bellevilles, Inc (www.keybellevilles.com). The springs were used to compensate any load drops in the system. The springs in pairs were stacked in series at the bottom of the creep rack.

The assembly of the creep rack started with the application of the strain gages as presented in Figure 3-23. The other components of the racks (plates, disk springs and pin connection) are shown in Figure 3-24.



Figure 3-23 – Application of the Strain Gages



a) Plates



b) Disk Springs



c) Pin Connection

Figure 3-24 – Plates, Disk springs and Pin Connection

Threaded rods were placed in the holes on the lower jacking plate and upper base plate. The springs are positioned under the upper base plate and the bottom base plate was placed. The assembly of the plates is shown in Figure 3-25.



Figure 3-25 – Assembly of the Plates

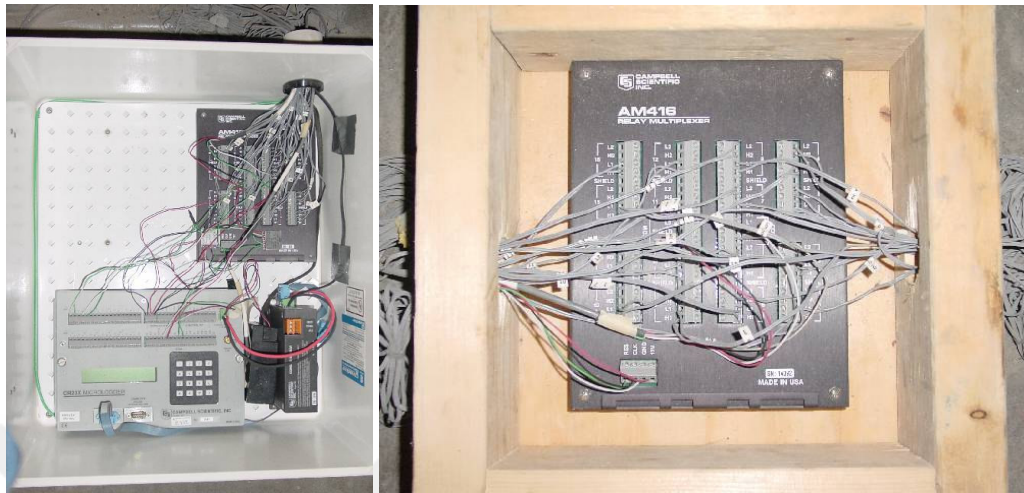
The creep racks were hinged to the walls to prevent any rolling over. Then the creep racks were erected side by side. The erection of the creep rack is shown in Figure 3-26.



Figure 3-26 – Erection of the Creep Rack

3.3.3 Instrumentation

Each threaded rod was instrumented with one YFLA-5-3L strain gage supplied from Texas Measurements, Inc (www.straingage.com). The gage factor and the resistance of these strain gages were 2.11 and 120.4 ohm, respectively. Two inches length of each threaded rod was cleaned from threads in Hamilton Machine Works. The strain gages were applied onto this cleaned portion of the threaded rod. The M-Bond 200 Adhesive Kit, supplied from Vishay Instruments, Inc. (www.vishay.com), was used to apply the strain gages. The application procedure of a strain gage onto a steel surface can be found in website www.vishay.com. These strain gages were connected to the CR23X-4M Datalogger which was supplied from Campbell Scientific, Inc. (www.campbellsci.com). The datalogger enabled continuous monitoring the loads on the threaded rods. To increase the number of strain gages that were connected to the datalogger, two AM416 Multiplexers were used. This enabled the datalogger to read 54 strain gages that were connected to the racks. Two MR1-350-127 Full Bridge Completion Modules supplied from Vishay Instruments, Inc. (www.vishay.com) were connected to the multiplexers to read the strain gages. The datalogger and the multiplexer are presented in Figure 3-27. The details on the program of the datalogger are presented in Appendix E.



a) Datalogger and Multiplexer

b) Multiplexer

Figure 3-27 – Campbell Scientific Datalogger and Multiplexer

Each of the concrete cylinders was instrumented using three pairs of demec points. To locate the demec points on the cylinder, the brass demec insert was bolted into the PVC mold by using a drill. These demec inserts were supplied from Humboldt Scientific, Inc. (www.humboldtmfg.com). The inserts were positioned in every 120° angle around the mold. The spacing between each pair of demec points was 8 in. The configuration of these demec points is shown in Figure 3-28. The cylindrical PVC molds and the insert are shown in Figure 3-29.

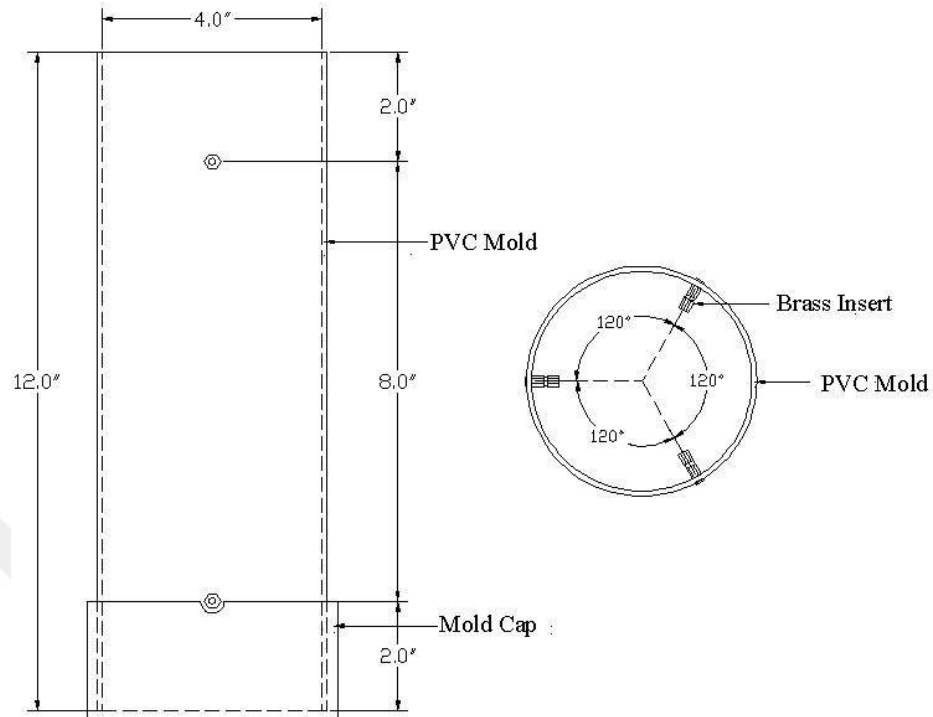


Figure 3-28 – Configuration of the Demec Points

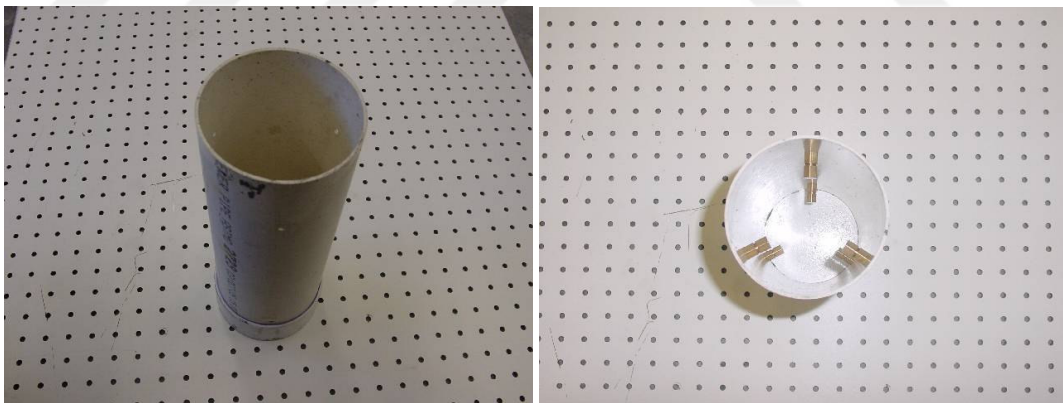


Figure 3-29 – PVC Mold and Demec Inserts

An 8 in. demec gage was used to monitor the strain in each specimen. The demec gage is presented in Figure 3-30. After all the demec inserts are located, the demec gage was used to control the spacing of the demec points.

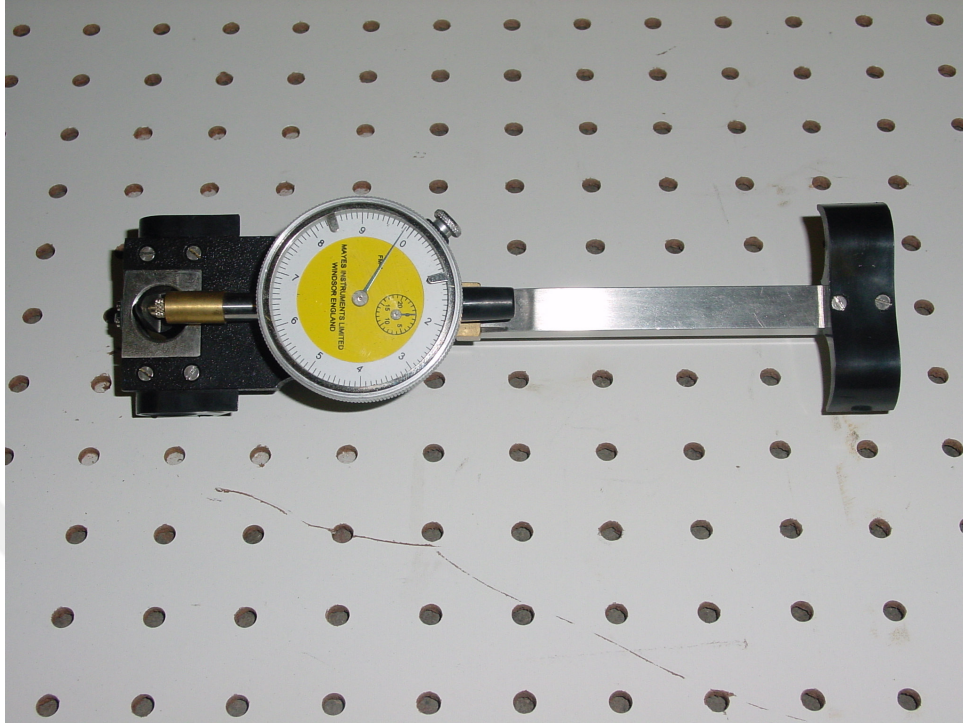


Figure 3-30 – Demec Gage

Creep tests were performed in an environment where the humidity could not be controlled. Therefore, the temperature and humidity were monitored during the experimental program using a hygro-thermometer clock supplied by Extech Instruments, Inc (www.extech.com) (Figure 3-31).



Figure 3-31 – Hygro-Thermometer Clock

The load on the specimens was applied using a 60 kip Enerpac hydraulic jack and manually controlled hand-pump. A pressure gage was used to measure the applied load. The pressure gage was calibrated each time before application of the load using 220 kip Universal MTS Testing Machine.

3.3.4 Test Procedure

For each concrete mixture design concrete mixture design, 14 cylinders were cast. Three of them were 1-day heat cured while the rest were 7-day moist cured. Two of the heat cured specimens were loaded in a rack. The other heat cured specimen was used to monitor the shrinkage of the heat cured cylindrical specimens. 10 of the moist cured specimens were loaded in the racks. The other moist cured specimen was used to monitor the shrinkage of the moist cured cylindrical creep specimens.

Both ends of the cylindrical creep and shrinkage specimens were grinded to remove the irregularities at the end surfaces and to ensure that the ends were perpendicular to the sides of the specimen. The decision in the selection of the end treatment was given based on the research performed by Logan (2005).

The two ends of the cylindrical shrinkage specimens were sealed with epoxy to simulate the same surface/volume ratio of the loaded creep cylinders. This prevented cylinders from exchanging moisture with the environment through their ends. Before placing the cylinders in the creep racks, each demec point on cylinders was numbered and the initial length was measured and recorded three times using a demec gage. Initial length of the cylindrical shrinkage specimen was also measured and recorded three times. The strain gages on the treaded rods were also recorded as the initial reading.

Two cylindrical creep specimens were stacked in series and positioned on the pin connection as shown in Figure 3-22. The pin connection and the cylindrical creep specimens were centered and aligned to avoid eccentric loading. The lower jacking plate was lowered and leveled to fix the specimens in the creep rack. The hydraulic jack was placed on the lower jacking plate and the upper jacking plate was positioned on the hydraulic jack. After fixing the necessary nut, 50 percent of the target load was applied on

3 Experimental Program

the specimens. At this stage, the forces in the steel threaded rods were monitored using the strain gages to ensure concentric loading of the specimens. If the load was not concentrically applied resulting differences in steel forces, the specimens would be unloaded. After repositioning, releveling and realigning the cylinders, 50 percent of the target load was applied again. When the desired distribution of the steel forces was achieved, the application of the load was continued up to the target load level. At this stage, the strain gages were recorded. The nuts above the lower jacking plate were tightened at this load level. The load applied by the jack was released and transferred to the lower jacking plate. The strain gages were checked to confirm that the target load level was applied on the specimens. The nuts were adjusted in case of a change of the load levels. The demec and strain gage readings were measured and recorded three times after achieving the target load level.

Creep and shrinkage of cylinders were simultaneously monitored at pre-determined intervals such as everyday in the first few days. On each monitoring day, every demec point was measured and recorded three times. The average of these three readings was used for calculation purposes. The monitoring period was increased to one month after three months. The specimens were monitored for approximately two years. The strain gages were monitored continuously using the datalogger. When the applied load dropped more than 5 percent of the target load, the specimens were loaded again to the original load level using the hydraulic jack.

3.4 Shrinkage Tests

Shrinkage tests were performed to evaluate the drying shrinkage behavior of HSC under drying conditions. The main parameters included in the testing program were concrete compressive strength, type of curing and day of initial monitoring. The testing scheme is presented in Table 3-6.

Table 3-6 – Testing Scheme for Prismatic Shrinkage Specimens

Specimen ID	Target Concrete Compressive Strength (ksi)	Curing Type	Day of Initial Monitoring
10SP1	10.0	1-Day Heat	1
10SP2			
10SP3			
10SP4		7-Day Moist	7
10SP5			
10SP6			
14SP1	14.0	1-Day Heat	1
14SP2			
14SP3			
14SP4		7-Day Moist	7
14SP5			
14SP6			
18SP1	18.0	1-Day Heat	1
18SP2			
18SP3			
18SP4		7-Day Moist	7
18SP5			
18SP6			

Three different HSC mixture designs were used to cast the prismatic shrinkage specimens. The target concrete compressive strengths of these mixtures at 28 days were 10.0 ksi, 14.0 ksi and 18.0 ksi. Details about the mixture design are presented in Section 3.2.3.1.

Two different curing methods, 1-day heat and 7-day moist, were used for the creep specimens. Details about the curing methods are presented previously in Section 1.1.

The 1-day heat cured and 7-day moist cured specimens were initially monitored in the first and the seventh day, respectively.

3.4.1 Test Specimens

Test series consisted of 18 prisms with $3 \times 3 \times 11\frac{1}{4}$ in. dimensions. The prismatic shrinkage specimens were designed and constructed in accordance with ASTM C 157, Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete. Prismatic steel molds used for casting are shown in Figure 3-32. Two inserts were embedded at the top and the bottom of each specimen in order to monitor the length change.



Figure 3-32 – Steel Mold for Prismatic Shrinkage Specimens

3.4.2 Instrumentations

A dial indicator system shown in Figure 3-33 was used to monitor the length change of each specimen. Before measuring the length of the concrete specimens each time, the dial indicator system was calibrated and reset using an invar rod.

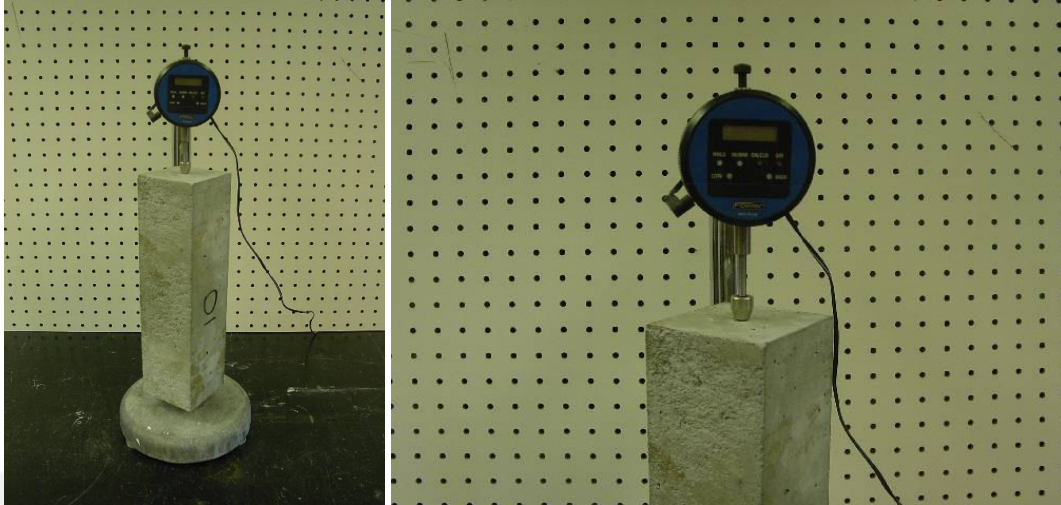


Figure 3-33 – Dial Indicator for Shrinkage Tests

3.4.3 Test Procedure

For each concrete mixture design, 6 prisms were cast. Three of them were 1-day heat cured while the rest were 7-day moist cured. The heat cured specimens were initially monitored on the first day whereas the moist cured specimens were initially monitored on the seventh day.

The dial indicator system was initially calibrated and reset using the invar rod. The prismatic shrinkage specimens were located in the dial indicator system using the inserts at the ends of the specimen.

Shrinkage of prisms was monitored at pre-determined intervals such as everyday in the first few days. On each monitoring day, the length change of the specimens was measured and recorded three times. The average of these three readings was used for calculation purposes. The monitoring period was increased to one month after three months. The specimens were monitored for approximately two years.

4 TEST RESULTS AND DISCUSSIONS

4.1 General

Test results presented in this chapter consists of results from 21 eccentric bracket, 36 creep and 24 shrinkage test specimens to evaluate the flexural, creep and shrinkage behavior of HSC, respectively.

4.2 Flexural Behavior

Eccentric bracket specimens were used to determine the stress-strain distribution of concrete with linear strain profile in the compression zone and to determine the parameters of the rectangular stress block. Details of the test set-up and test method are presented in Section 3.2.

A total of 21 specimens were used to evaluate the flexural behavior of HSC. Five, six and ten specimens were tested for 10, 14 and 18 ksi target concrete compressive strength, respectively. The concrete compressive strengths of cylinders at the time of testing varied from 10.4 to 16 ksi. Specimens with 10 ksi target concrete compressive strength were cast in two batches. The first three specimens (10EB1 to 10EB3) were cast as the first batch whereas the rest of the specimens (10EB4 and 10EB5) were cast as the second batch. Specimens with 18 ksi target concrete compressive strength were also cast in two batches. The first six specimens (18EB1 to 18EB6) were cast as the first batch and the rest of the specimens (18EB7 to 18EB10) were cast as the second batch. Entire specimens with 14 ksi target concrete compressive strength were cast in a single batch.

The ages of specimens at testing day are given in Table 4-1. The average of three- or four- 4×8 in. concrete cylinders used to obtain the concrete compressive strength at the time of testing of the specimens are tabulated in this table. Details about the cylinder compression tests are presented in Appendix C. This table also shows the rate of loading in microstrains per second at the extreme compression face of each specimen.

Table 4-1 – Tabulated Test Results for Eccentric Bracket Specimens

Specimen ID	Age of Specimen @ Test Day	f_c' (ksi) @ Test Day	Average Loading Rate ($\mu\text{E}/\text{sec}$)
10EB1	63	11.0	12.2
10EB2	109	11.4	2.0
10EB3	111	11.7	2.4
10EB4	63	10.4	2.1
10EB5	62	10.9	2.2
14EB1	49	14.6	2.3
14EB2	51	14.3	1.8
14EB3	52	14.7	2.2
14EB4	57	15.0	2.3
14EB5	100	15.4	5.3
14EB6	101	15.2	4.1
18EB1	76	15.8	2.2
18EB2	77	16.0	2.3
18EB3	81	15.6	2.4
18EB4	82	15.8	2.6
18EB5	83	16.0	2.1
18EB6	84	15.5	2.1
18EB7	96	15.0	2.5
18EB8	97	14.5	2.7
18EB9	99	14.9	2.2
18EB10	102	14.6	2.0

It was intended to apply the load at a rate of approximately 2 microstrains per second at the other extreme compression face of each specimen. However, 10EB1 was loaded inconsistently, since the specimen was the first specimens to be tested and there were not enough experience with the controlling unit of the compression testing machine. Other specimens were tested with a rate of loading reasonably close to the intended rate.

4.2.1 General Observations for Eccentric Bracket Specimen Tests

During the tests, no significant difficulty was observed in the application of the main and the secondary load. The main load was applied at a constant displacement rate throughout the testing. However, the rate of application of the secondary load varied depending on the concrete compressive strength of the specimen. Furthermore, the rate of application of the secondary load was constant when the behavior of the specimen was elastic; however, this rate increased rapidly as the specimen got closer to failure.

Similar behaviors were observed for all the specimens regardless of concrete compressive strength. The eccentric bracket specimens showed an explosive failure of the compression face. Large pieces of concrete exploded towards the surrounding area of the testing

4 Test Results and Discussions

machine. After examining the test specimens in details, it was observed that the failure surface was passing through the aggregates indicating the cement paste and the interface between the cement paste and aggregate were stronger than the aggregate itself.

LVDT measurements for all the test specimens did not show any significant change at the outer face which is intended to be the position of the neutral axis. However, these measurements were incorporated in the calculation of the lever arms of the main and the secondary loads for calculation purposes.

Failure modes of entire eccentric bracket specimens are shown in Figure 4-1 to Figure 4-21.



Figure 4-1 – Failure Mode of 10EB1



Figure 4-2 – Failure Mode of 10EB2



Figure 4-3 – Failure Mode of 10EB3



Figure 4-4 – Failure Mode of 10EB4



Figure 4-5 – Failure Mode of 10EB5



Figure 4-6 – Failure Mode of 14EB1



Figure 4-7 – Failure Mode of 14EB2



Figure 4-8 – Failure Mode of 14EB3



Figure 4-9 – Failure Mode of 14EB4



Figure 4-10 – Failure Mode of 14EB5



Figure 4-11 – Failure Mode of 14EB6



Figure 4-12 – Failure Mode of 18EB1



Figure 4-13 – Failure Mode of 18EB2



Figure 4-14 – Failure Mode of 18EB3



Figure 4-15 – Failure Mode of 18EB4

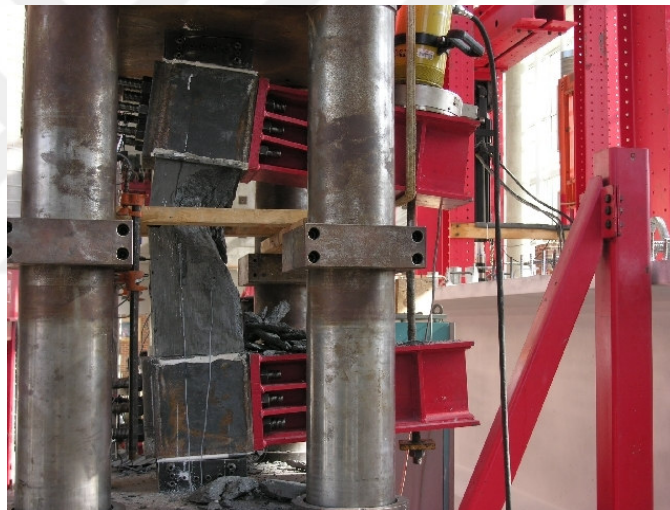


Figure 4-16 – Failure Mode of 18EB5



Figure 4-17 – Failure Mode of 18EB6



Figure 4-18 – Failure Mode of 18EB7



Figure 4-19 – Failure Mode of 18EB8



Figure 4-20 – Failure Mode of 18EB9



Figure 4-21 – Failure Mode of 18EB10

The detailed views of specimen 18EB6 are shown in Figure 4-22 and Figure 4-23. Similar behaviors were observed for other eccentric bracket specimens.



Figure 4-22 – Detailed Views of Specimen 18EB6



Figure 4-23 – Detailed Views of Specimen 18EB6

4.2.2 Surface Strain Measurements

During each test, the strain gages on all the surfaces were continuously monitored up to failure. The surface strain measurements versus applied main axial load of specimen 18EB4 are shown in Figure 4-24. Similar behaviors were observed for other eccentric bracket specimens.

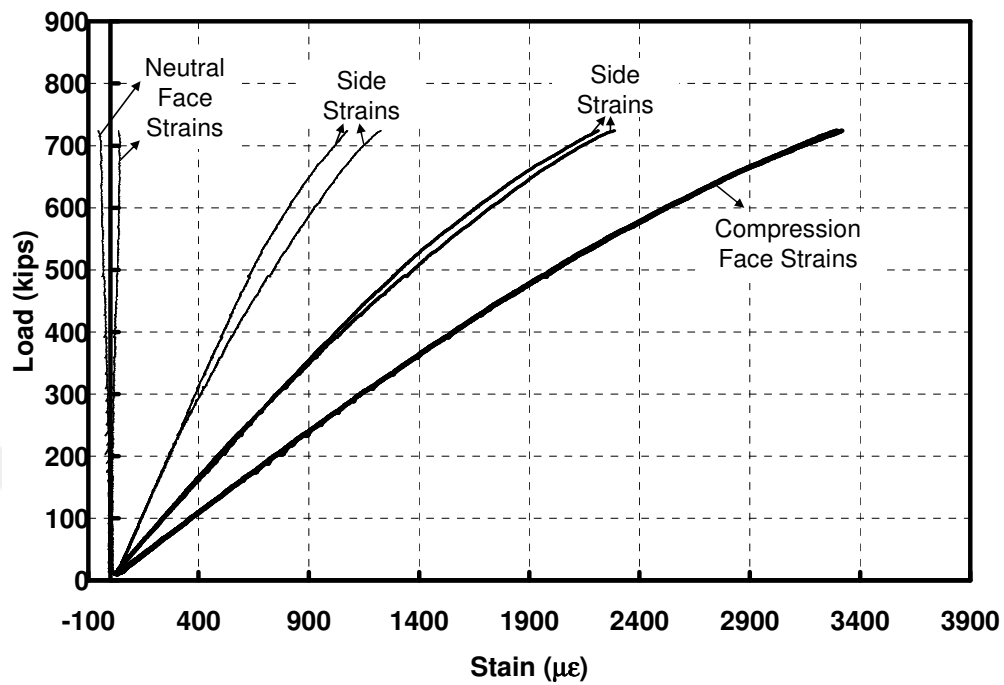


Figure 4-24 – Surface Strain Measurements vs. Applied Main Axial Load (18EB4)

The surface strain measurements at different loading stages of Specimen 18EB#2 are shown in Figure 4-25. Each bold point in the graph shows surface strain measurement at specific load level. The line connecting the points presents the measurements at the same load level. The graph proves the assumption that plane sections remain plane after deformation is valid for HSC.

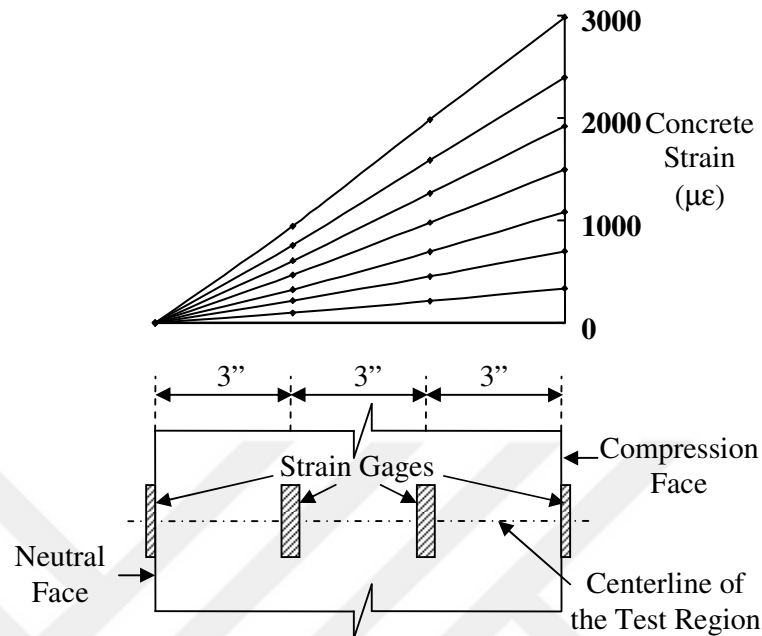


Figure 4-25 – Strain Distribution on Side Face of Specimen 18EB2

The average value of the two strain gages on the neutral and compression faces were used in all of the calculations. The average eliminated the effect of any loading eccentricity. No significant strain measurement was observed on the neutral face of the specimen. The strain gage measurements were in the range of ± 80 microstrains.

4.2.3 Ultimate Compressive Strain of Concrete

Ultimate compressive strains of concrete measured from eccentric bracket specimens tested in this research are tabulated in Table 4-2. These ultimate compressive strain values obtained from this research combined with the data from the researches in the literature are shown in Figure 4-26. The researches in the literature on ultimate compressive strain of concrete can also be found in Figure 2-23. Test results obtained from this research are compatible with the results in the literature. There is no significant trend of ultimate compressive strain as concrete compressive strength increases. Furthermore, it can be concluded from the graph that the lower bound for ultimate concrete compressive strain is independent of concrete compressive strength. Test results obtained from this research combined with the data in the literature were used to propose an ultimate compressive strain for HSC in flexure in Section 5.1.

Table 4-2 – Tabulated Results for Ultimate compressive strain of concrete

Specimen ID	f'_c (ksi) @ Test Day	Ultimate Strain ($\mu\epsilon$)
10EB1	11.0	3738
10EB2	11.4	3138
10EB3	11.7	3407
10EB4	10.4	3102
10EB5	10.9	3023
14EB1	14.6	3316
14EB2	14.3	3162
14EB3	14.7	3177
14EB4	15.0	3032
14EB5	15.4	2868
14EB6	15.2	2954
18EB1	15.8	3684
18EB2	16.0	3364
18EB3	15.6	2914
18EB4	15.8	3306
18EB5	16.0	3144
18EB6	15.5	3404
18EB7	15.0	3585
18EB8	14.5	3507
18EB9	14.9	3494
18EB10	14.6	3532

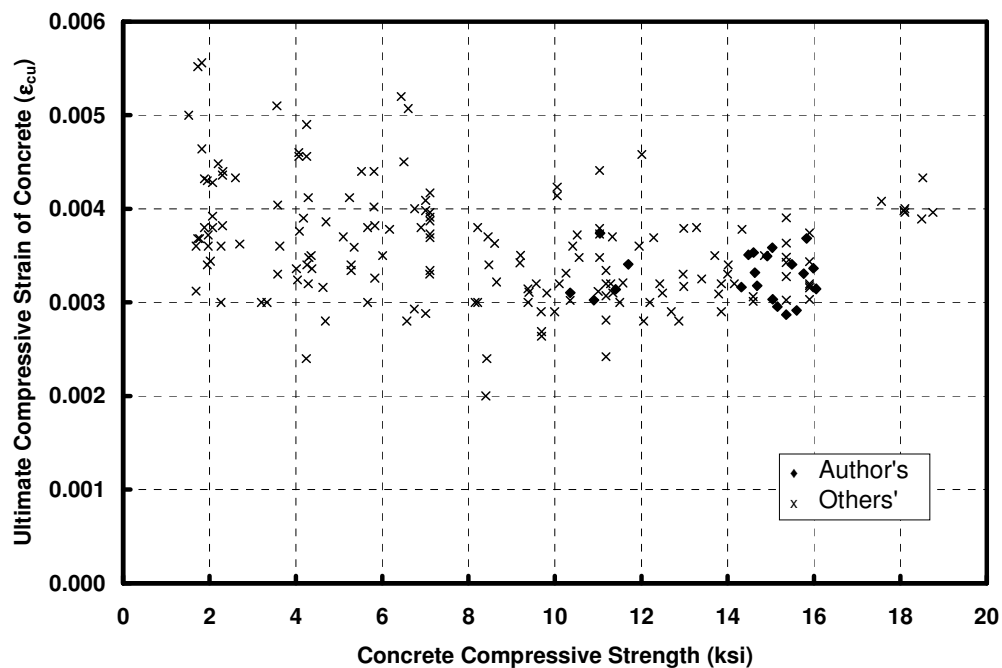


Figure 4-26 – Ultimate Compressive Strain of Concrete Obtained from This Research and Other Researches in Literature

4.2.4 Poisson's Ratio

Poisson's ratio was evaluated from the measured longitudinal compressive strains and their corresponding lateral tensile strains in the eccentric bracket specimens. There were three strains gages on the compression face of all eccentric bracket specimens, two in the longitudinal and one in the transverse direction. The measured strain obtained from transverse strain gage was divided by the average of the measured strains obtained from longitudinal strain gages to calculate Poisson's ratio for each specimen. The compression face of an eccentric bracket specimen is shown in Figure 4-27.



Figure 4-27 – Compression Face of Eccentric Bracket Specimen

The calculated Poisson's ratios using surface strain measurements obtained from eccentric brackets are tabulated in Table 4-3. Note that no result was shown for specimen 10EB1 due to some complications occurred in the readings of the transverse strain gage.

Table 4-3 – Tabulated Results for Poisson's Ratio

4 Test Results and Discussions

Specimen ID	f_c' (ksi) @ Test Day	Poisson's Ratio	Specimen ID	f_c' (ksi) @ Test Day	Poisson's Ratio
10EB2	11.0	0.25	18EB1	15.8	0.23
10EB3	11.4	0.21	18EB2	16.0	0.22
10EB4	11.7	0.19	18EB3	15.6	0.23
10EB5	10.4	0.20	18EB4	15.8	0.20
14EB1	14.6	0.22	18EB5	16.0	0.24
14EB2	14.3	0.20	18EB6	15.5	0.22
14EB3	14.7	0.23	18EB7	15.0	0.24
14EB4	15.0	0.23	18EB8	14.5	0.22
14EB5	15.4	0.24	18EB9	14.9	0.23
14EB6	15.2	0.23	18EB10	14.6	0.24

Poisson's ratios vs. longitudinal compressive strain for eccentric bracket specimens with 10, 14 and 18 ksi target concrete compressive strengths are shown in Figure 4-28 to Figure 4-30, respectively. The limits for the average compression face strain in these figures ranges from 200 to 1400 microstrains. The data below 200 microstrains showed a lot of fluctuations since the measurements were too small to compare. The data over 1400 microstrains exhibited high Poisson's ratios since the readings from the transverse strain gage included the effects resulting from the micro cracks. The micro cracks increased the readings of transverse strain gage producing unrealistic measurements for Poisson's ratio such as 0.5 and over.

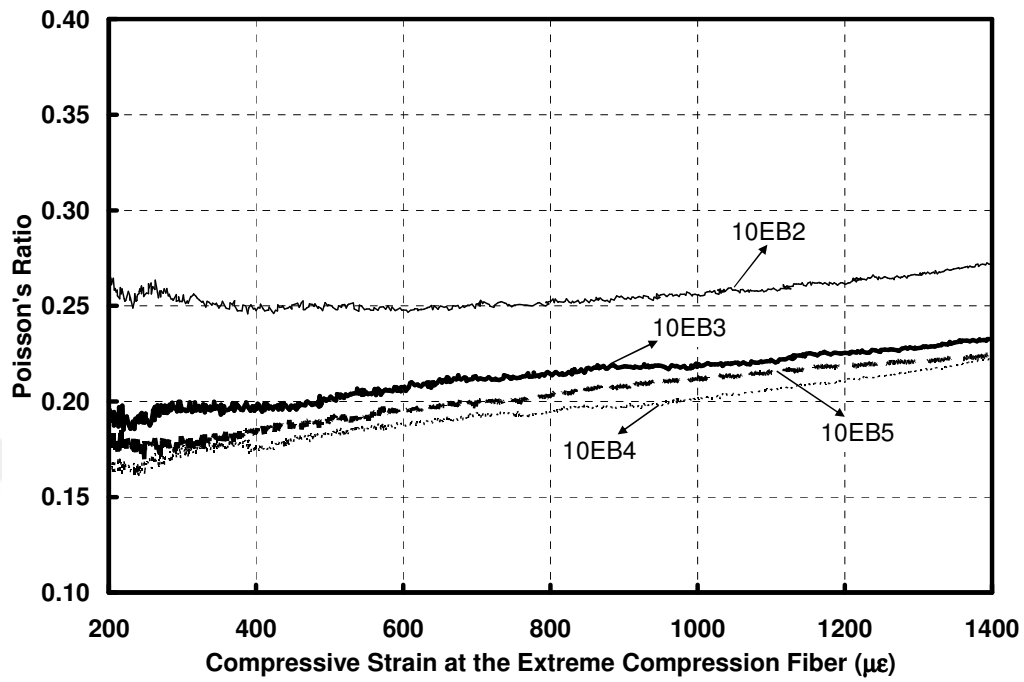


Figure 4-28 – Poisson's Ratio for Eccentric Bracket Specimens with 10 ksi Target Concrete Compressive Strength

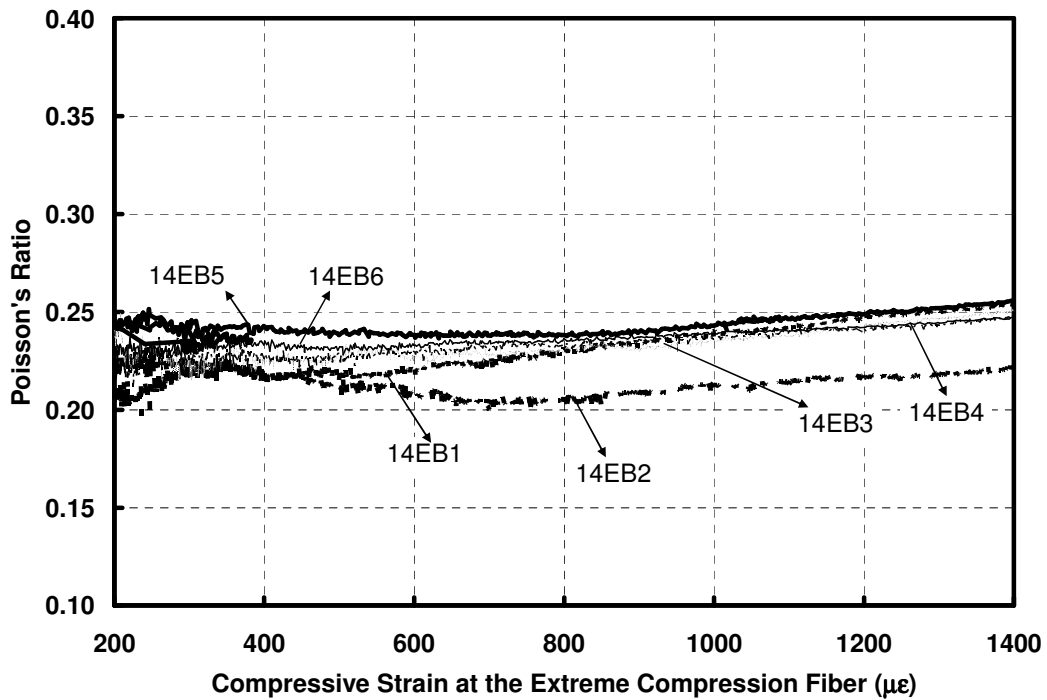


Figure 4-29 – Poisson's Ratio for Eccentric Bracket Specimens with 14 ksi Target Concrete Compressive Strength

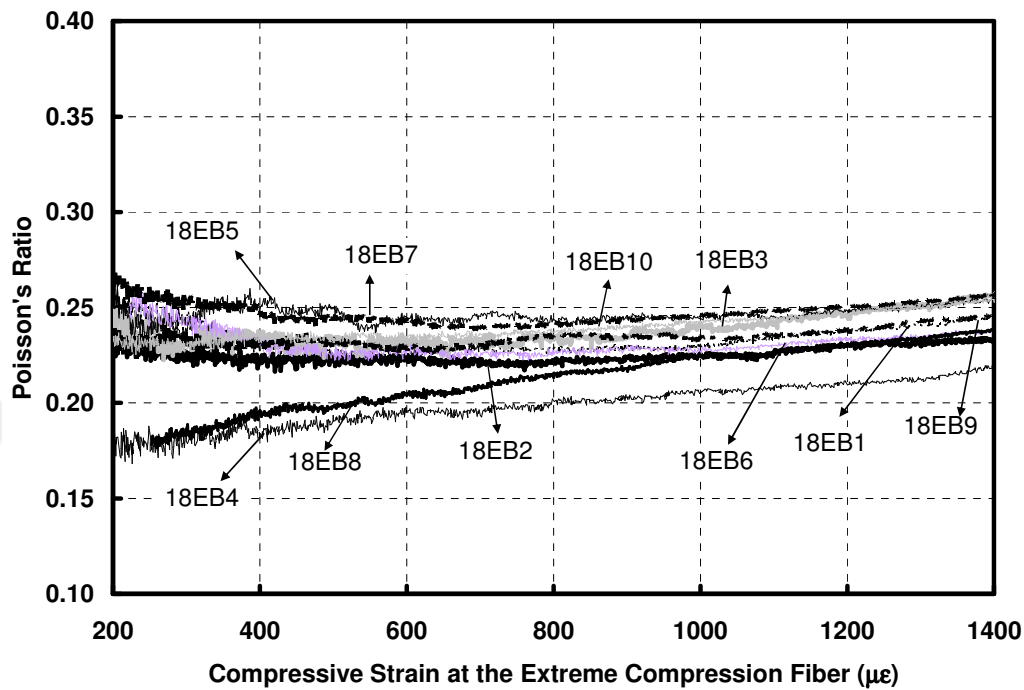


Figure 4-30 – Poisson's Ratio for Eccentric Bracket Specimens with 18 ksi Target Concrete Compressive Strength

Poisson's ratio values obtained from this research combined with the data from the researches in the literature are shown in Figure 4-31. The researches in the literature on Poisson's ratio can also be found in Figure 2-28. There is no apparent trend in Poisson's ratio as concrete compressive strength increases. Furthermore, the graph indicates that Poisson's ratio is independent of concrete compressive strength. Test results obtained from this research combined with the data in the literature were used to propose a Poisson's ratio for HSC in Section 5.3.

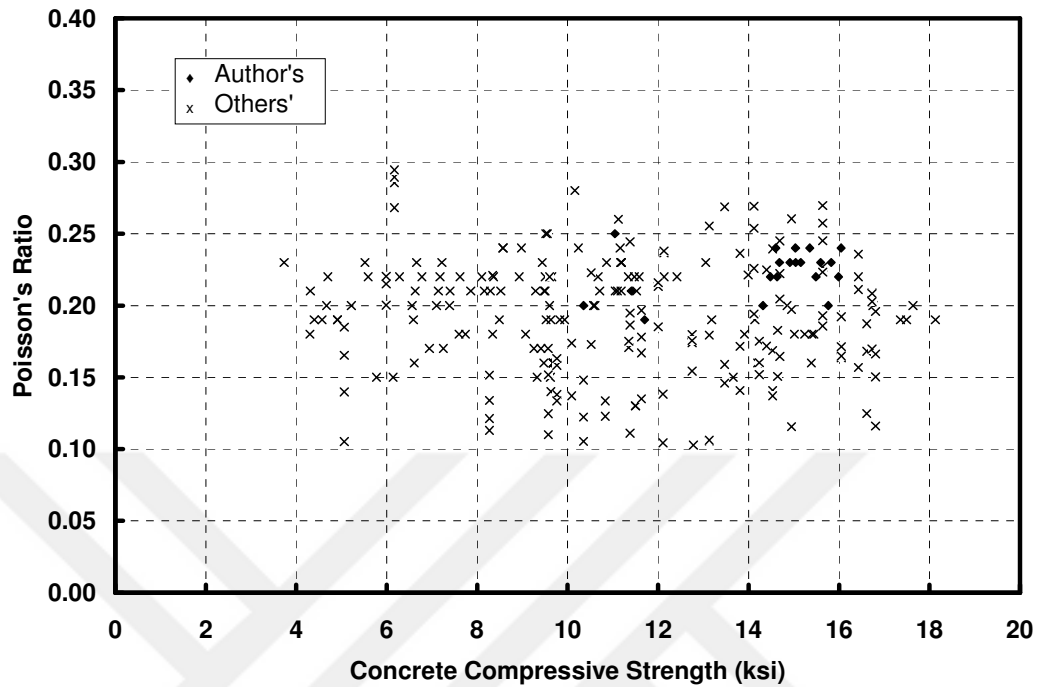


Figure 4-31 – Poisson's Ratio Obtained from This Research and Other Researches in Literature

4.2.5 Stress-Strain Distribution of Compression Zones

The approach presented by Hognestad et al. (1955) was used to determine the stress-strain relationship for each specimen. This approach assisted to calculate the concrete stress, f_c , as a function of measured strain at the most compressed fiber, ϵ_c , and the applied stresses f_o and m_o . The following equations were obtained from equilibrium of external and internal loads and moments. Note that the eccentricities due to deflection of the member were also considered in the calculation of applied moment, M .

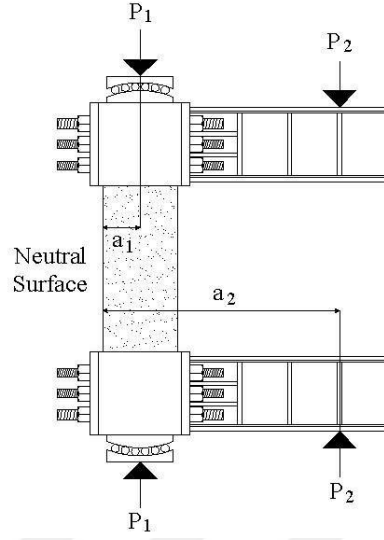


Figure 4-32 – Applied Forces on Eccentric Bracket Specimens

$$C = P_1 + P_2 = f_o bc = \frac{bc}{\epsilon_c} \int_0^{\epsilon_c} \sigma(\epsilon_x) d\epsilon_x \quad \text{Equation 4-1}$$

$$M = P_1 a_1 + P_2 a_2 = m_o bc^2 = \frac{bc^2}{\epsilon_c^2} \int_0^{\epsilon_c} \sigma(\epsilon_x) \epsilon_x d\epsilon_x \quad \text{Equation 4-2}$$

where, C is the total applied load, M is the total applied moment, P_1 is the main axial load, P_2 is the secondary load, a_1 and a_2 are the eccentricities, b is the width of the section, c is the depth of neutral axis,

$$f_o = \frac{P_1 + P_2}{bc} \quad \text{Equation 4-3}$$

and

$$m_o = \frac{P_1 a_1 + P_2 a_2}{bc^2} \quad \text{Equation 4-4}$$

are the applied stresses. Some of these definitions are presented in Figure A2-7.

4 Test Results and Discussions

Differentiating the last terms of the equations for C and M with respect to would ϵ_c would yield the following equations.

$$\sigma_c = \epsilon_c \frac{df_o}{d\epsilon_c} + f_o \quad \text{Equation 4-5}$$

$$\sigma_c = \epsilon_c \frac{dm_o}{d\epsilon_c} + 2m_o \quad \text{Equation 4-6}$$

Using these equations, two similar stress-strain relationships were obtained for each eccentric bracket specimens as shown in Figure 4-33. The average of these two relationships was used as the average stress-strain relationship of that specimen as shown in Figure 4-34.

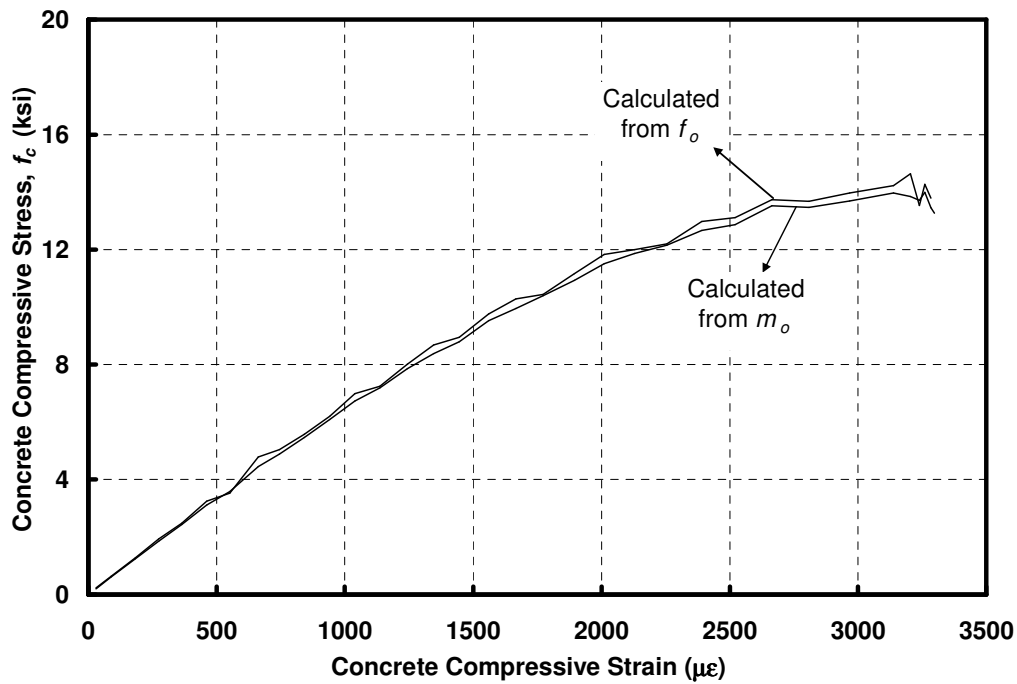


Figure 4-33 – Two Similar Stress-Strain Relationships (18EB4)

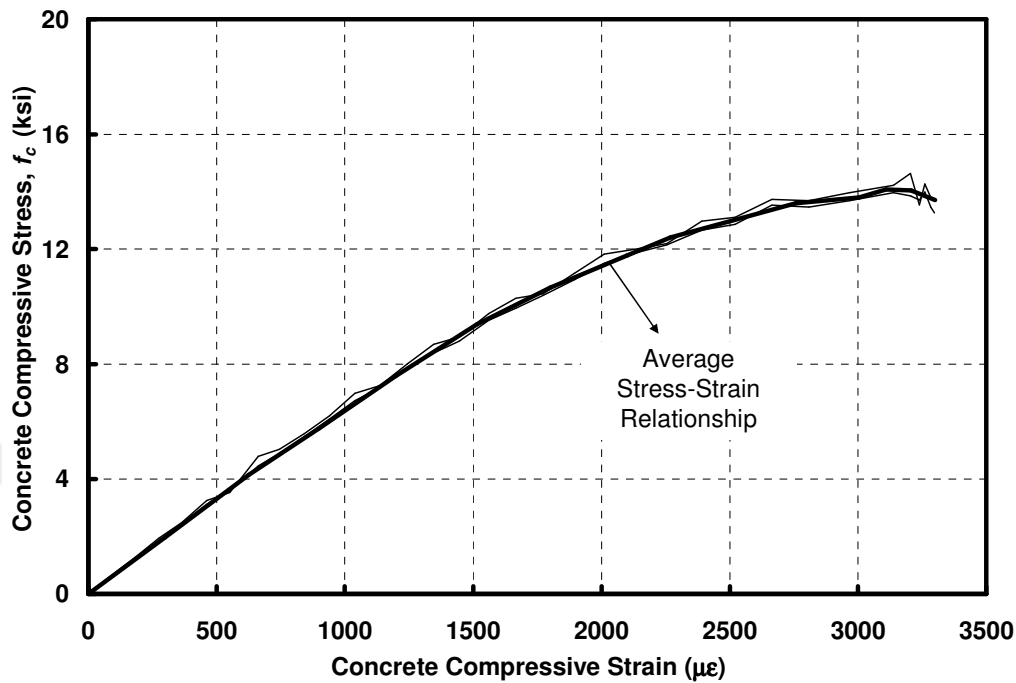


Figure 4-34 – Average Stress-Strain Relationship (18EB4)

The average stress-strain relationships of the specimens with 10, 14 and 18 ksi target concrete compressive strengths are shown in Figure 4-35 to Figure 4-37. The numerical values of the simplified stress-strain relationships for all the specimens are given in Appendix F. The stress-strain relationships obtained from this research indicate that as concrete compressive strength increases, the strain at the peak stress increases and gets closer to 0.003. The shape of the ascending branch of the stress-strain relationship becomes more linear and steeper, and the slope of the descending part also becomes steeper. It can be concluded that the descending branch of the stress-strain relationship for HSC is so hard to capture since, after peak stress, HSC fails suddenly with an explosive manner. The stress-strain relationships obtained from this research were used to propose and validate an analytical stress-strain relationship for HSC in Section 5.2.

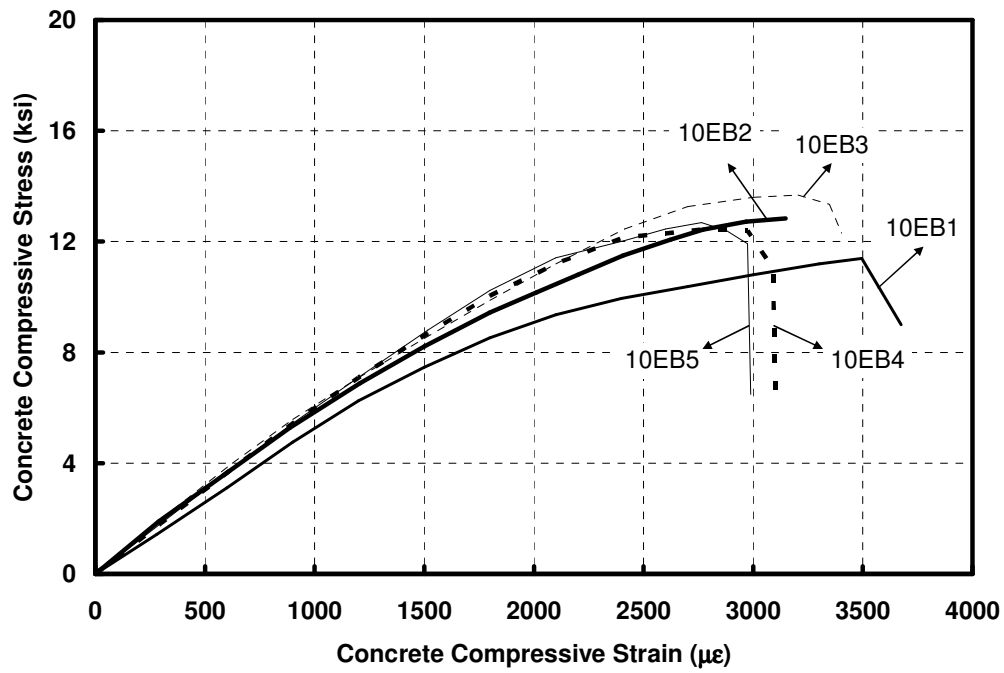


Figure 4-35 – Stress-Strain Relationships for Specimens with 10 ksi Target Concrete Compressive Strength

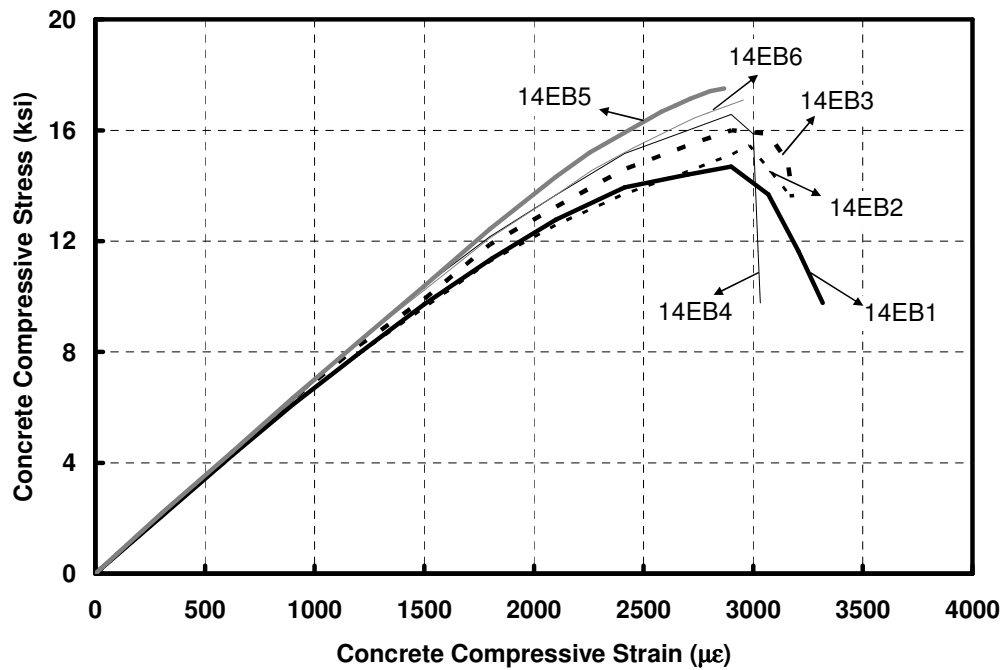


Figure 4-36 – Stress-Strain Relationships for Specimens with 14 ksi Target Concrete Compressive Strength

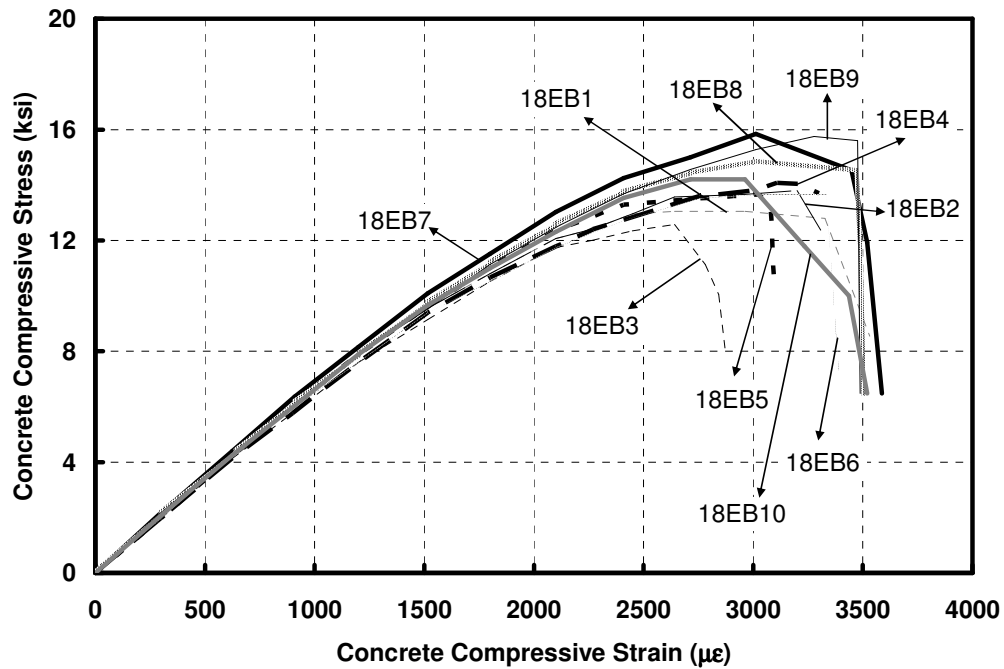


Figure 4-37 – Stress-Strain Relationships for Specimens with 18 ksi Target Concrete Compressive Strength

4.2.6 Stress Block Parameters

Stress-strain relationships obtained from eccentric bracket specimens were used to calculate the generalized and rectangular stress block parameters for HSC. Details of this calculation are given in Section 2.2.1. These parameters for concrete compressive strengths ranging from 10.4 to 16 ksi are tabulated in Table 4-4.

Table 4-4 – Calculated Stress Block Parameters for Eccentric Bracket Specimens

Specimen ID	f_c' (ksi) @ Test Day	k_1	k_2	k_3	α_1	β_1
10EB2	11.0	0.65	0.38	1.03	0.90	0.75
10EB3	11.4	0.62	0.36	1.12	0.95	0.72
10EB4	11.7	0.65	0.36	1.14	1.02	0.73
10EB5	10.4	0.64	0.36	1.20	1.06	0.73
10EB6	10.9	0.62	0.36	1.16	1.01	0.72
14EB1	14.6	0.63	0.37	1.00	0.85	0.74
14EB2	14.3	0.60	0.36	1.08	0.85	0.72
14EB3	14.7	0.61	0.36	1.09	0.93	0.71
14EB4	15.0	0.58	0.35	1.10	0.92	0.70
14EB5	15.4	0.57	0.34	1.10	0.92	0.68
14EB6	15.2	0.60	0.35	1.06	0.91	0.69
18EB1	15.8	0.69	0.38	0.82	0.74	0.77
18EB2	16.0	0.67	0.37	0.85	0.77	0.74
18EB3	15.6	0.63	0.37	0.81	0.69	0.73
18EB4	15.8	0.65	0.36	0.88	0.78	0.73
18EB5	16.0	0.65	0.36	0.85	0.76	0.72
18EB6	15.5	0.66	0.37	0.88	0.78	0.74
18EB7	15.0	0.64	0.37	1.05	0.90	0.75
18EB8	14.5	0.65	0.37	1.03	0.91	0.74
18EB9	14.9	0.62	0.36	1.06	0.91	0.72
18EB10	14.6	0.64	0.38	0.97	0.82	0.77

These stress block parameters obtained from this research combined with the data from the researches in the literature are shown in Figure 4-38 to Figure 4-44. The researches in the literature on stress block parameters can be found in Figure 2-16 to Figure 2-22.

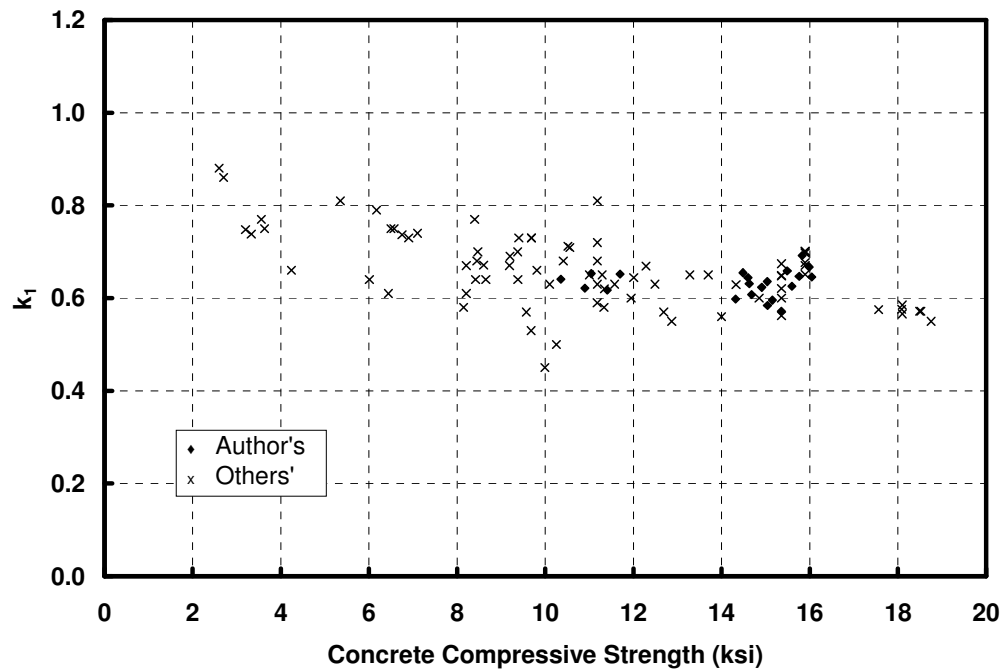


Figure 4-38 – k_1 Values Obtained from This Research and Other Researches in Literature

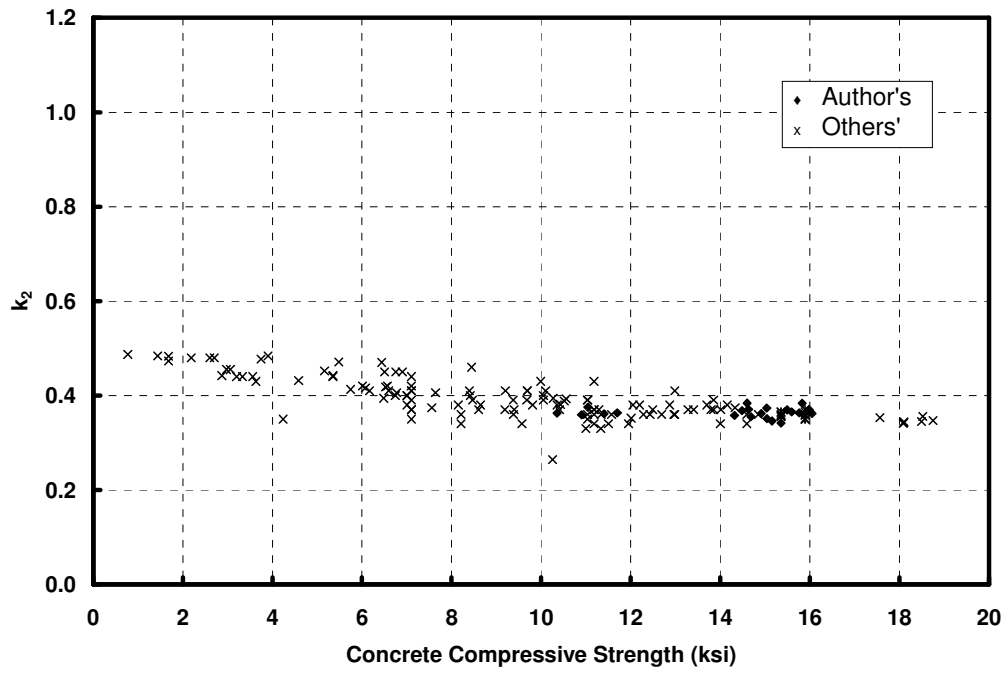


Figure 4-39 – k_2 Values Obtained from This Research and Other Researches in Literature

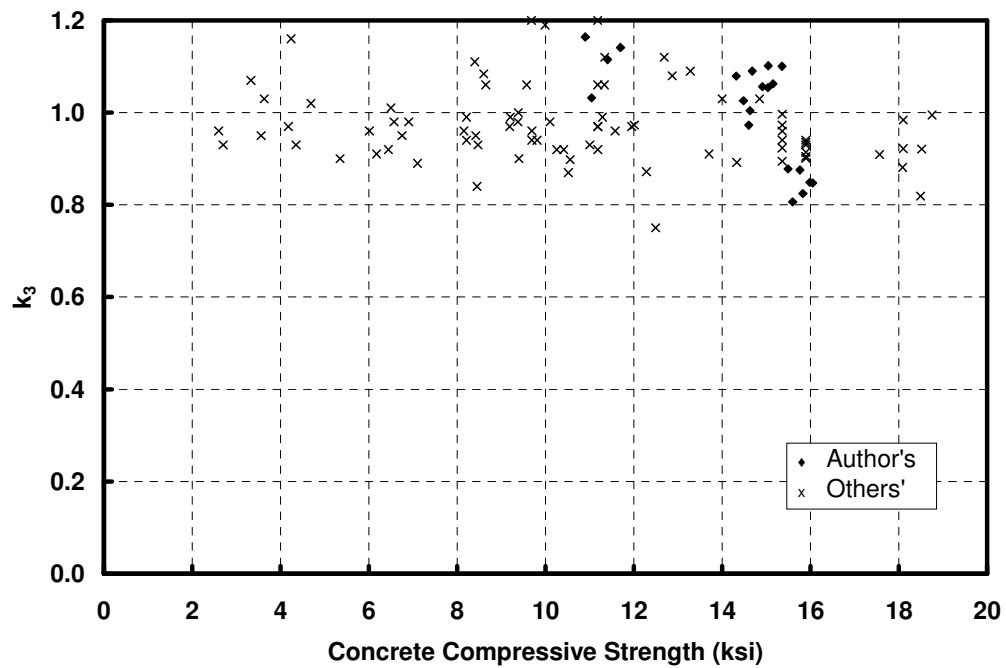


Figure 4-40 – k_3 Values Obtained from This Research and Other Researches in Literature

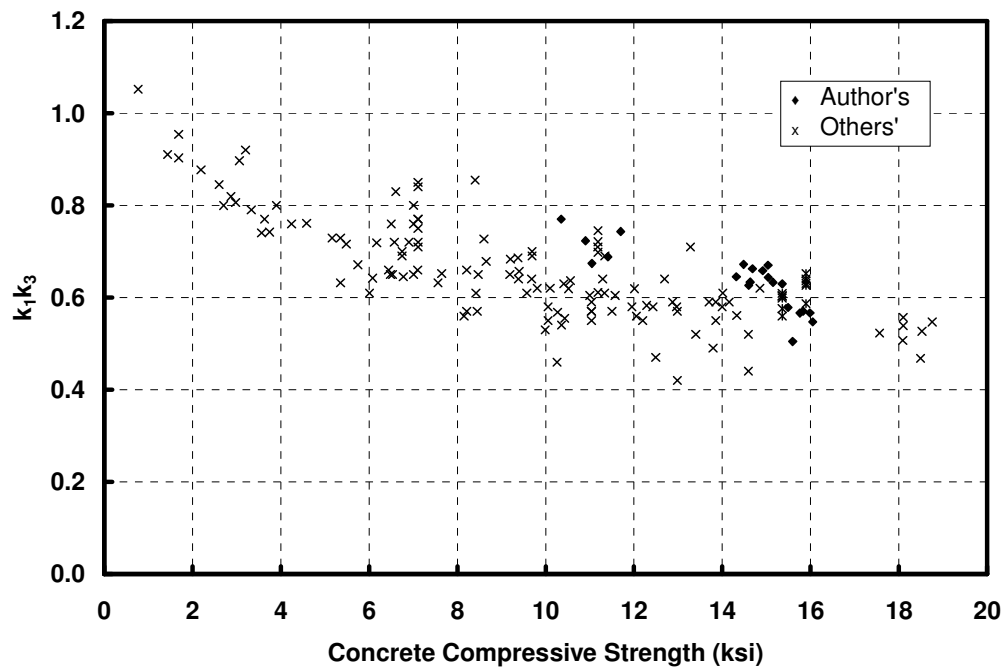


Figure 4-41 – k_1k_3 Values Obtained from This Research and Other Researches in Literature

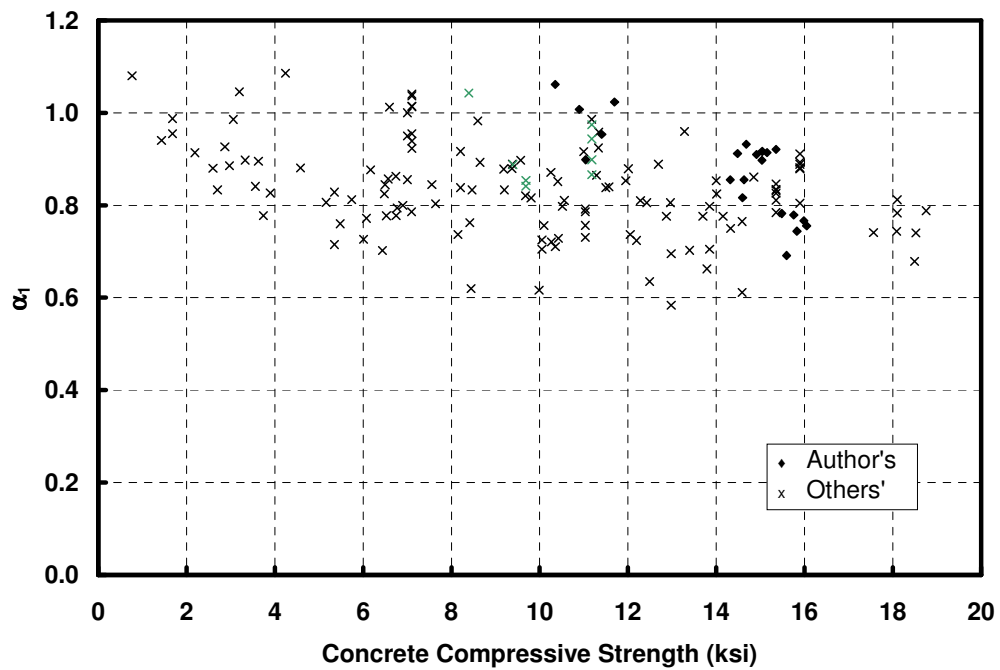


Figure 4-42 – α_1 Values Obtained from This Research and Other Researches in Literature

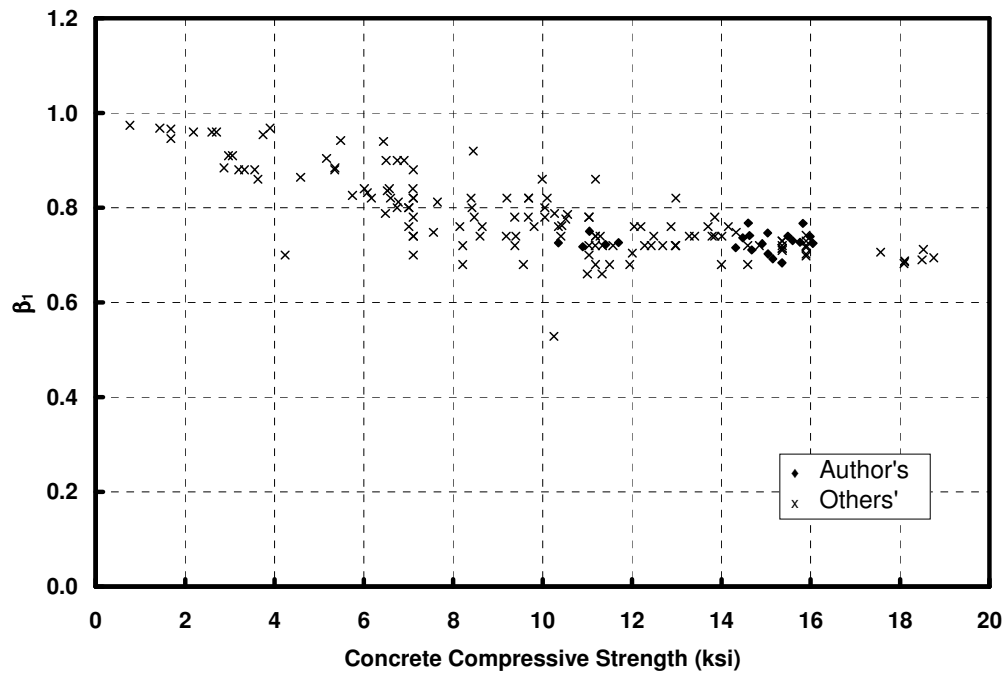


Figure 4-43 – β_1 Values Obtained from This Research and Other Researches in Literature

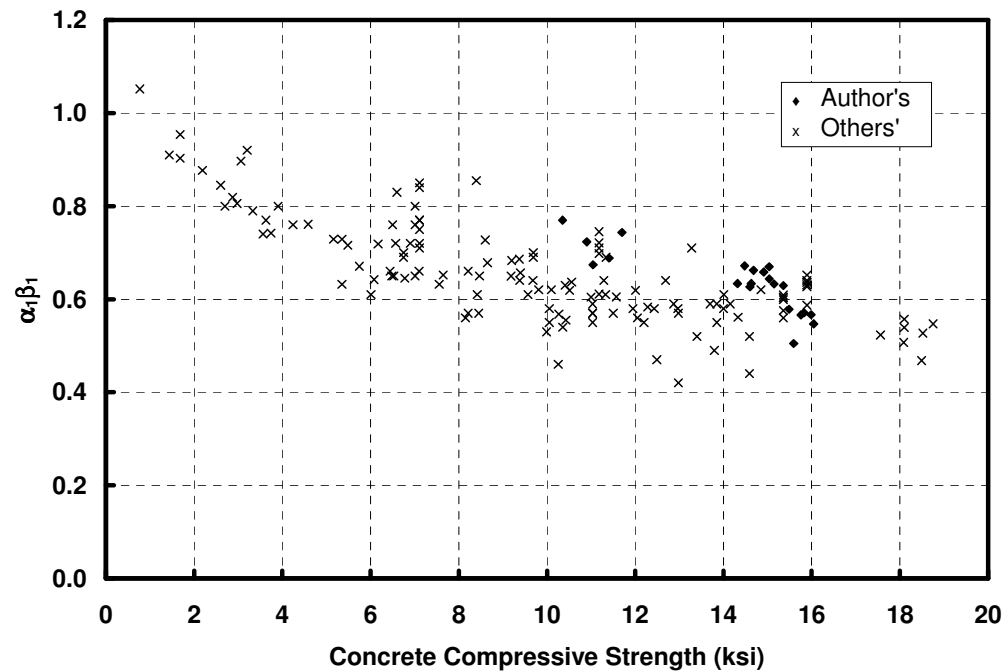


Figure 4-44 – $\alpha_1\beta_1$ Values Obtained from This Research and Other Researches in Literature

4 Test Results and Discussions

Test results indicate that the generalized stress block parameter k_1 is rarely less than 0.58 when concrete compressive strengths varied between 10 and 18 ksi. The generalized stress block parameter k_2 is found to be compatible with the tests results in literature. The generalized stress block parameter k_3 for HSC is determined to be similar to NSC. It can be concluded that as concrete compressive strength increases over 10 ksi, the rectangular stress block parameters α_1 decreased. However, the rectangular stress block parameters β_1 does not seem to be affected for variations in the concrete strength over 10 ksi.



4.3 Creep Behavior

A total of (36) 4×8 in. cylindrical specimens were used to evaluate the creep behavior of HSC. Specimens of each target concrete strength were cast in one batch. Twelve cylinders were loaded in 6 creep racks for 10, 14 and 18 ksi target concrete compressive strengths. The concrete compressive strengths of 4×8 in. cylinders at 28 days varied from 10.4 to 16.7 ksi. Test set-up and test method are presented in Section 3.3. Details about the creep tests are given in Table 4-5.

Table 4-5 – Details about Creep Tests

Rack No	Target Concrete Compressive Strength (ksi)	Curing Type	Concrete Compressive Strength @ 28 days (ksi)	Day of Loading (days)	Concrete Compressive Strength @ Day of Loading (ksi)	Applied Stress (ksi)
10Rack1	10.0	1-Day Heat	10.4	1	9.6	2
10Rack2		7-Day Moist	12.1	14	10.8	
10Rack3				28	12.1	
10Rack4				8	8.3	4
10Rack5				14	10.8	
10Rack6				28	12.1	
14Rack1	14.0	1-Day Heat	14.3	1	12.8	2.8
14Rack2		7-Day Moist	15.7	14	14.5	
14Rack3				28	15.7	
14Rack4				7	11.4	5.6
14Rack5				14	14.5	
14Rack6				28	15.7	
18Rack1	18.0	1-Day Heat	14.4	1	11.4	3.6
18Rack2		7-Day Moist	16.7	14	15.0	
18Rack3				28	16.7	
18Rack4				7	12.0	7.2
18Rack5				14	15.0	
18Rack6				28	16.7	

The measured shrinkage strain of the cylindrical shrinkage specimens with 10, 14 and 18 ksi target concrete compressive strengths are given in Appendix G. To calculate the creep strain, these shrinkage strains were deducted from the total measured strain of the loaded creep specimens. Average creep strains of specimens with 10, 14 and 18 ksi target concrete compressive strengths are shown in Figure 4-45 to Figure 4-47 and tabulated in Appendix G. Average specific creep strains, defined as creep strain per ksi, for all the creep specimens are presented in Figure 4-48 to Figure 4-50 and tabulated in Appendix G.

4 Test Results and Discussions

The average creep coefficients, defined as the ratios between the creep deformations at time t and the instantaneous elastic ones are also shown in Figure 4-51 to Figure 4-53 and presented in Appendix G.

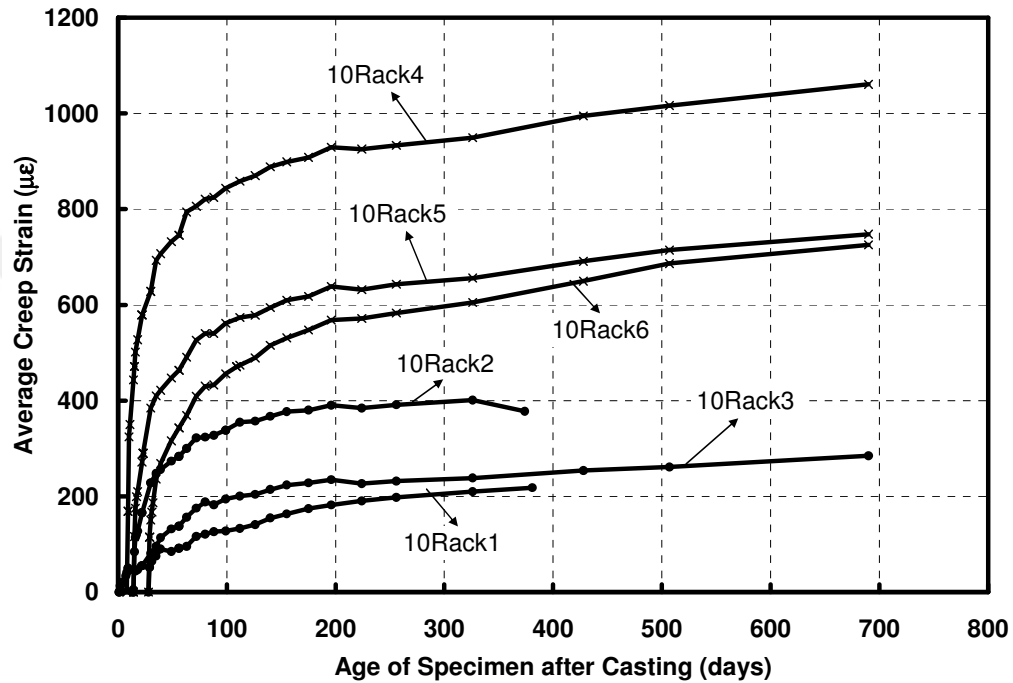


Figure 4-45 – Average Creep Strains of Specimens with 10 ksi Target Concrete Compressive Strength

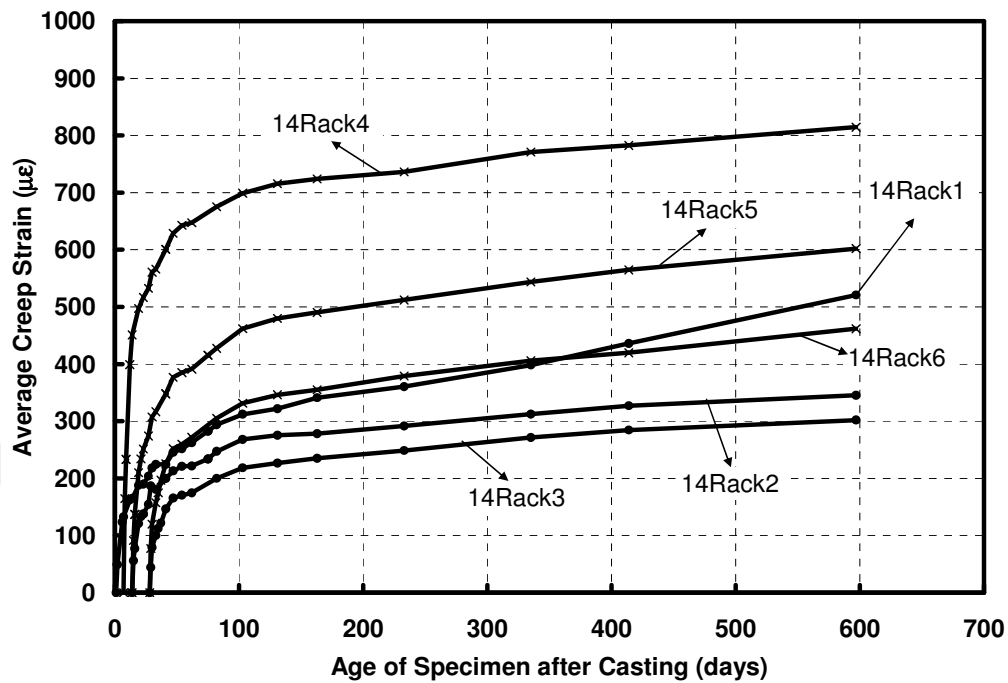


Figure 4-46 – Average Creep Strains of Specimens with 14 ksi Target Concrete Compressive Strength

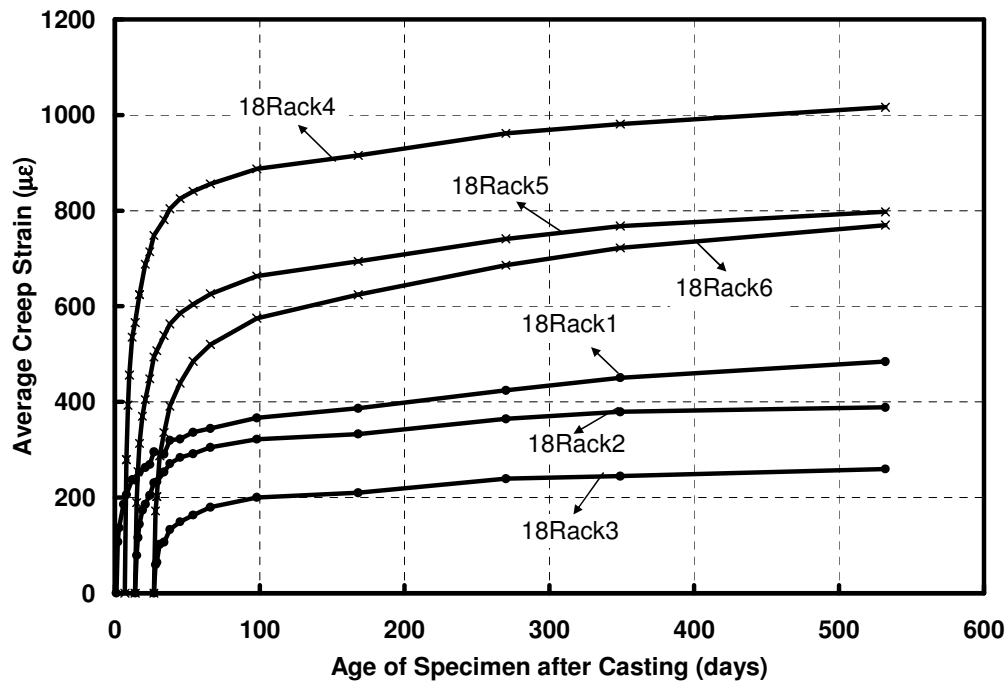


Figure 4-47 – Average Creep Strains of Specimens with 18 ksi Target Concrete Compressive Strength

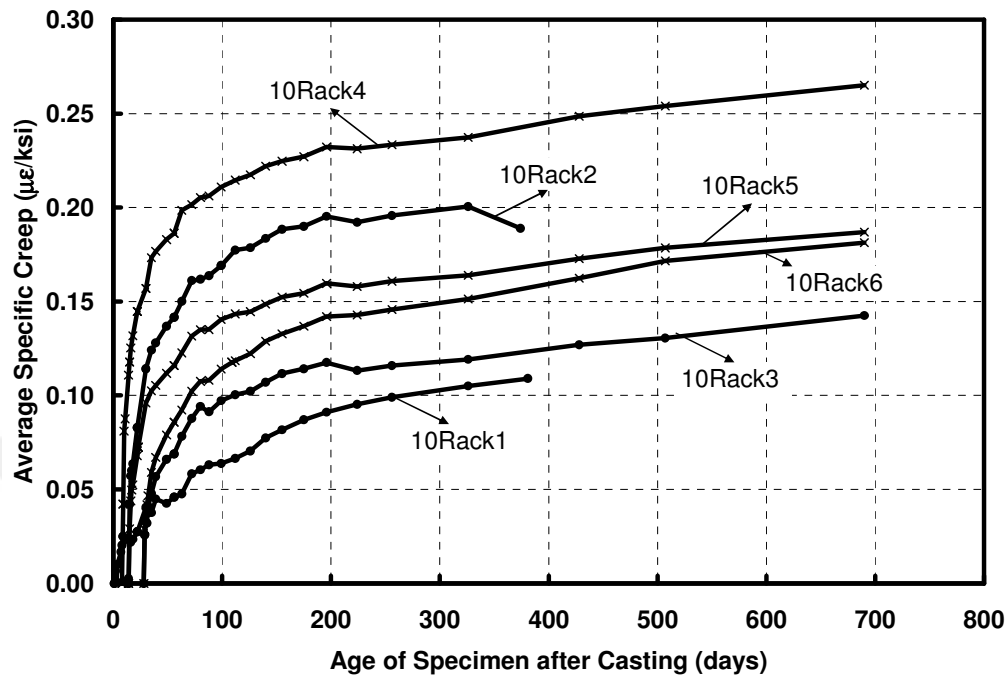


Figure 4-48 – Average Specific Creep Strains of Specimens with 10 ksi Target Concrete Compressive Strength

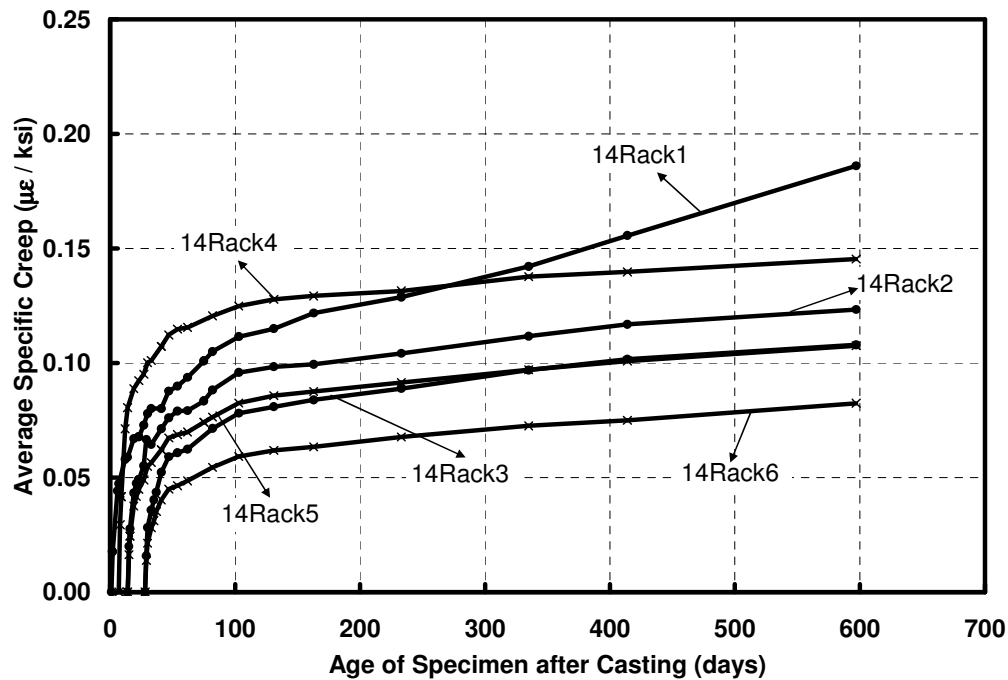


Figure 4-49 – Average Specific Creep Strains of Specimens with 14 ksi Target Concrete Compressive Strength

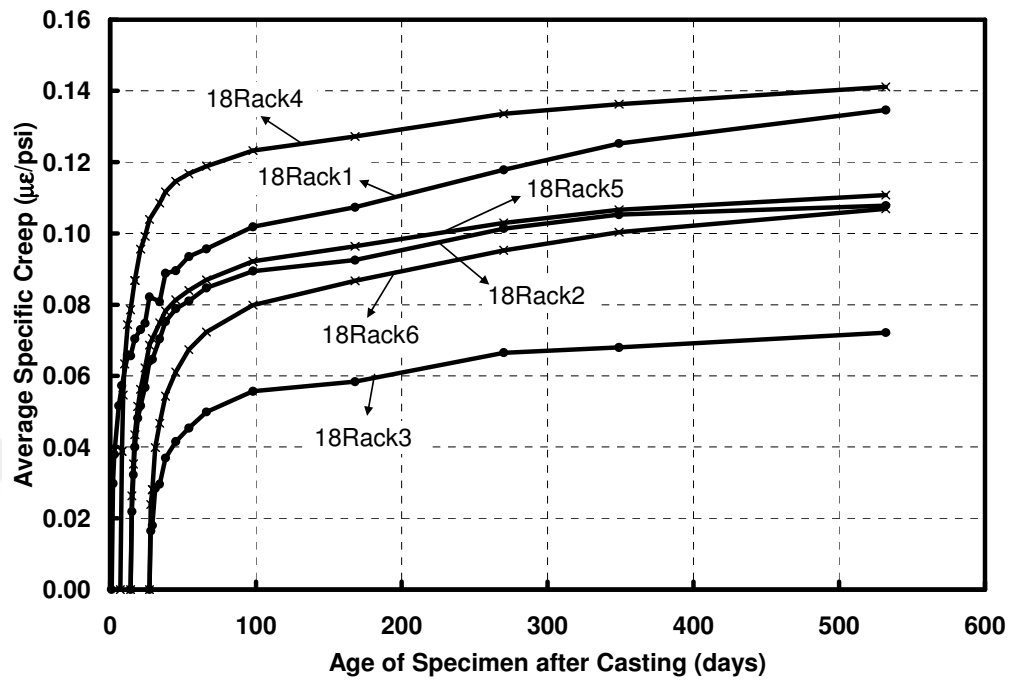


Figure 4-50 – Average Specific Creep Strains of Specimens with 18 ksi Target Concrete Compressive Strength

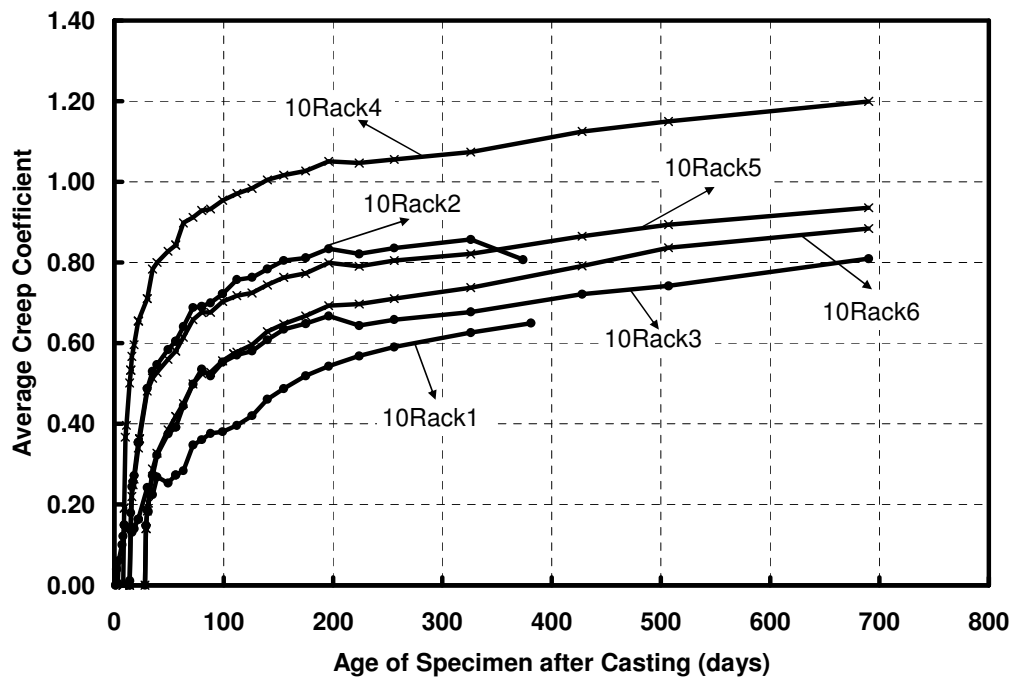


Figure 4-51 – Average Creep Coefficients of Specimens with 10 ksi Target Concrete Compressive Strength

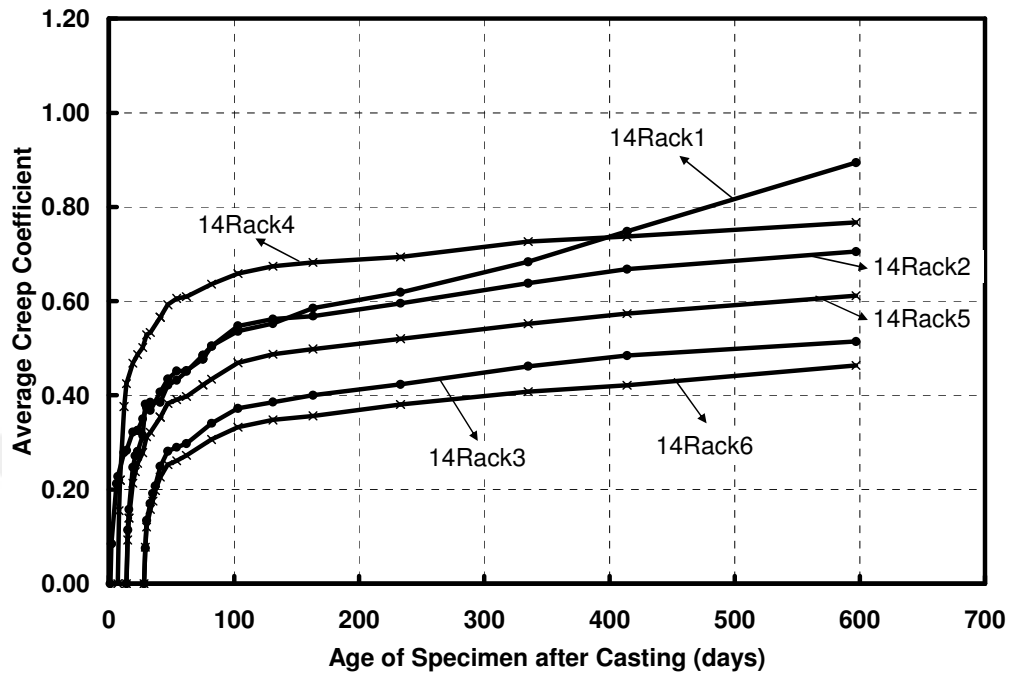


Figure 4-52 – Average Creep Coefficients of Specimens with 14 ksi Target Concrete Compressive Strength

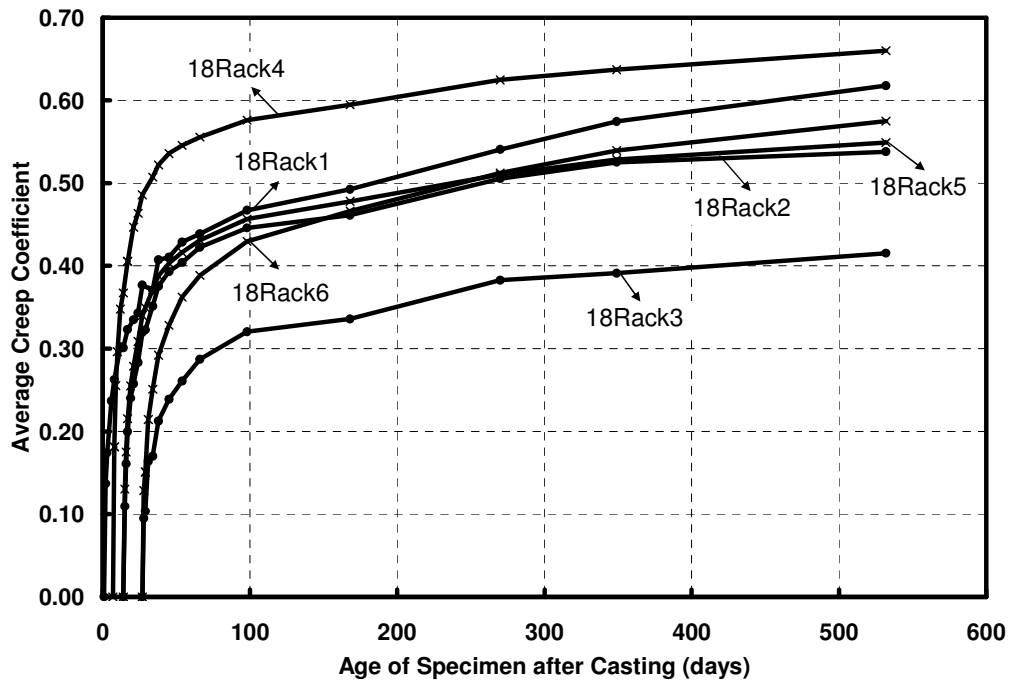


Figure 4-53 – Average Creep Coefficients of Specimens with 18 ksi Target Concrete Compressive Strength

4 Test Results and Discussions

The temperature and humidity of the ambient air where the creep specimens were loaded varied during the experimental program. The variations in temperature and humidity are given in Figure 4-54. Although the temperature during the experimental program was fairly constant, the variation in the humidity of the environment was quite significant.

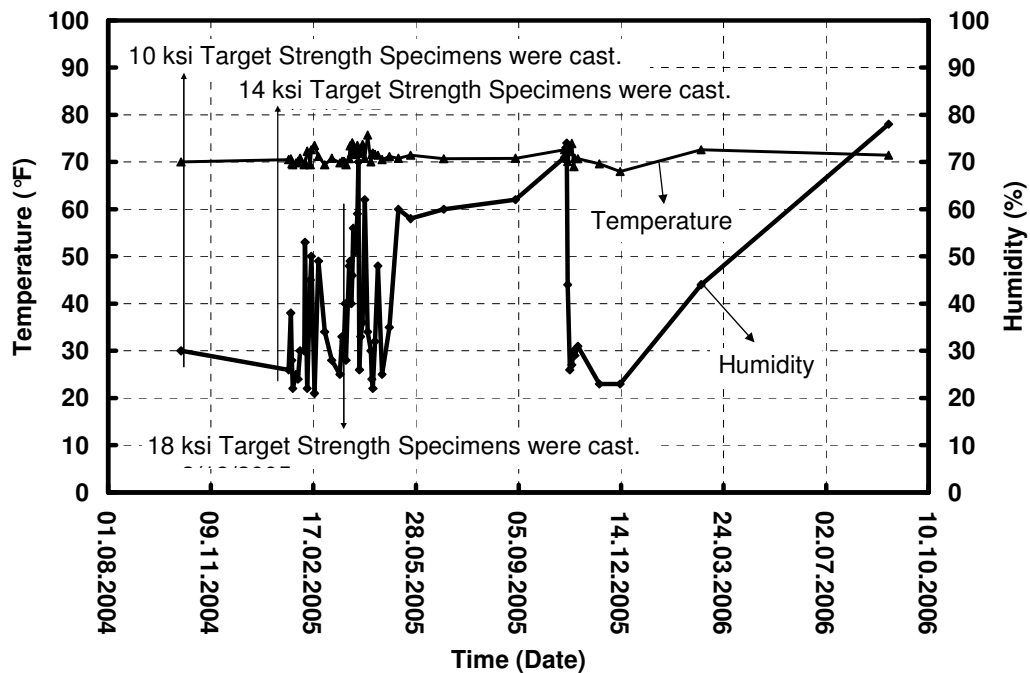


Figure 4-54 – Variations in Temperature and Humidity for Creep and Shrinkage Specimens

To account for effects of the variation in humidity on the creep behavior of HSC, the average creep coefficients were adjusted accordingly. The procedure used to calculate the adjusted average creep coefficients for each rack is given as follows:

1. Since each rack consists of two cylindrical creep specimens, the total strains of two cylinders were measured by using a demec gage (3rd and 4th column in Table 4-6).
2. The average of total strains was determined using the measured strains of two cylinders in each rack (5th column in Table 4-6).
3. The drying shrinkage of the companion cylindrical shrinkage specimen was measured by using a demec gage (6th column in Table 4-6), and deducted from the total average strain to obtain the average creep strain of each rack (7th column in Table 4-6).

4 Test Results and Discussions

4. The average creep coefficient of the rack at a given time was obtained based on the ratio of the average of the measured creep strain of each rack to the average initial elastic strain immediately after the cylinders were loaded (8th column in Table 4-6).
5. The creep coefficient for each rack was divided by the relative humidity correction factor ($k_{hc} = 1.58 - H/120$) which is specified by AASHTO LRFD Bridge Design Specifications (2004) and Tadros et al. (2003). This correction was performed incrementally, which means that the difference between the two consequent creep coefficient values was divided by the factor and added to the previous adjusted creep coefficient (9th column in Table 4-6).

A sample numerical illustration of this procedure for 18Rack2 is presented Table 4-6.

Table 4-6 – Numerical Illustration of the Calculation Procedure

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9
Age After Casting (days)	Humidity (%)	Total Strain of Specimen 1 (µε)	Total Strain of Specimen 2 (µε)	Average Total Strain of Rack #14 (µε)	Drying Shrinkage Strain of the Companion Cylinder (µε)	Average Creep Strain of Rack #14 (µε)	Average Creep Coefficient of Rack #14	Adjusted Average Creep Coefficient of Rack #14
14	59	0.0	0.0	0.0	0.0	0.0	0.0	0.000
15	70	108.1	90.4	99.3	20.3	78.9	0.109	0.109
16	26	166.6	143.6	155.1	39.0	116.1	0.160	0.148
17	33	203.9	178.2	191.0	46.9	144.0	0.199	0.177
19	36	242.0	214.5	228.3	54.9	173.3	0.240	0.209
21	62	282.8	251.8	267.3	81.5	185.7	0.257	0.225
24	34	305.0	279.3	292.1	87.7	204.3	0.283	0.245
27	30	369.7	336.0	352.8	122.3	230.5	0.319	0.273
29	22	387.4	359.9	373.7	140.9	232.7	0.322	0.275
34	48	422.0	391.9	406.9	153.3	253.5	0.351	0.300
38	25	454.8	422.0	438.4	167.5	270.8	0.375	0.317

Note that, the average initial elastic strain for Rack #14 is 721.7 µε.

Using this procedure, the adjusted average creep coefficients for each rack were calculated as shown in Figure 4-55 to Figure 4-57.

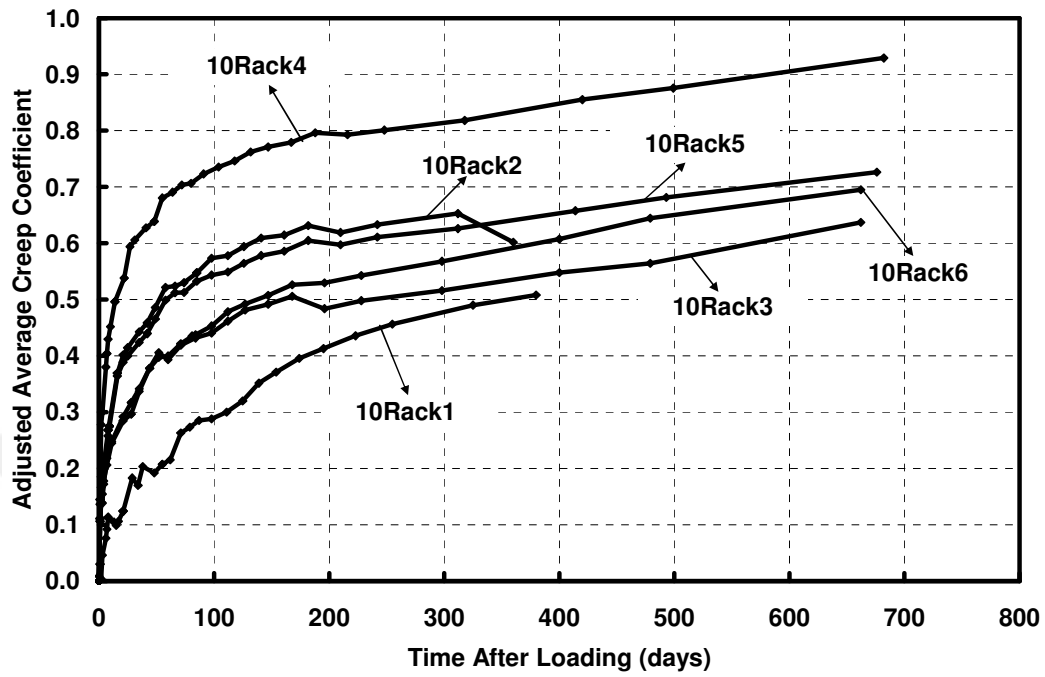


Figure 4-55 – Adjusted Average Creep Coefficients of Specimens with 10 ksi Target Concrete Compressive Strength

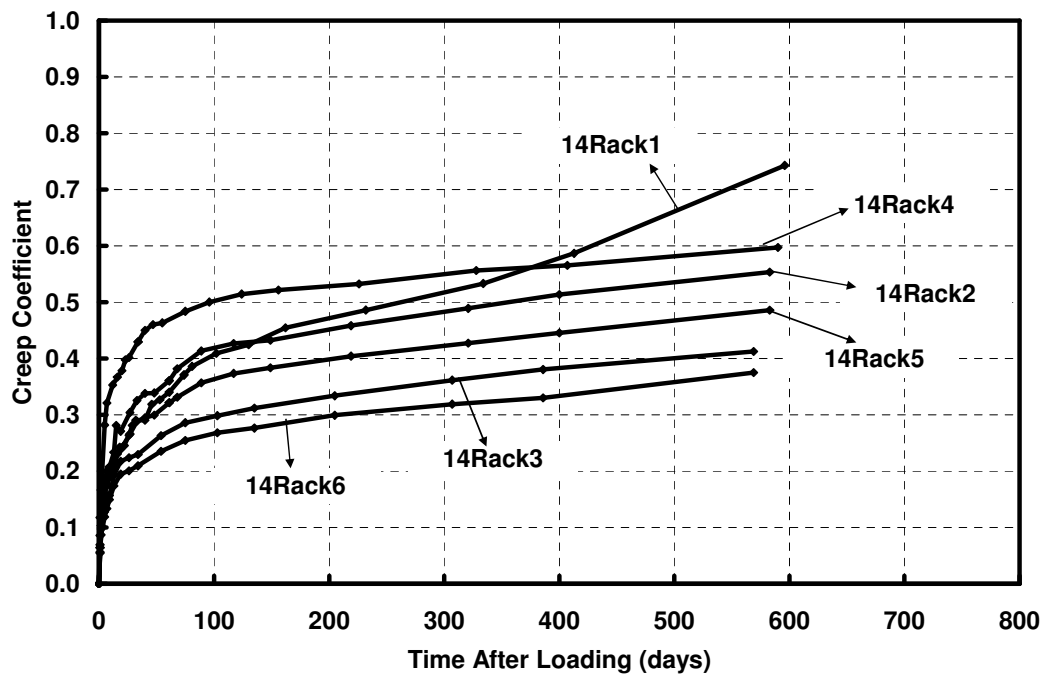


Figure 4-56 – Adjusted Average Creep Coefficients of Specimens with 14 ksi Target Concrete Compressive Strength

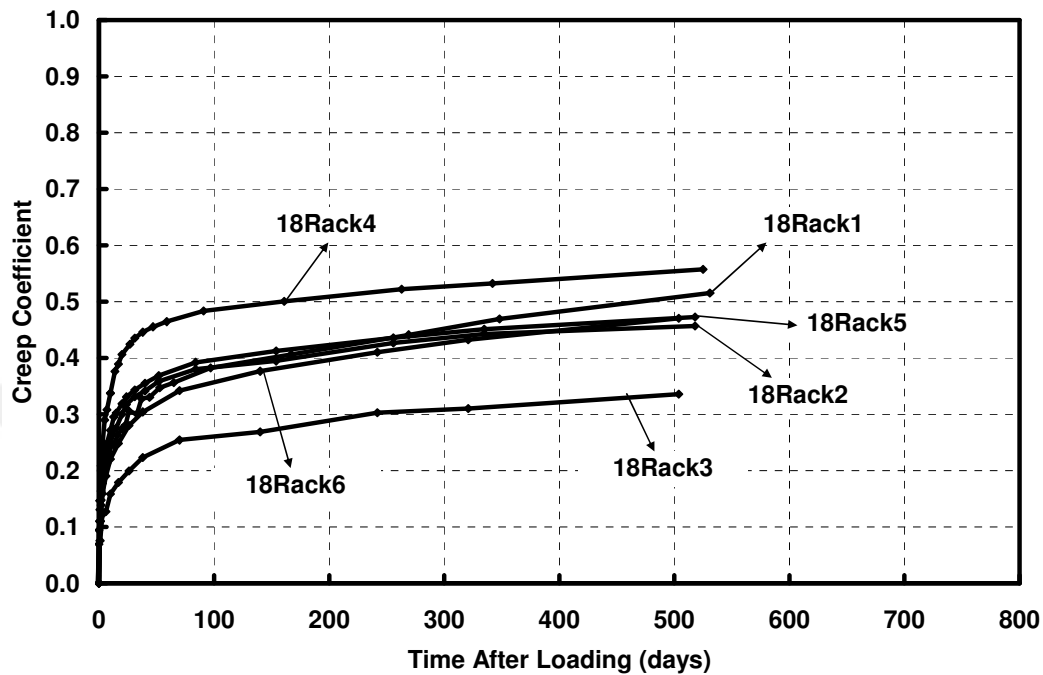


Figure 4-57 – Adjusted Average Creep Coefficients of Specimens with 18 ksi Target Concrete Compressive Strength

In general, test results indicate that as concrete gets mature and older, the creep of HSC decreases. The creep behavior of heat cured cylinders is less than that of the moist cured cylinders. Creep for HSC is proportional to the applied stress provided that the applied stress is less than the proportional limit. Test results obtained from this research combined with the data in the literature were used to propose a relationship to predict the creep coefficient for HSC in Section 5.4.

4.4 Shrinkage Behavior

Six 4×12 in. cylindrical and eighteen 3×3×11¼ in. prismatic specimens were monitored to evaluate the shrinkage behavior of HSC. Specimens of each target concrete strength were cast in one batch. The concrete compressive strengths of 4×8 in. cylinders at 28 days varied from 10.4 to 16.7 ksi. Test set-up and test method are presented in Section 3.4. Details about the cylindrical and prismatic shrinkage specimens are given in Table 4-7 and Table 4-8, respectively.

Table 4-7 – Details about Cylindrical Shrinkage Specimens

Specimen ID	Target Concrete Compressive Strength (ksi)	Curing Type	Concrete Compressive Strength @ 28 days (ksi)	Day of Initial Monitoring	Concrete Compressive Strength @ Day of Initial Monitoring (ksi)
10SC1	10.0	1-Day Heat	10.4	1	9.6
10SC2		7-Day Moist	12.1	7	8.3
14SC1	14.0	1-Day Heat	14.3	1	12.8
14SC2		7-Day Moist	15.7	7	11.4
18SC1	18.0	1-Day Heat	14.4	1	11.4
18SC2		7-Day Moist	16.7	7	12.0

Table 4-8 – Details about Prismatic Shrinkage Specimens

Specimen ID	Target Concrete Compressive Strength (ksi)	Curing Type	Concrete Compressive Strength @ 28 days (ksi)	Day of Initial Monitoring	Concrete Compressive Strength @ Day of Initial Monitoring (ksi)
10SP1	10.0	1-Day Heat	10.4	1	9.6
10SP2					
10SP3					
10SP4		7-Day Moist	12.1	7	8.3
10SP5					
10SP6					
14SP1	14.0	1-Day Heat	14.3	1	12.8
14SP2					
14SP3					
14SP4		7-Day Moist	15.7	7	11.4
14SP5					
14SP6					
18SP1	18.0	1-Day Heat	14.4	1	11.4
18SP2					
18SP3					
18SP4		7-Day Moist	16.7	7	12.0
18SP5					
18SP6					

The measured shrinkage strains of cylindrical and prismatic specimens with 10, 14 and 18 ksi target concrete compressive strengths are shown in Figure 4-58 to Figure 4-63 and tabulated in Appendix G. Using the measured weights of the prismatic specimens, the weight loss is calculated for prismatic specimens with 10, 14 and 18 ksi target concrete compressive strengths. The weight losses are also shown in Figure 4-64 to Figure 4-66 and tabulated in Appendix G.

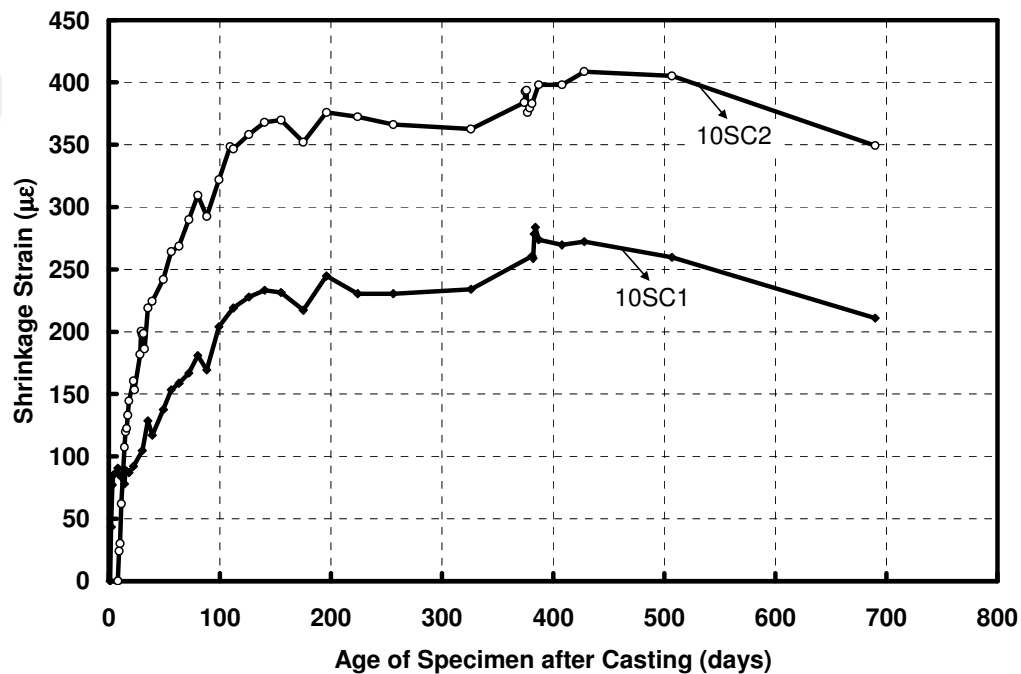


Figure 4-58 – Shrinkage Strain of Cylindrical Specimens with 10 ksi Target Concrete Compressive Strength

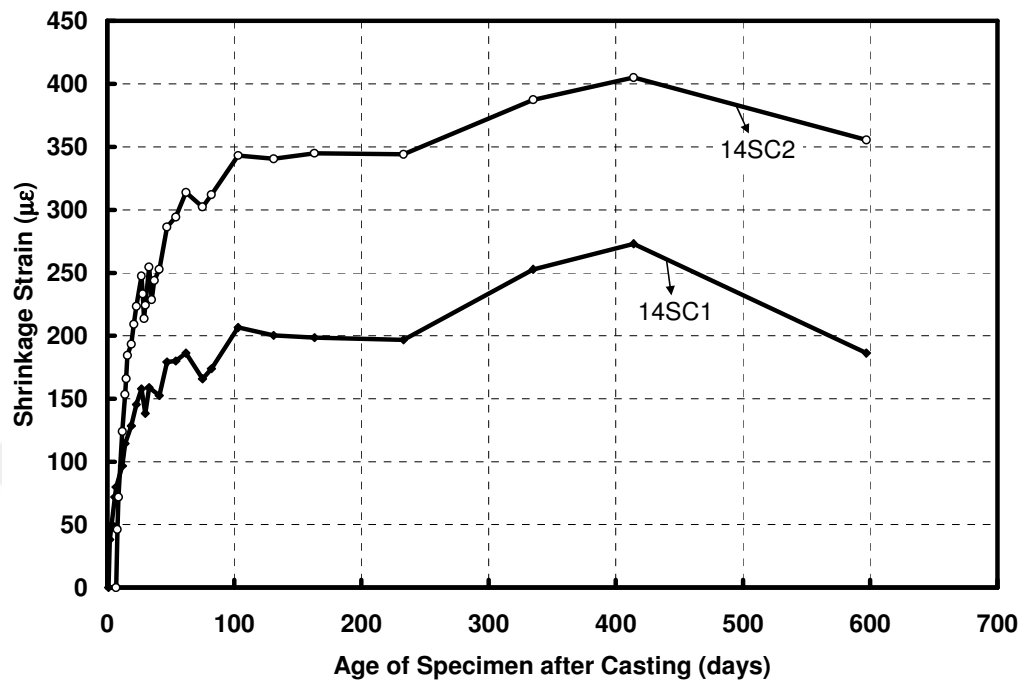


Figure 4-59 – Shrinkage Strain of Cylindrical Specimens with 14 ksi Target Concrete Compressive Strength

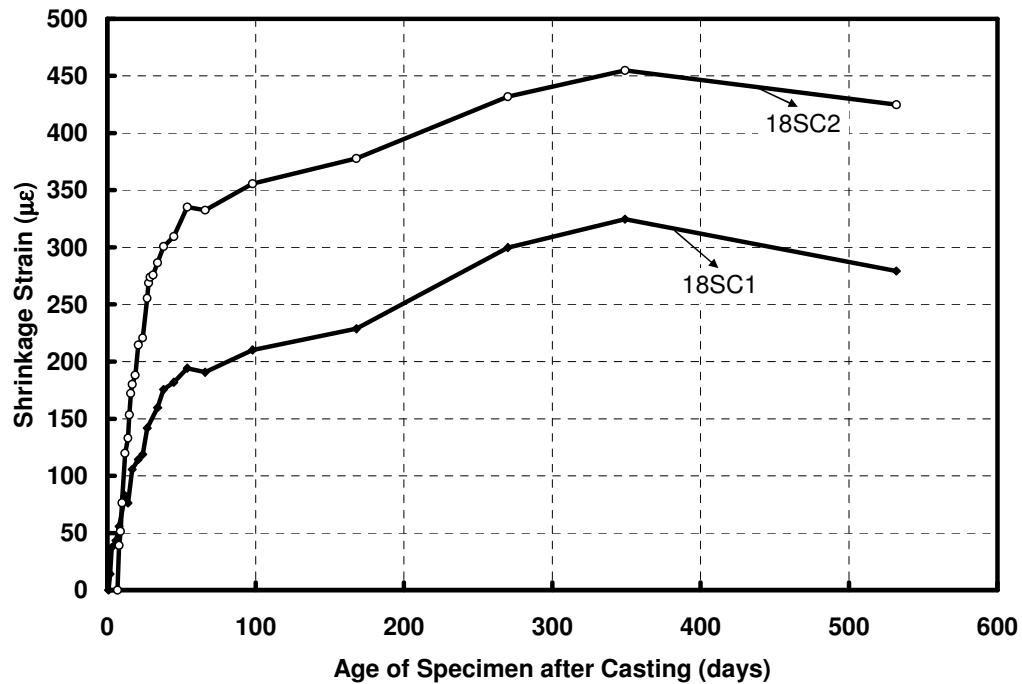


Figure 4-60 – Shrinkage Strain of Cylindrical Specimens with 18 ksi Target Concrete Compressive Strength

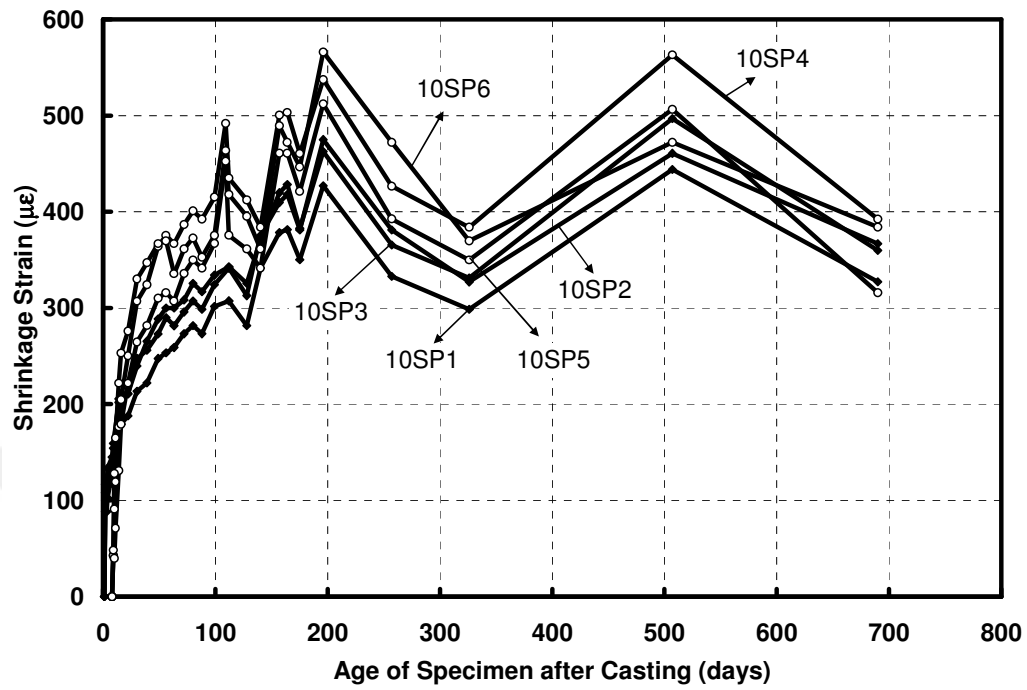


Figure 4-61 – Shrinkage Strain of Prismatic Specimens with 10 ksi Target Concrete Compressive Strength

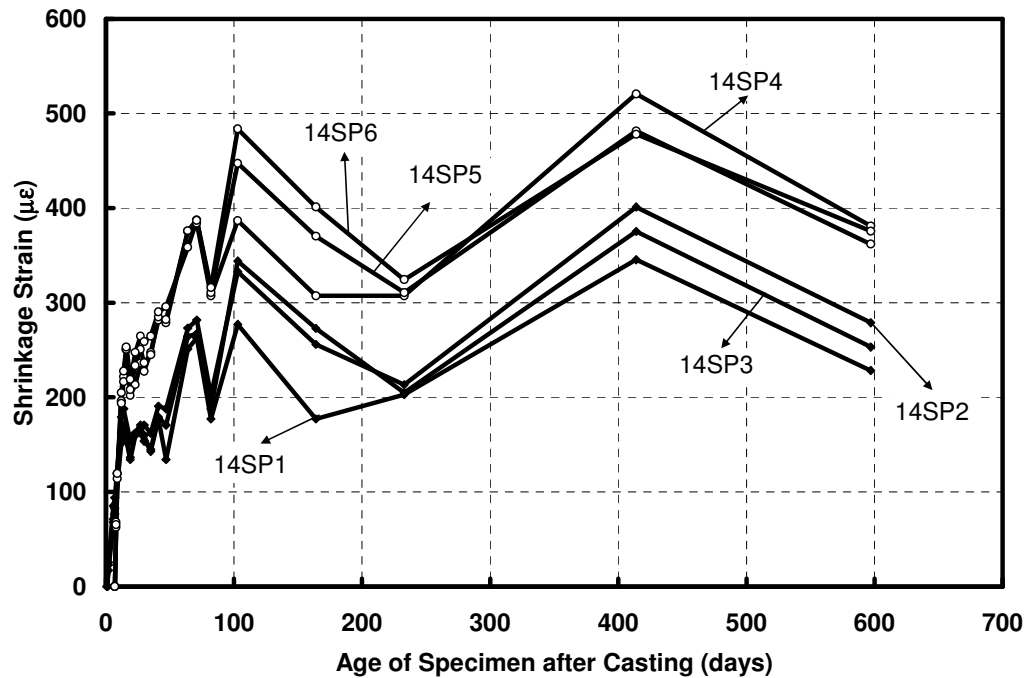


Figure 4-62 – Shrinkage Strain of Prismatic Specimens with 14 ksi Target Concrete Compressive Strength

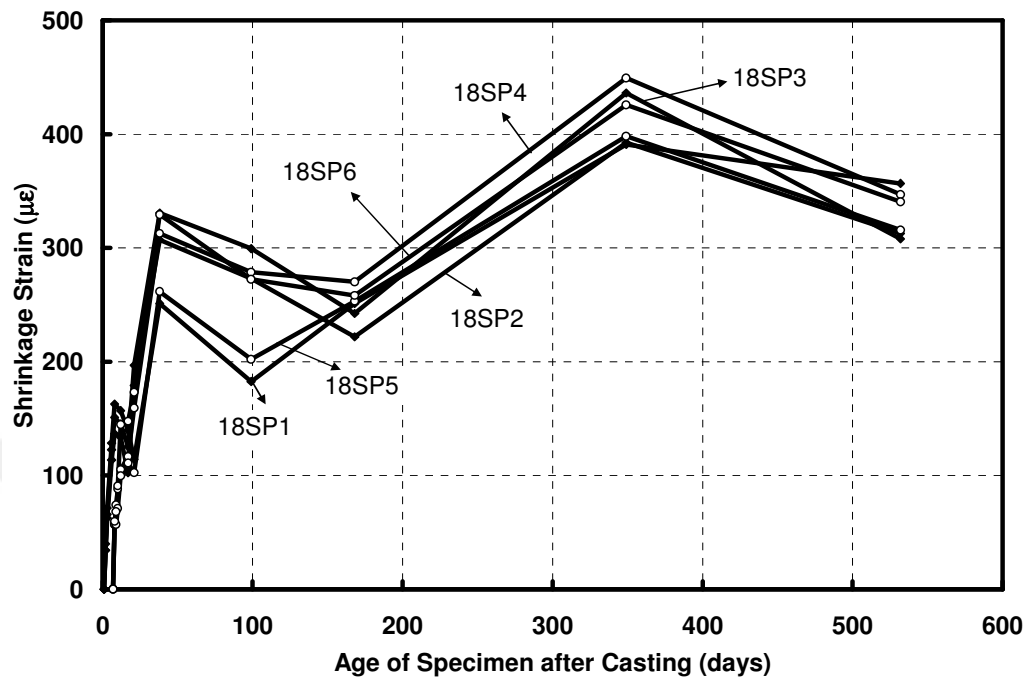


Figure 4-63 – Shrinkage Strain of Prismatic Specimens with 18 ksi Target Concrete Compressive Strength

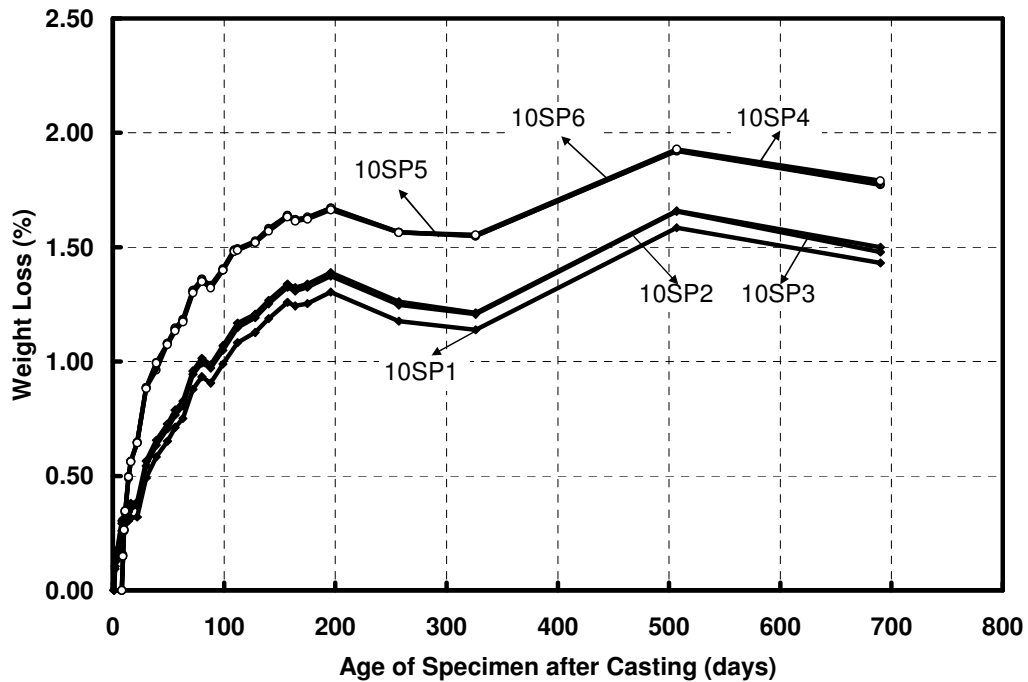


Figure 4-64 – Weight Loss of Prismatic Specimens with 10 ksi Target Concrete Compressive Strength

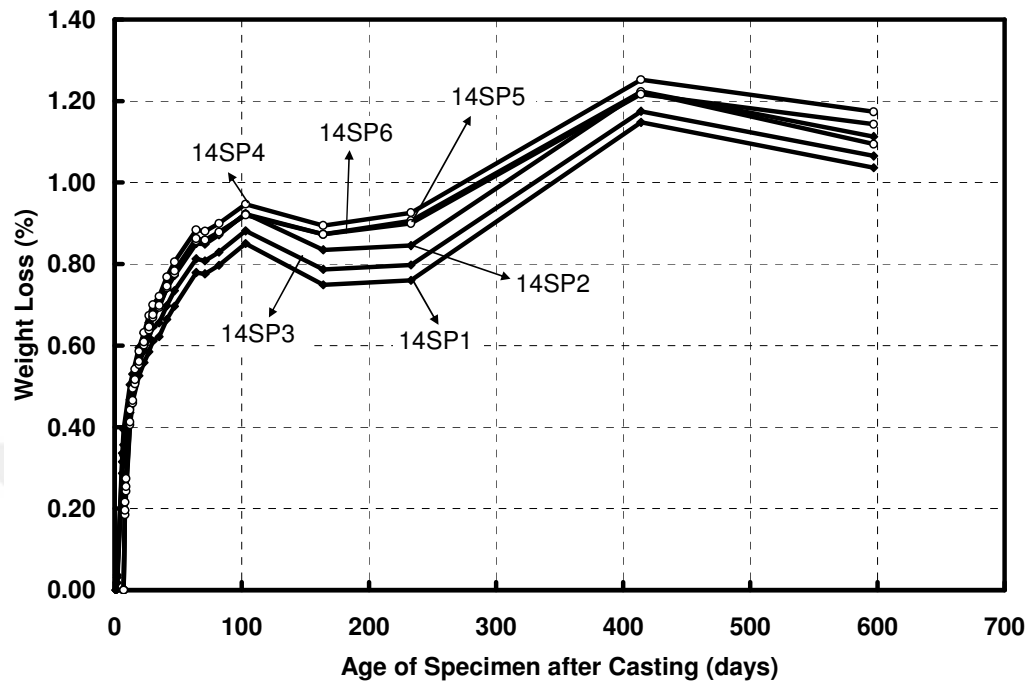


Figure 4-65 – Weight Loss of Prismatic Specimens with 14 ksi Target Concrete Compressive Strength

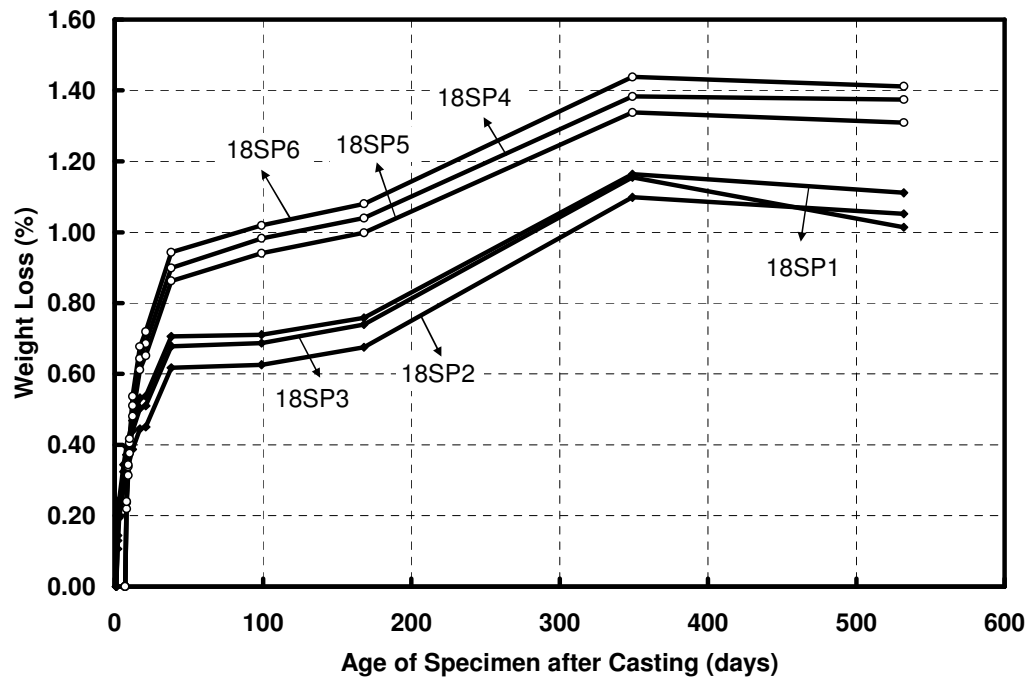


Figure 4-66 – Weight Loss of Prismatic Specimens with 18 ksi Target Concrete Compressive Strength

4 Test Results and Discussions

The temperature and humidity of the ambient air where the cylindrical and prismatic shrinkage specimens were kept varied during the experimental program. The variations in temperature and humidity are given in Figure 4-54. Although the temperature during the experimental program was fairly constant, the variation in the humidity of the environment was quite significant.

Since the relative humidity of the ambient air varied throughout the testing period, the shrinkage strain was adjusted accordingly. The procedure used to calculate the adjusted shrinkage strain for each specimen is given as follows:

1. The shrinkage strain of cylindrical and prism specimens were measured by using a demec gage and shrinkage testing apparatus, respectively.
2. The shrinkage strain for each specimen was divided by the relative humidity correction factor ($k_{hs} = (140-H)/70$) which is specified by AASHTO LRFD Bridge Design Specifications (2004) and Tadros et al. (2003). This correction was performed incrementally which means that the difference between two consequent shrinkage strains was divided by the factor and added to the previous adjusted shrinkage strain (same procedure illustrated to adjust the creep coefficients in Section 4.3 was used).

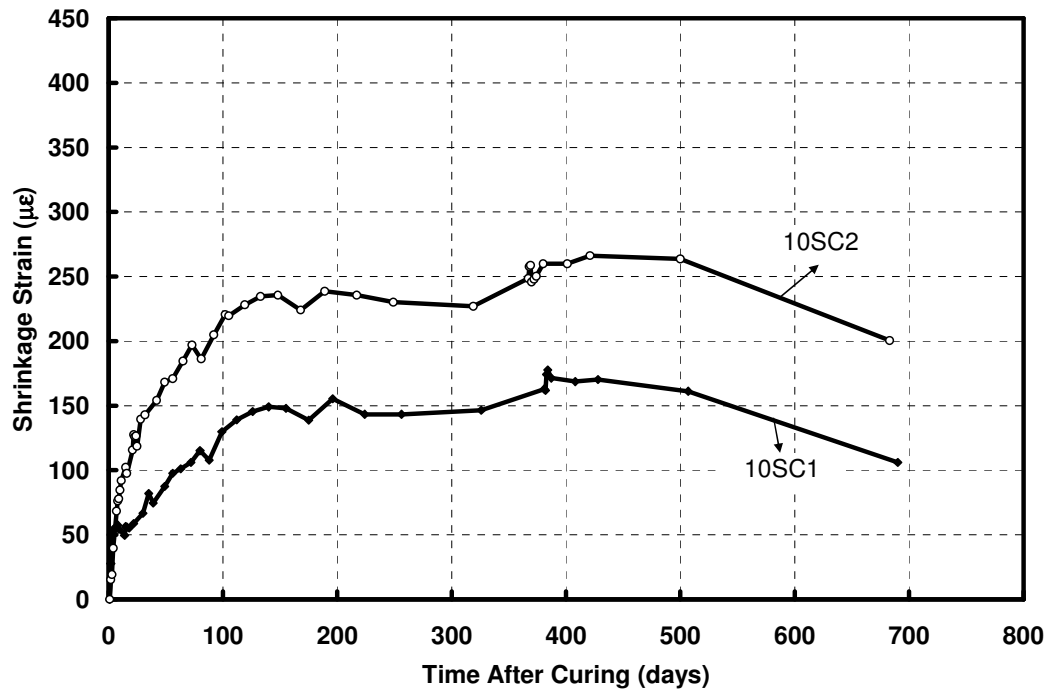


Figure 4-67 – Adjusted Shrinkage Strain of Cylindrical Specimens with 10 ksi Target Concrete Compressive Strength

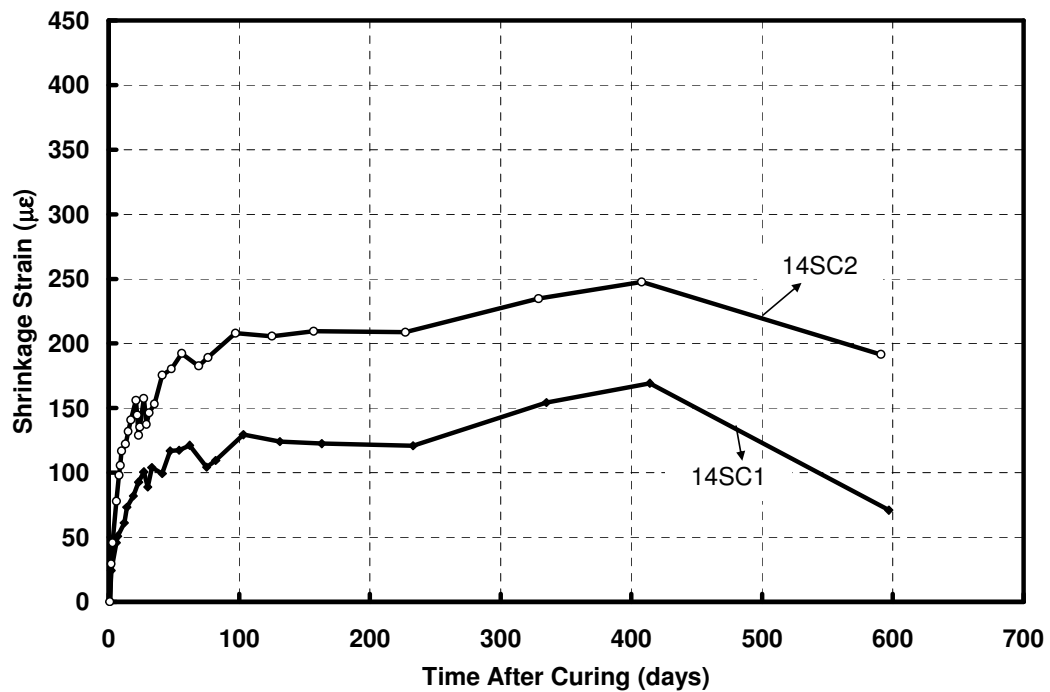


Figure 4-68 – Adjusted Shrinkage Strain of Cylindrical Specimens with 14 ksi Target Concrete Compressive Strength

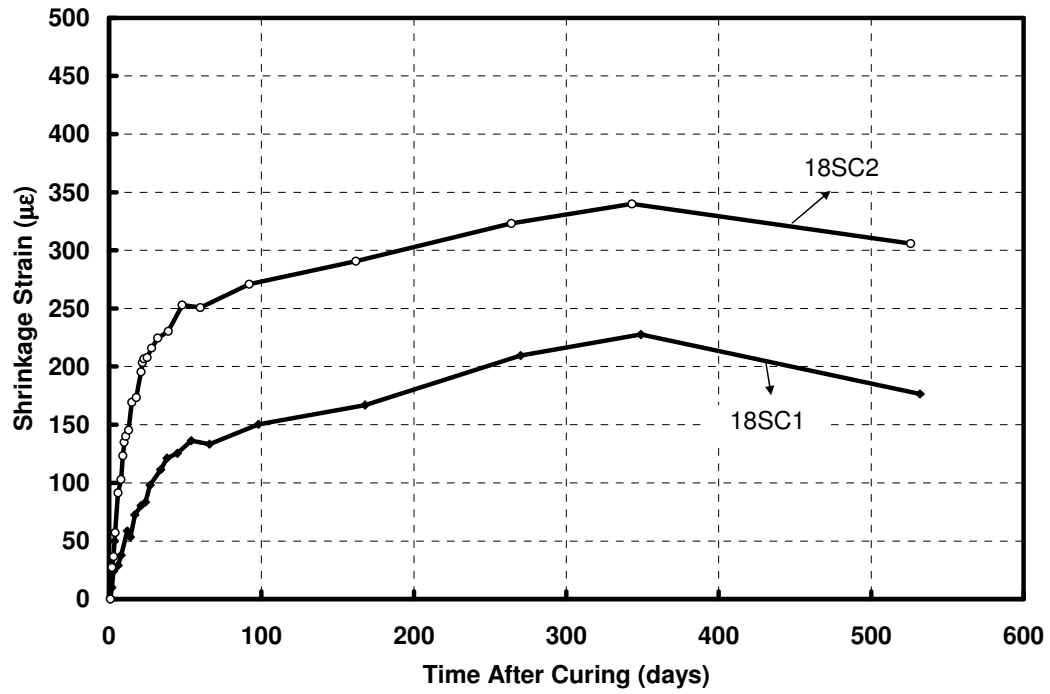


Figure 4-69 – Adjusted Shrinkage Strain of Cylindrical Specimens with 18 ksi Target Concrete Compressive Strength

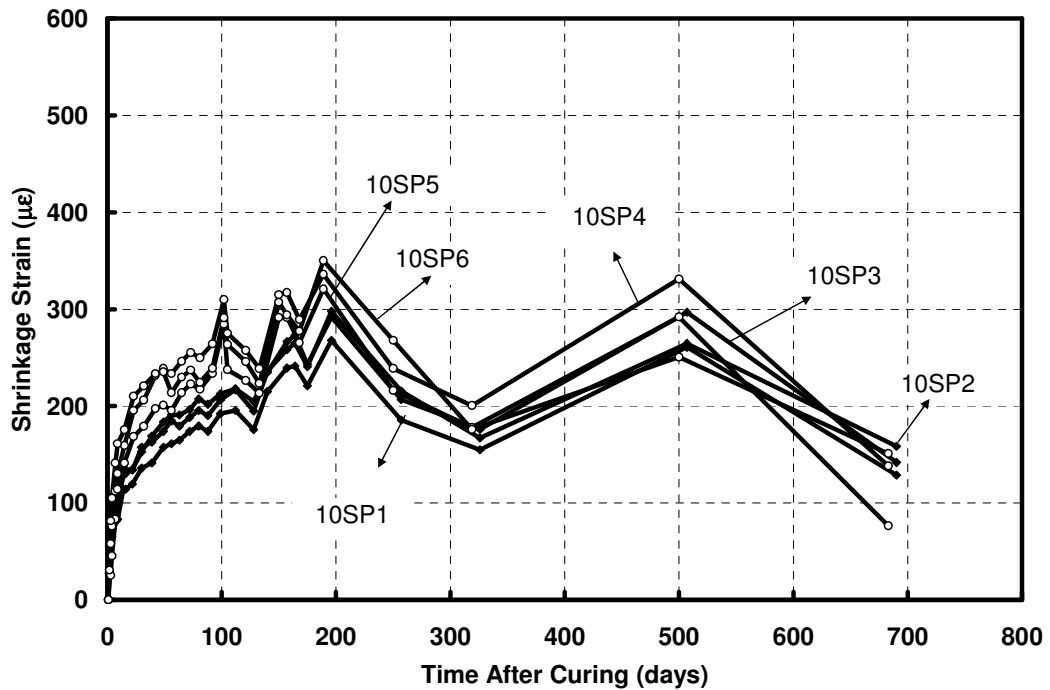


Figure 4-70 – Adjusted Shrinkage Strain of Prismatic Specimens with 10 ksi Target Concrete Compressive Strength

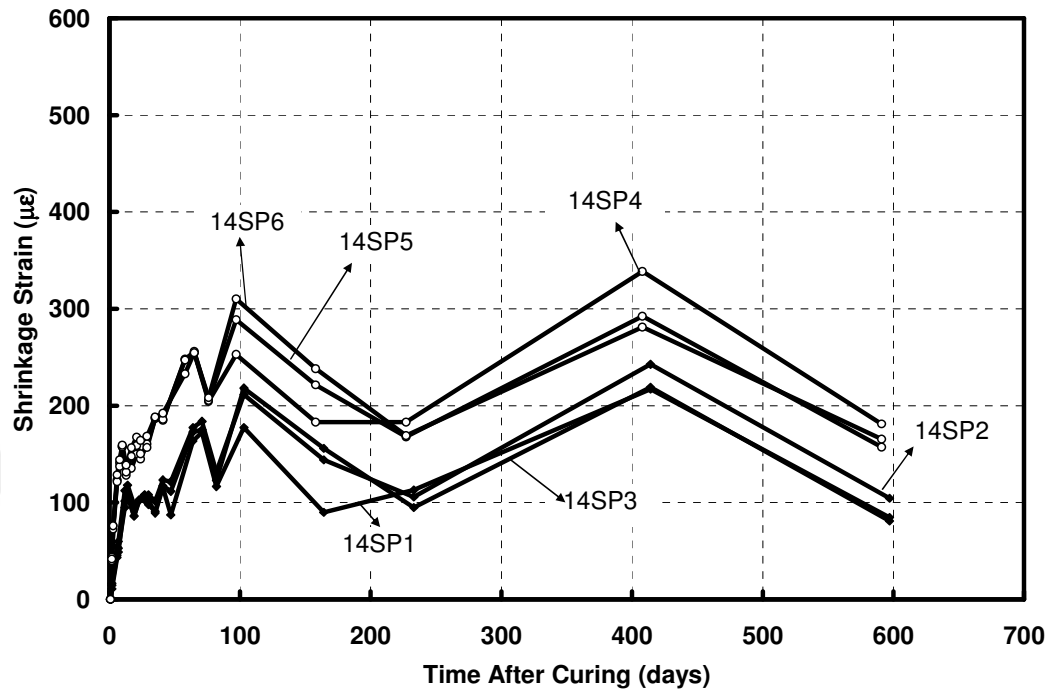


Figure 4-71 – Adjusted Shrinkage Strain of Prismatic Specimens with 14 ksi Target Concrete Compressive Strength

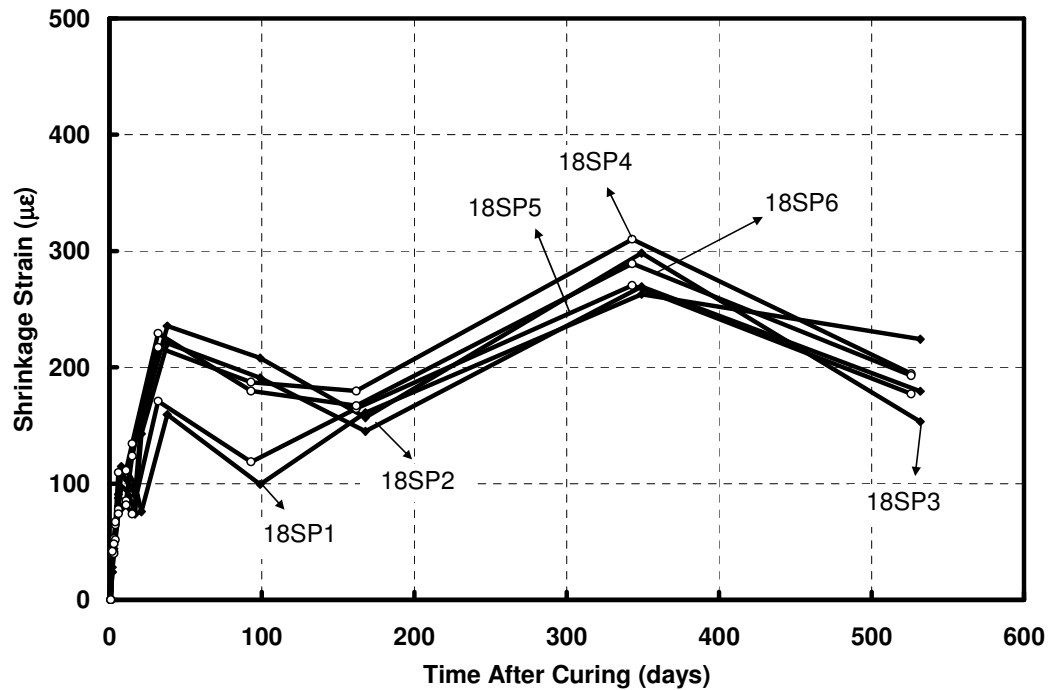


Figure 4-72 – Adjusted Shrinkage Strain of Prismatic Specimens with 18 ksi Target Concrete Compressive Strength

4 Test Results and Discussions

Test results of this research on shrinkage of HSC indicate that heat cured specimens have less shrinkage compared to moist cured specimens. There is little difference in shrinkage of HSC specimens with target concrete strengths of 10, 14 and 18 ksi. The average weight loss of the specimens is directly proportional to shrinkage of HSC. Test results obtained from this research combined with the data in the literature were used to propose a relationship to predict the shrinkage strain for HSC in Section 5.4.



5 ANALYTICAL WORK

This section provides critical reviews of the researches, test results, analyses, relationships and equations presented in Chapter 2. Based on the test results of this research combined with the data in the literature, either new relationships or currently used relationships with some modifications are proposed to define the characteristics of HSC for concrete compressive strengths up to 18 ksi. The results of this research combined with the available data in the literature are used to recommend revisions for the AASHTO LRFD Bridge Design Specifications (2004) to extend the current limitation of 10 ksi concrete compressive strength up to 18 ksi. The proposed relationships are also validated and confirmed for concrete compressive strengths up to 18 ksi using statistical and parametric analyses.

In general, there are four different types of loading conditions that act on the structural members such as: static (short term) loading, sustained loading, fatigue loading and impact loading conditions. The discussions in this chapter are mainly concerned with static loading condition except for creep and shrinkage discussions which are based on sustained loading conditions. Note that, in case of a beam subjected to loads for a long time, the stress-strain relationship of concrete will be modified due to the time-dependent behavior of concrete, such that, the descending branch of the stress-strain relationship may have no significance in the behavior of the structural member.

5.1 Proposed Stress Block Parameters and Ultimate Compressive Strain for HSC

The researches reported in the literature established some basic characteristic properties of HSC. In general, as concrete compressive strength increases, the strain at the peak stress increases and gets closer to 0.003. The shape of the ascending branch of the stress-strain relationship becomes more linear and steeper, and the slope of the descending part also becomes steeper. The general shape of the stress-strain relationship becomes more likely to be a triangle.

The stress-strain distribution of NSC can be generalized by the curved shape as shown in Figure 5-1. For this type of distribution, k_1 and k_2 are equal to 0.85 and 0.425,

respectively. When converted to a rectangular distribution, α_l and β_l correspond to k_3 and 0.85, respectively. If the stress-strain distribution of HSC is assumed to be a triangular distribution, k_1 and k_2 are equal to 0.50 and 0.333, respectively. The rectangular stress block parameters, α_l and β_l , for triangular distribution are $0.75k_3$ and 0.667, respectively. These parameters are shown in Figure 5-1.

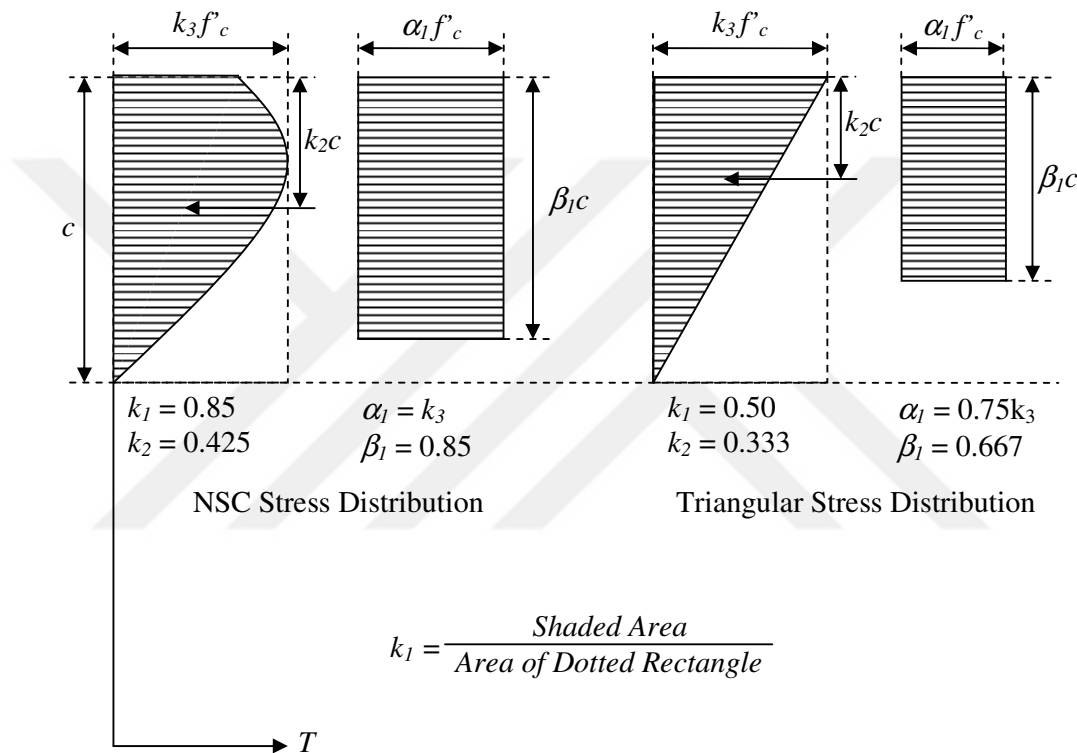


Figure 5-1 – Stress Block Parameters for Different Stress Distributions

Based on the researches in the literature, the rectangular stress block parameters also decrease as the concrete compressive strength increases. But there is no agreement on the magnitude and the variation of this reduction with respect to concrete compressive strengths. Aziznamini (1994) proposed to reduce the rectangular stress block parameter α_l , from 0.85 to 0.60 for concrete compressive strengths varying from 10 to 15 ksi, respectively. Li (1993) recommended varying α_l , from 0.85 to 0.75 for concrete compressive strengths varying from 8 to 11.6 ksi, respectively. Ibrahim (1994) suggested reducing α_l , from 0.85 to 0.725 linearly for concrete compressive strengths from 0 to 14.5 ksi, respectively. Bae and Bayrak (2003) reduced α_l , from 0.85 to 0.67 for concrete compressive strengths from 4.35 to 13.4 ksi, respectively. However for β_l , the reduction

occurs much earlier than that of α_l . Mattock et al. (1961) proposed to decrease β_l , from 0.85 to 0.65 for concrete compressive strengths from 4 to 8 ksi, respectively. Rangan et al. (1999) suggested reducing β_l , from 0.85 to 0.65 for concrete compressive strengths from 4.35 to 8 ksi, respectively. Details about the proposed rectangular stress block parameters can be found in Section 2.2.4.

The test results of this research and other researches in the literature indicate that the majority of the collected data for the generalized stress block parameter k_l for HSC is higher than 0.58 for concrete compressive strengths ranging between 10 and 18 ksi as shown in Figure 5-2. Therefore, using the lower bound of 0.58 is suggested for k_l parameter for concrete compressive strengths beyond 15 ksi.

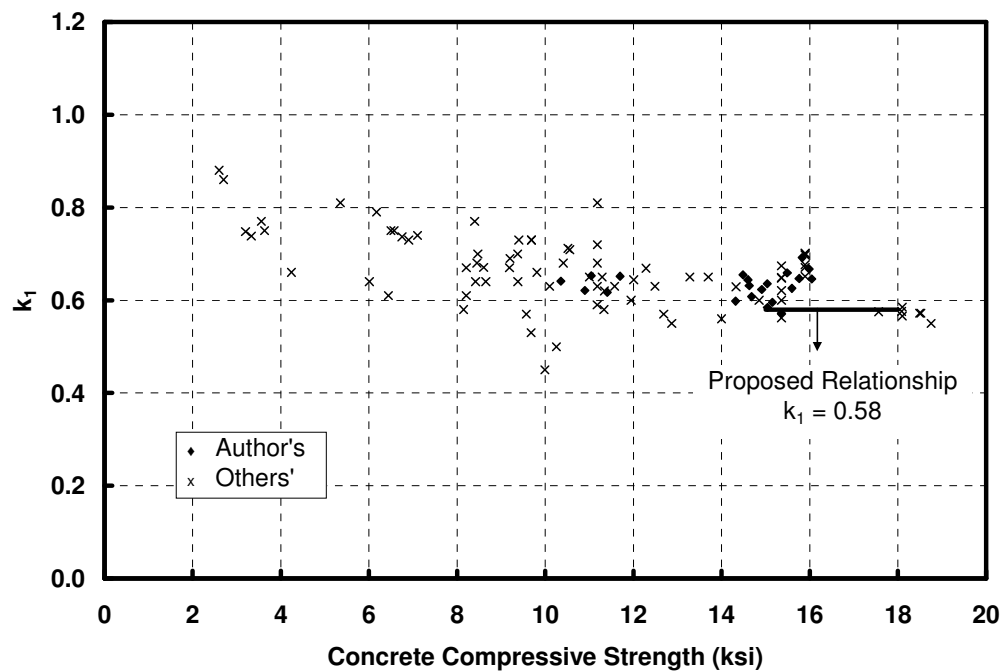


Figure 5-2 – Proposed Relationship for the Stress Block Parameter k_l

The k_2 parameter in AASHTO LRFD Bridge Design Specifications (2004) is already set to 0.33 for concrete compressive strengths beyond 8 ksi, since the assumed β_l parameter used in design is equal to 0.65. The test results of this research and other researches in the literature indicate that the stress block parameter k_2 for HSC between 8 ksi and 18 ksi can be assumed to be 0.33 as shown in Figure 5-3.

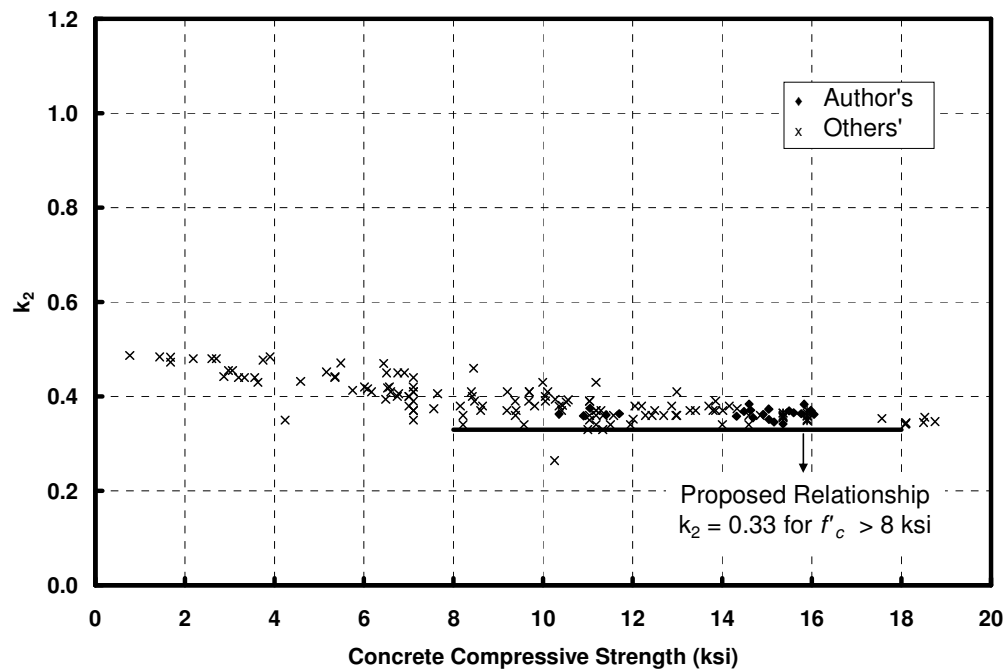


Figure 5-3 – Proposed Relationship for the Stress Block Parameter k_2

The test results of this research and other researches in the literature indicate that the generalized stress block parameter k_3 for HSC is similar to NSC as shown in Figure 5-4. Hence, using the same value of k_3 parameter, 0.85, for concrete compressive strengths up to 18 ksi is found to be completely appropriate for design purposes. The comparison of the product of the proposed k_1 and k_3 to the test results of this research and other researchers in the literature are shown in Figure 5-5.

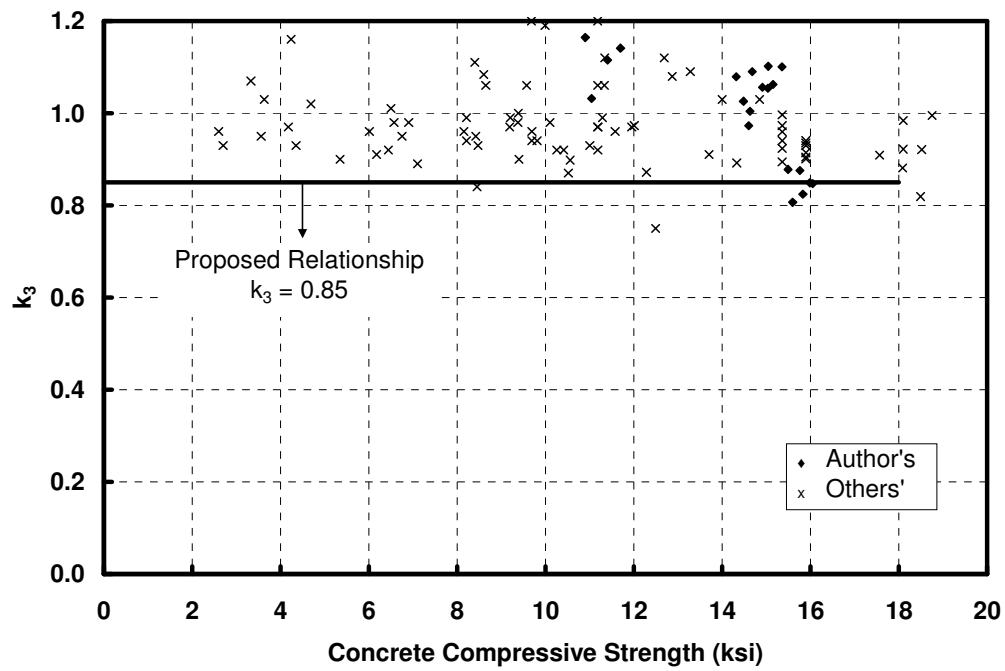


Figure 5-4 – Proposed Relationship for the Stress Block Parameter k_3

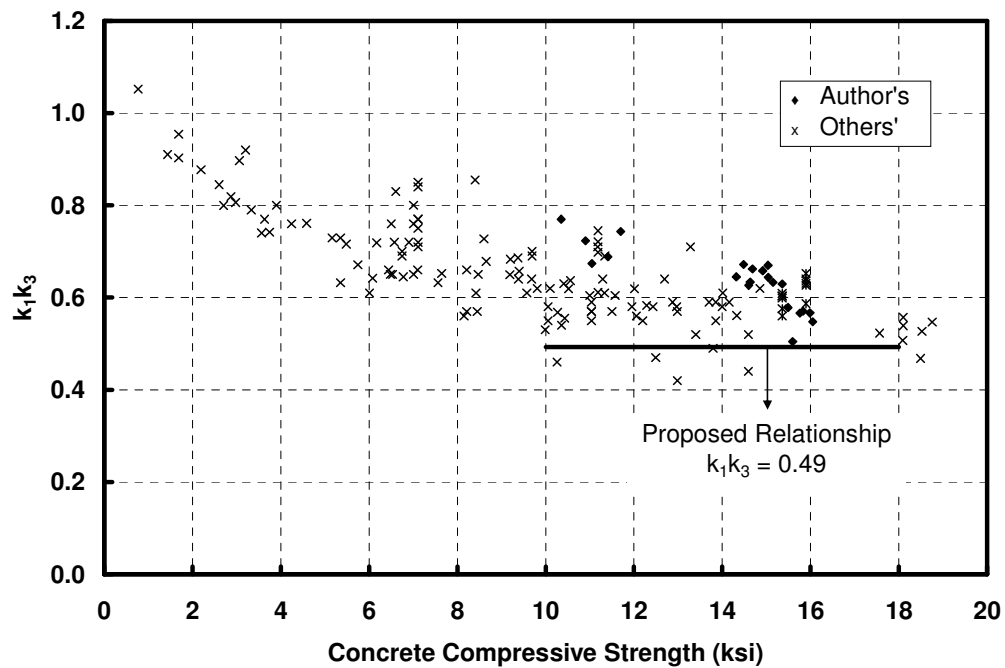


Figure 5-5 – Proposed Relationship for the Product of Stress Block Parameters k_1k_3

Using the values proposed for the generalized stress block parameters, the lower bound relationships for rectangular stress block parameters α_l and β_l can be obtained as follows;

$$\alpha_l = \frac{k_1 k_3}{2k_2} = \frac{0.58 \times 0.85}{2 \times 0.33} = 0.75 \quad \text{Equation 5-1}$$

$$\beta_l = 2k_2 = 2 \times 0.33 \approx 0.65 \quad \text{Equation 5-2}$$

Based on the test results of this research and other researches literature, the ultimate concrete compressive strain of 0.003 is proposed to be used for design purposes for concrete compressive strengths up to 18 ksi. The comparison of the proposed ultimate concrete compressive strain to the test results of this research and other researches in literature is shown in Figure 5-6.

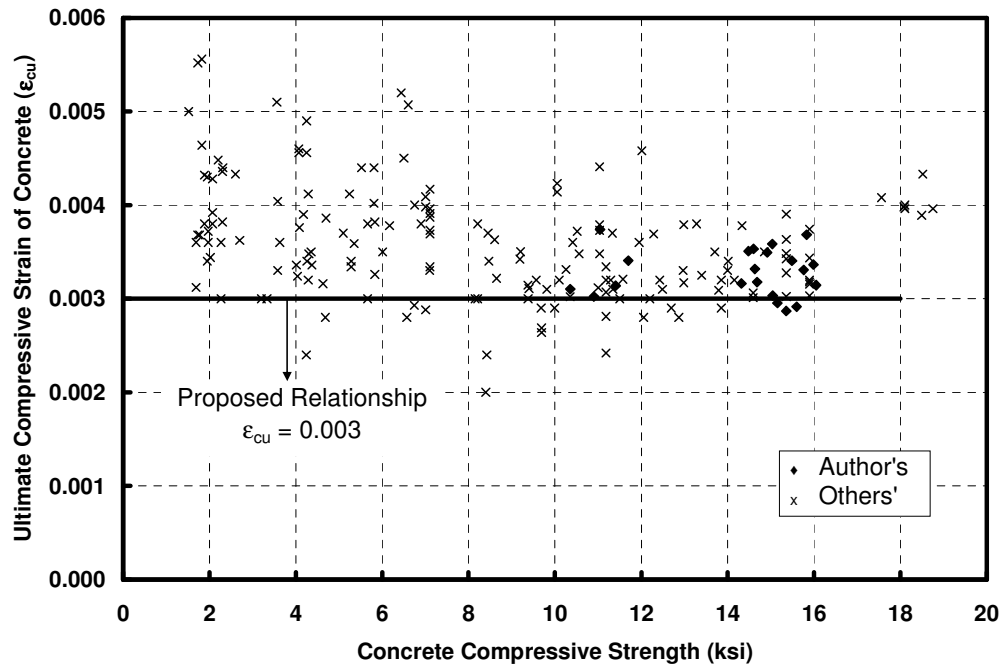


Figure 5-6 – Proposed Relationship for Ultimate Concrete Compressive Strain, ϵ_{cu}

In the light of the previous discussions, the following relationship is proposed for the rectangular stress block parameters, α_l and β_l , for concrete compressive strengths up to 18 ksi using the results of this research and other researchers in the literature.

$$\alpha_1 = \begin{cases} 0.85 & \text{for } f'_c \leq 10 \text{ ksi} \\ 0.85 - 0.02(f'_c - 10) \geq 0.75 & \text{for } f'_c > 10 \text{ ksi} \end{cases} \quad \text{Equation 5-3}$$

$$\beta_1 = \begin{cases} 0.85 & \text{for } f'_c \leq 4 \text{ ksi} \\ 0.85 - 0.05(f'_c - 4) \geq 0.65 & \text{for } f'_c > 4 \text{ ksi} \end{cases} \quad \text{Equation 5-4}$$

at

$$\epsilon_{cu} = 0.003. \quad \text{Equation 5-5}$$

Note that current rectangular stress block parameters specified by AASHTO LRFD Bridge Design Specifications (2004) for concrete compressive strengths below 10 ksi are not affected by the proposed relationships since these stress block parameters have been used for more than half a century for concrete compressive strengths up to 10 ksi. The proposed relationship only reduces α_1 from 0.85 to 0.75 for concrete compressive strengths from 10 to 15 ksi. The comparisons of the proposed relationships and the product of the relationships to the test results of this research and other researchers in the literature are shown in Figure 5-7 to Figure 5-9.

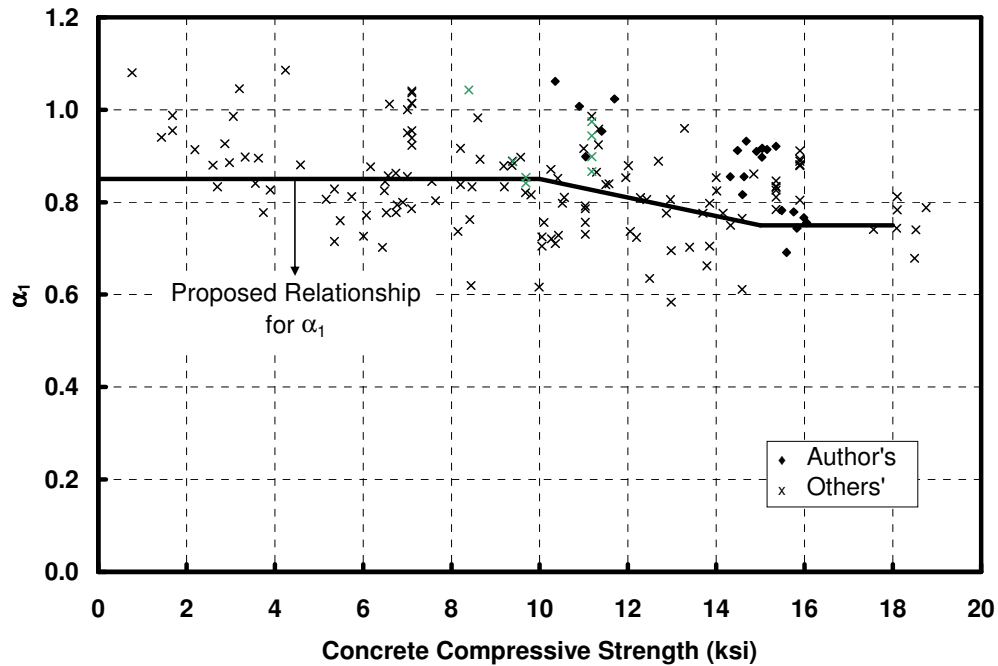


Figure 5-7 – Proposed Relationship for the Rectangular Stress Block Parameters α_1

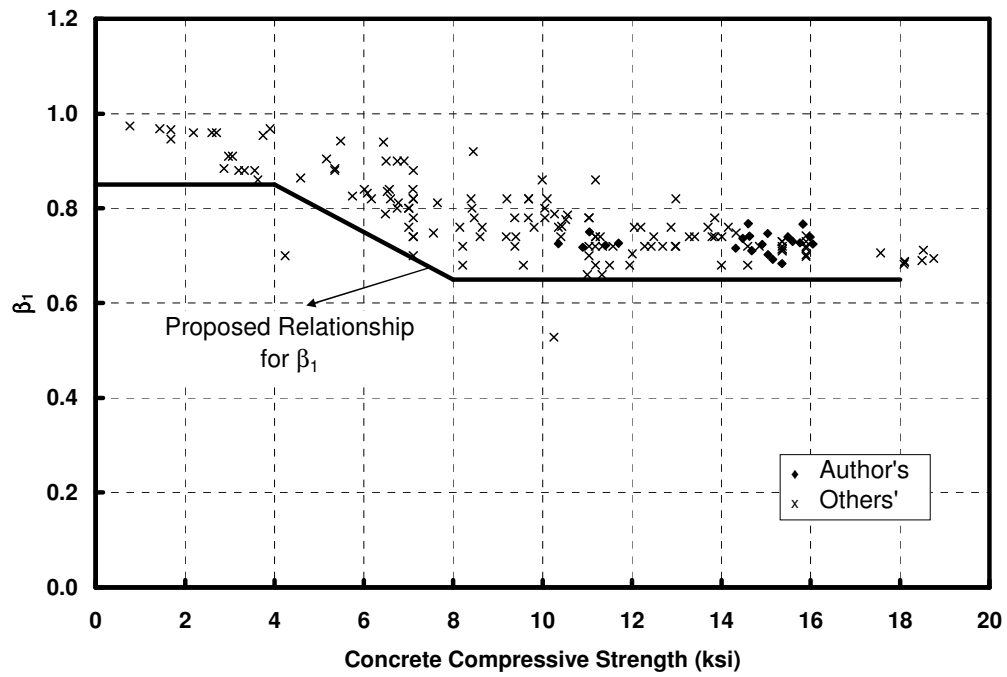


Figure 5-8 – Proposed Relationship for the Rectangular Stress Block Parameters β_1

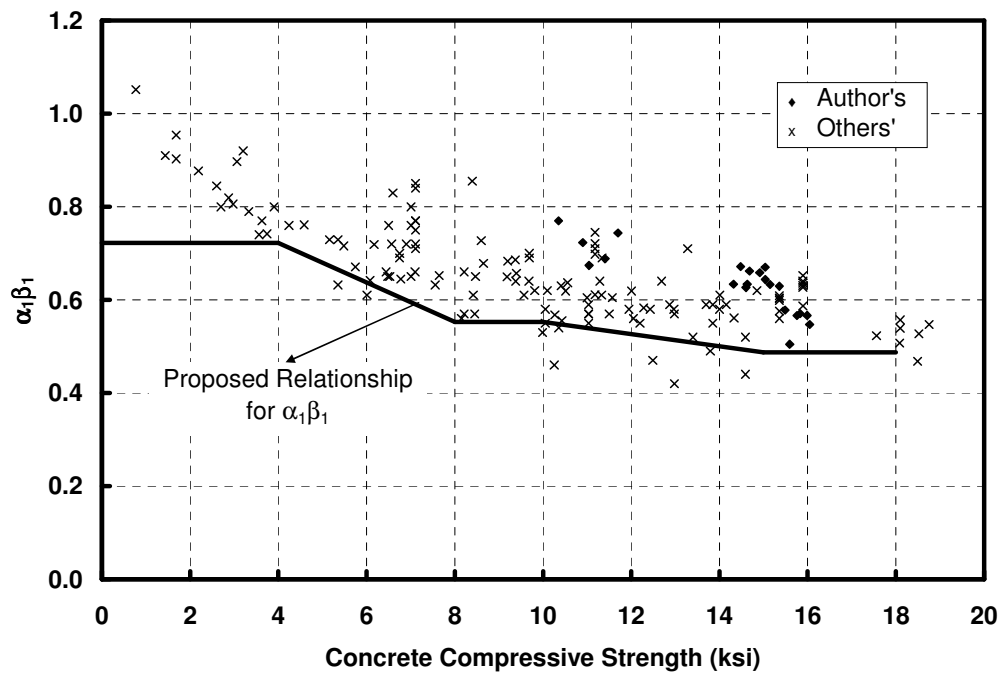


Figure 5-9 – Proposed Relationship for the Product of Rectangular Stress Block Parameters $\alpha_1\beta_1$

5.1.1 Regression Analysis for Rectangular Stress Block Parameters and Ultimate Compressive Strain of Concrete

Regression analysis is a statistical technique to determine the relationship between two or more variables. In this part of the research, a simple linear regression analysis was used to model and investigate the relationship between the concrete compressive strength and characteristics of HSC such as the rectangular stress block parameters and the ultimate compressive strain of concrete. The least square fitting method was used to establish these relationships for the tests results. Details and method about the regression analyses are presented in Appendix H.

A total of 159 eccentric bracket specimen test results with concrete compressive strengths up to 20 ksi were investigated using regression analysis technique to develop the relationship between the rectangular stress block parameters, α_l and β_l and concrete compressive strength, f'_c . The collected data consists of test results obtained by Hognestad et al. (1955), Nedderman (1973), Kaar et al. (1978a, 1978b), Swartz et al. (1985), Pastor (1986), Schade (1992), Ibrahim (1994), Tan and Nguyen (2005) and this research. Sixty nine of the test results have concrete compressive strengths below 10 ksi. The concrete compressive strengths of other test results are above 10 ksi. The data was analyzed for three different concrete compressive strength ranges: below 10 ksi to evaluate the results for NSC, over 10 ksi to evaluate the results for HSC and up to 20 ksi to combine the results to evaluate for NSC and HSC. The tabulated results of the regression analysis for rectangular stress block parameters are presented in Table 5-1. Note that the values given in this table are for 50 percent regression lines corresponding to the medians of the test results. The regression analysis graphs of rectangular stress block parameters for three concrete compressive strength ranges are shown in Figure 5-10 to Figure 5-15. These graphs include the 50 percent regression lines, 90 percent regression lines corresponding to the lower bounds of the 90 percent of the test results and the proposed relationship in this research (Section 5.1) for the rectangular stress block parameters.

Table 5-1 – Tabulated Results of the Regression Analysis for the Rectangular Stress Block Parameters

Concrete Compressive Strength Range (ksi)	Regression Line for α_l			Regression Line for β_l		
	Slope (a)	Intercept (b)	Standard Deviation	Slope (c)	Intercept (d)	Standard Deviation
0 – 10	-0.0117	0.9463	0.0965	-0.0236	0.9849	0.0548
10 – 20	-0.0065	0.9089	0.0919	-0.0045	0.7894	0.0397
0-20	-0.0077	0.923	0.0942	-0.0143	0.9233	0.0521

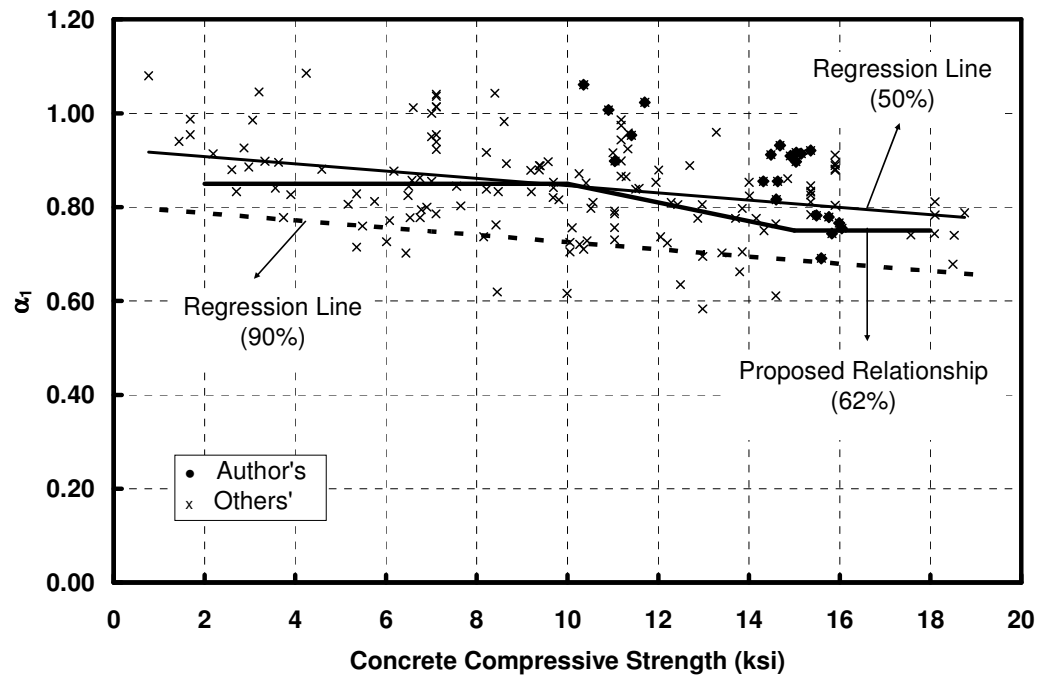


Figure 5-10 – Regression Analysis of α_l for Concrete Compressive Strengths up to 20 ksi

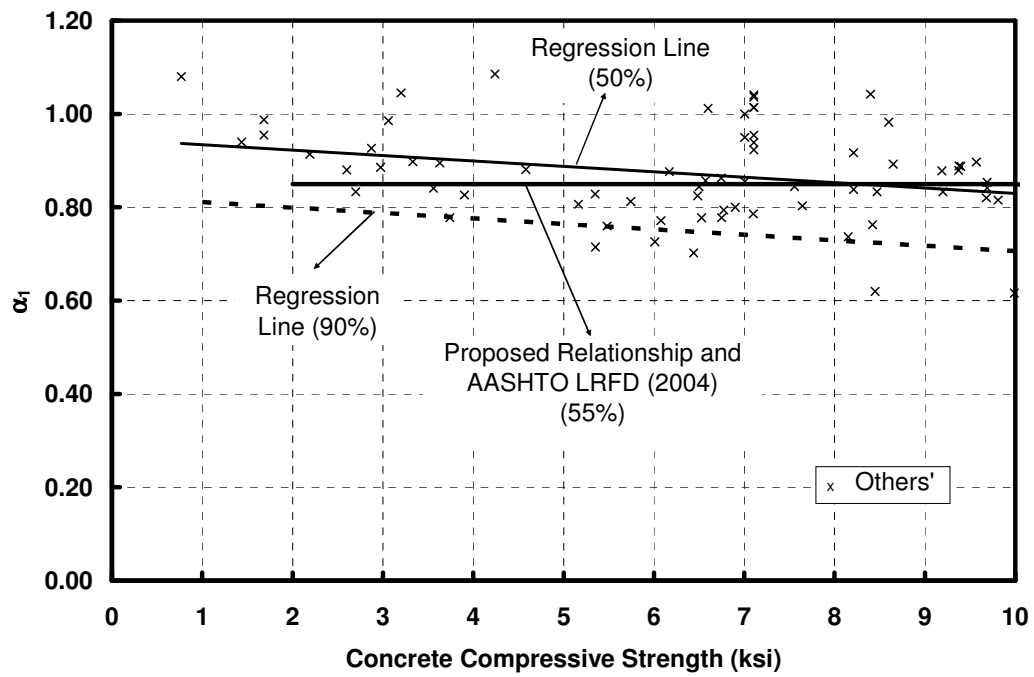


Figure 5-11 – Regression Analysis of α_i for Concrete Compressive Strengths below 10 ksi

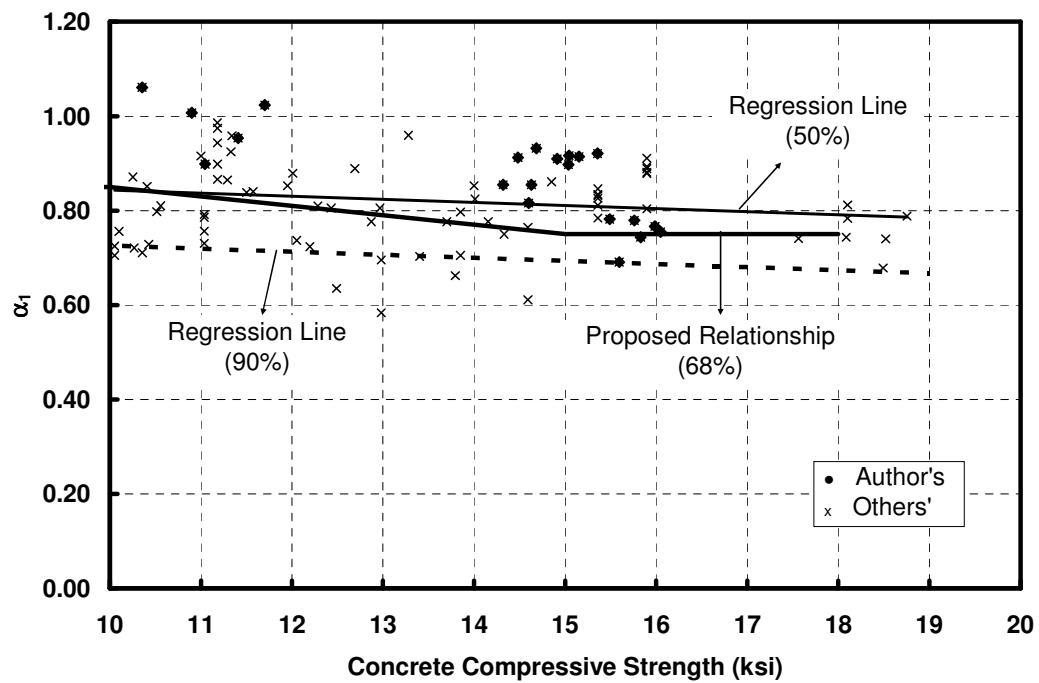


Figure 5-12 – Regression Analysis of α_i for Concrete Compressive Strengths over 10 ksi

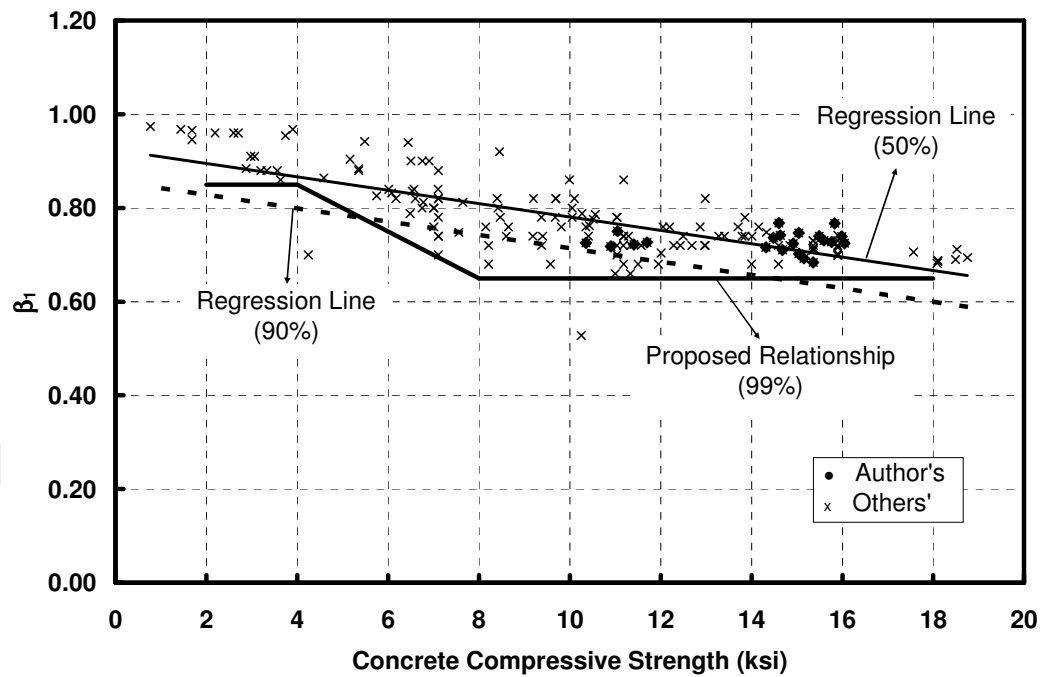


Figure 5-13 – Regression Analysis of β_1 for Concrete Compressive Strengths up to 20 ksi

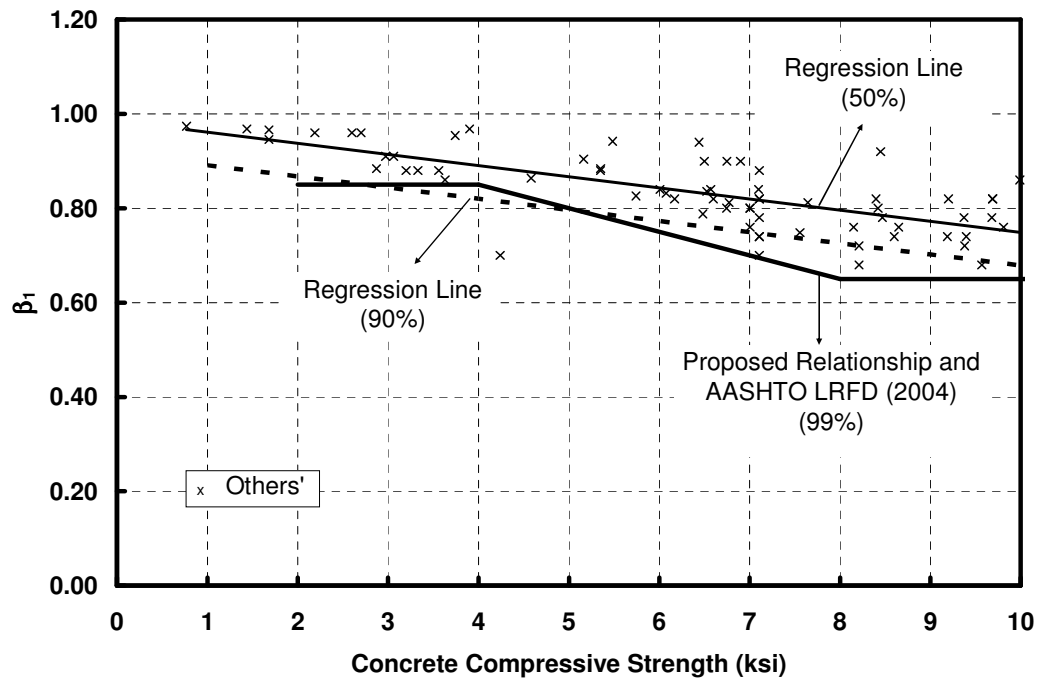


Figure 5-14 – Regression Analysis of β_1 for Concrete Compressive Strengths below 10 ksi

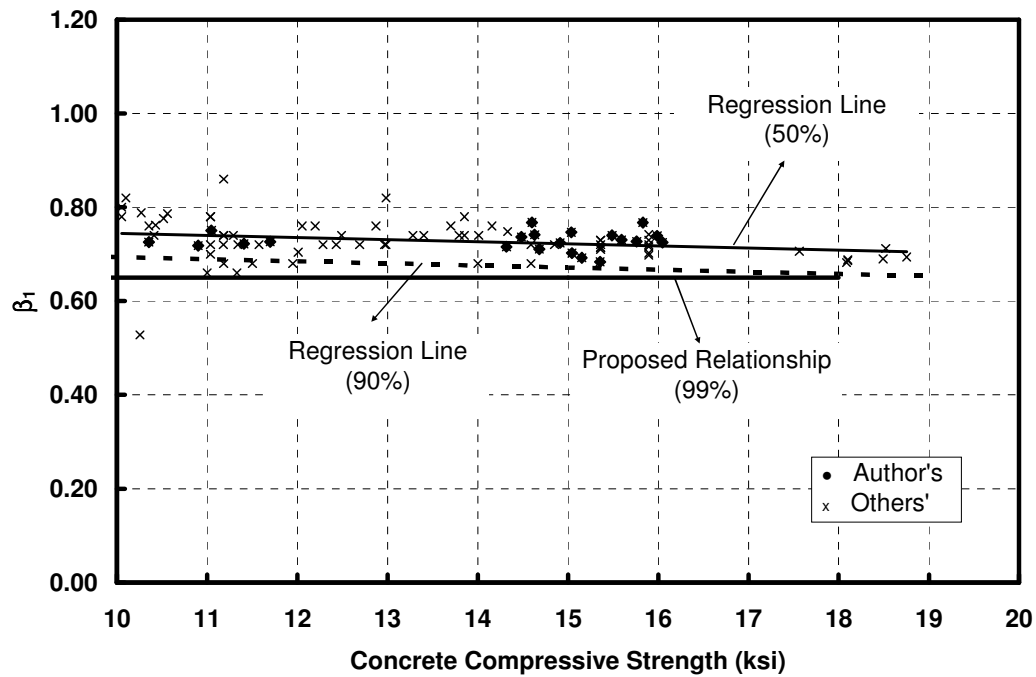


Figure 5-15 – Regression Analysis of β_1 for Concrete Compressive Strengths over 10 ksi

The standard deviations of the rectangular stress block parameter, α_1 , for all three concrete compressive strength ranges (below 10 ksi, over 10 ksi and up to 20 ksi) are very close to each other which indicate that the variability of the α_1 is same for all three ranges. The slope of the regression analysis of test results for α_1 for concrete compressive strengths below 10 ksi is greater than that of test results for concrete compressive strengths over 10 ksi. The results indicate that the proposed relationship for α_1 would seem appropriate for concrete compressive strengths up to 18 ksi.

The standard deviations of the rectangular stress block parameter, β_1 , show more variation for concrete compressive strengths below 10 ksi compared to that of over 10 ksi. Also, the slope of the regression analysis of test results for β_1 for concrete compressive strengths below 10 ksi is greater than that of test results for concrete compressive strengths over 10 ksi. When the test results for concrete compressive strength over 10 ksi are considered, the 90 percent regression line for β_1 becomes almost identical with the proposed equation for HSC. The results indicate that the proposed relationship for β_1 would seem appropriate for concrete compressive strengths up to 18 ksi.

A total of 188 test results with concrete compressive strengths up to 20 ksi under eccentric loading were investigated using regression analysis technique to develop the relationship between the ultimate compressive strain of concrete, ϵ_{cu} , and concrete compressive strength, f'_c . The collected data consists of test results obtained by Hognestad (1951), Sargin (1971), Nedderman (1973), Kaar et al. (1978a, 1978b), Swartz et al. (1985), Pastor (1986), Schade (1992), Ibrahim (1994), Tan and Nguyen (2005) and this research. Hundred and two of the test results have concrete compressive strengths below 10 ksi. The concrete compressive strengths of other test results are over 10 ksi. The data was analyzed for three different concrete compressive strength ranges: below 10 ksi to evaluate the results for NSC, over 10 ksi to evaluate the results for HSC and up to 20 ksi to combine the results to evaluate for NSC and HSC. The tabulated results of the regression analysis for rectangular stress block parameters are presented in Table 5-2. Note that the values given in this table are for 50 percent regression lines corresponding to the medians of the test results. The regression analysis graphs of ultimate compressive strain of concrete for three concrete compressive strength ranges are shown in Figure 5-16 to Figure 5-18. These graphs include the 50 percent regression lines, 90 percent regression lines corresponding to the lower bounds of the 90 percent of the test results and the proposed relationship in this research (Section 5.1) for ultimate compressive strain of concrete.

Table 5-2 – Tabulated Results of the Regression Analysis for Ultimate Compressive Strain of Concrete

Concrete Compressive Strength Range (ksi)	Regression Line for ϵ_{cu}		
	Slope (e)	Intercept (g)	Standard Deviation
0 – 10	-10.85E-5	0.0043	0.0006
10 – 20	2.69E-5	0.0030	0.0004
0 – 20	-3.95E-5	0.0039	0.0005

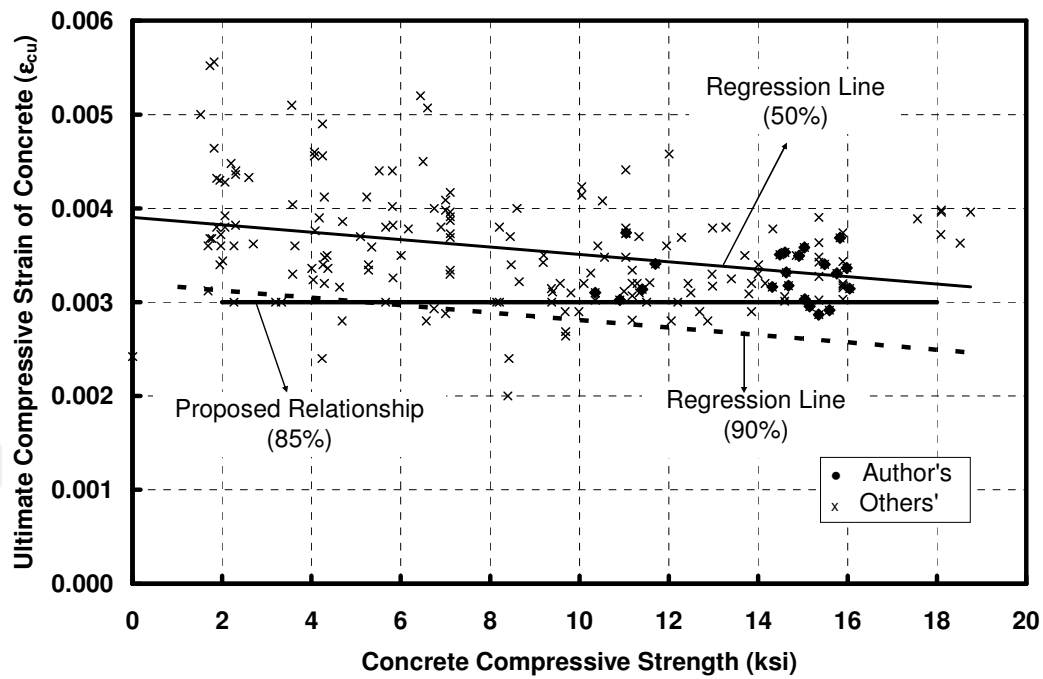


Figure 5-16 – Regression Analysis of ϵ_{cu} for Concrete Compressive Strengths up to 20 ksi

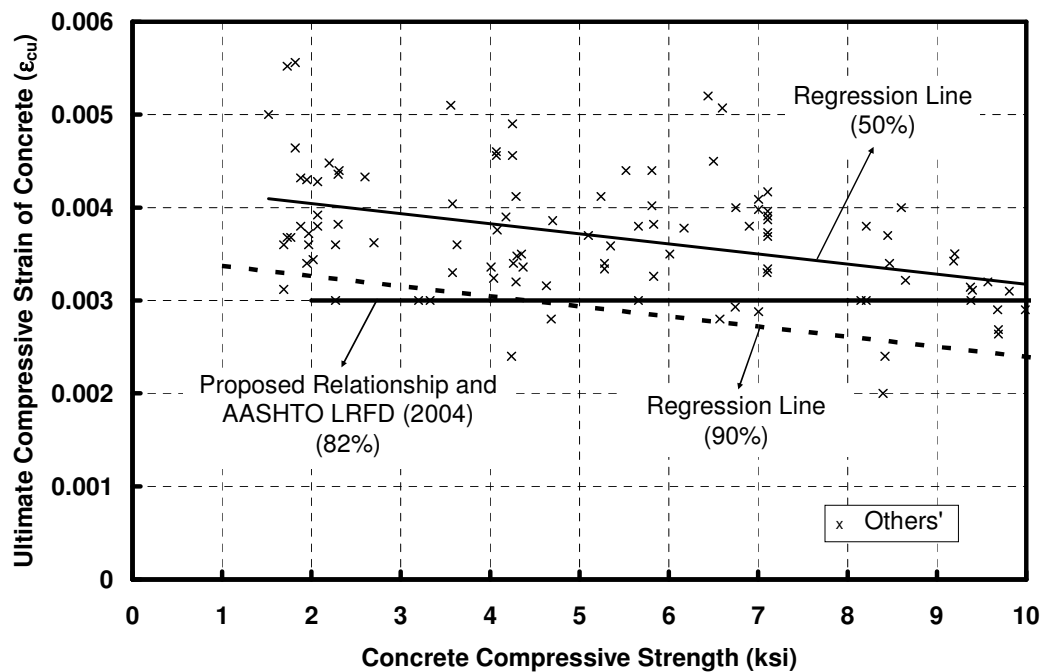


Figure 5-17 – Regression Analysis of ϵ_{cu} for Concrete Compressive Strengths below 10 ksi

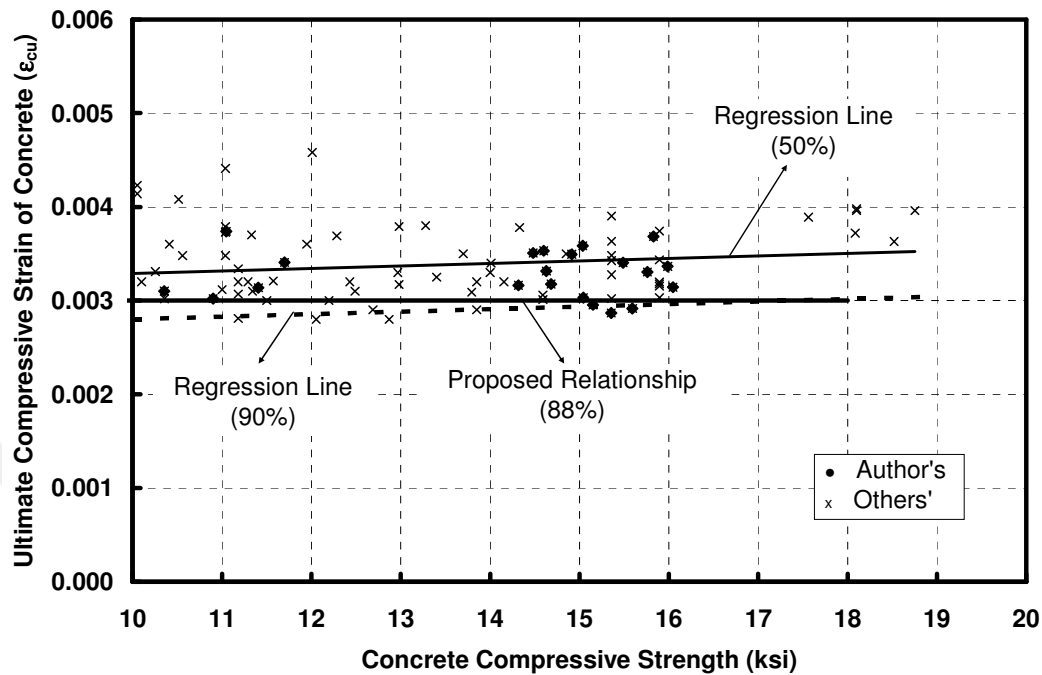


Figure 5-18 – Regression Analysis of ϵ_{cu} for Concrete Compressive Strengths over 10 ksi

The standard deviations of the ultimate compressive strain of concrete, ϵ_{cu} , show more variation for concrete compressive strengths below 10 ksi compared to that of over 10 ksi. Also the slope of the regression analysis of test results for ϵ_{cu} for concrete compressive strengths below 10 ksi is greater than that of for concrete compressive strengths over 10 ksi. When only the test results for concrete compressive strength over 10 ksi are considered, the 90 percent regression line for ϵ_{cu} becomes very close to the proposed relationship for HSC. These results indicate that the proposed equation for ϵ_{cu} would seem appropriate for concrete compressive strengths up to 18 ksi.

5.1.2 Sensitivity Analysis for Rectangular Stress Block Parameters and Ultimate Compressive Strain of Concrete

The sensitivity analysis is a procedure to determine the sensitivity of the result to the changes in its variables. If a small change in a variable provides relatively large changes in the results, the results can be defined to be sensitive to that variable. A sensitivity analysis was performed to evaluate the sensitivity of the ultimate moment capacity of a reinforced concrete member to the rectangular stress block parameters, α_l and β_l , and

ultimate compressive strain of concrete, ϵ_{cu} . The analyses were performed for under-reinforced and over-reinforced concrete sections. For an under-reinforced concrete section, yielding of longitudinal reinforcement occurs before crushing of concrete. For an over-reinforced concrete section, crushing of concrete occurs before yielding of longitudinal reinforcement. Note that for a balanced reinforced concrete section, crushing of concrete and yielding of longitudinal reinforcement occur simultaneously.

The summary of the sensitivity analyses are shown in Table 5-3. The graphical presentations of the sensitivity analyses are shown in Figure 5-19 to Figure 5-21. The results of the sensitivity analysis for flexural under-reinforced concrete sections indicate that the change in the rectangular stress block parameters and the ultimate compressive strain of concrete have very little or no effect on the ultimate moment capacity of the section. On the other hand the results of the sensitivity analysis for the flexural over-reinforced concrete sections indicate that the change in the rectangular stress block parameters affects the ultimate moment capacity of the sections significantly. However, the change in the ultimate compressive strain of concrete has very little effect on the ultimate moment capacity of the section for flexural over-reinforced sections.

Table 5-3 – Summary of the Sensitivity Analysis

Section Type	Reduction in α_f by 11.8% leads to (0.85 $\rightarrow$ 0.75)	Reduction in β_f by 23.5% leads to (0.85 $\rightarrow$ 0.65)	Reduction in ϵ_{cu} by 16.7% leads to (0.003 $\rightarrow$ 0.0025)
Under-Reinforced	1.9% reduction in the Ultimate Moment Capacity	No reduction in the Ultimate Moment Capacity	No reduction in the Ultimate Moment Capacity
Over-Reinforced	10.3% reduction in the Ultimate Moment Capacity	12.2% reduction in the Ultimate Moment Capacity	2.6% reduction in the Ultimate Moment Capacity

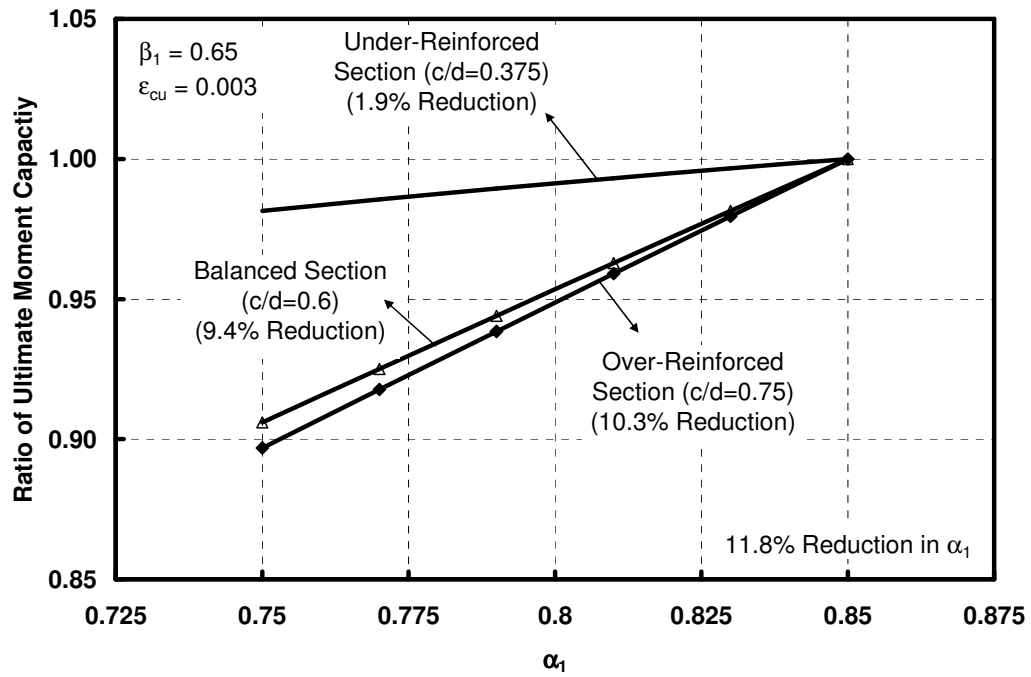


Figure 5-19 - Ratio of Ultimate Moment Capacity versus Change in α_1

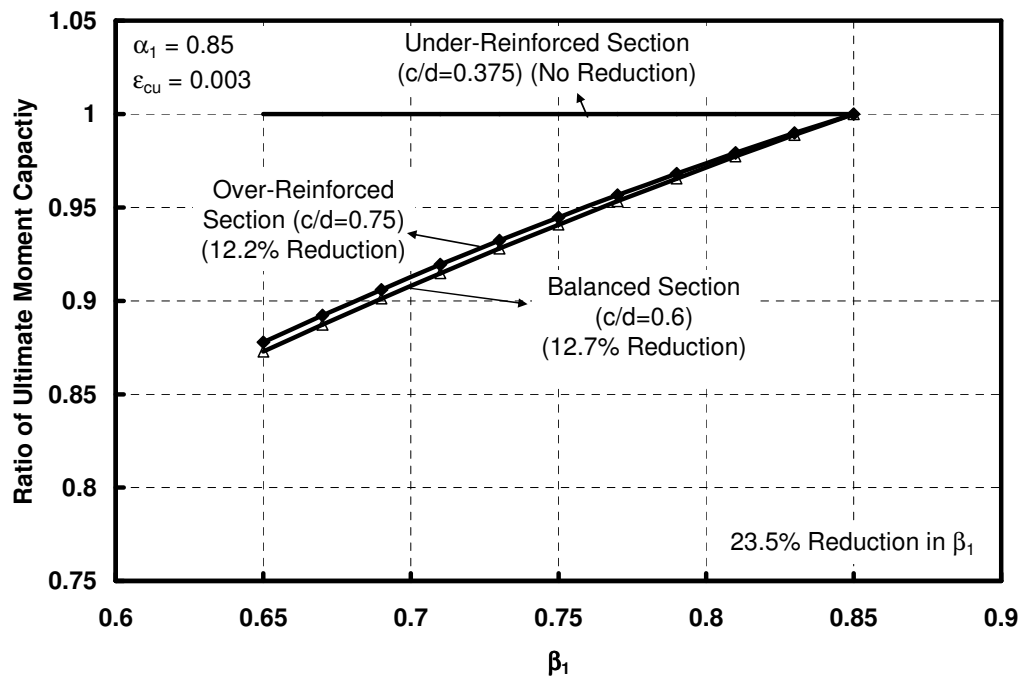


Figure 5-20 - Ratio of Ultimate Moment Capacity versus Change in β_1

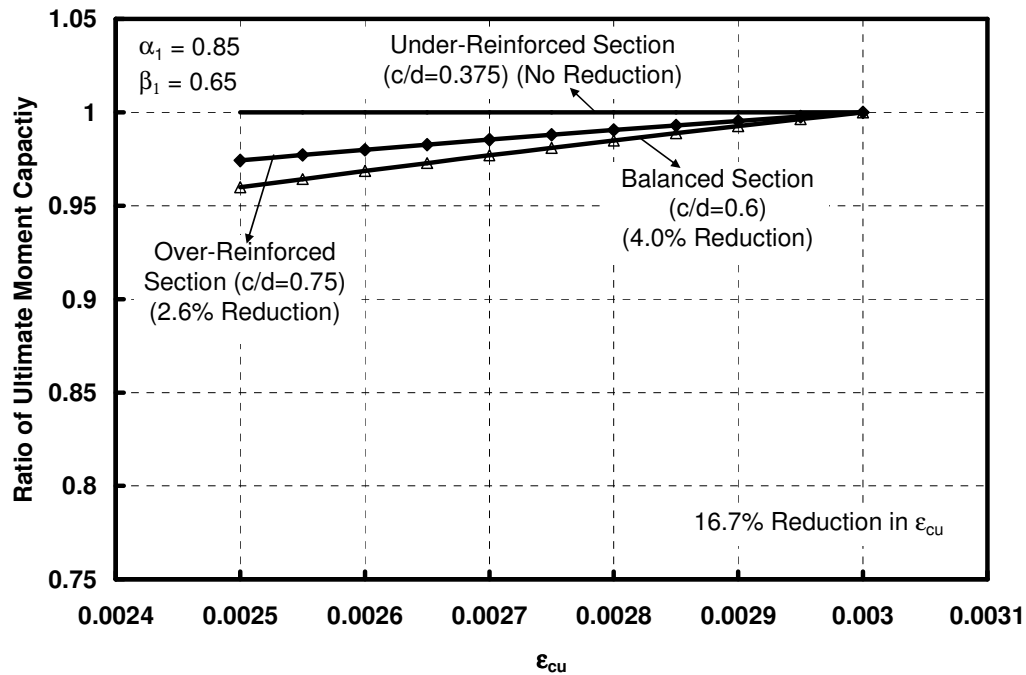


Figure 5-21 - Ratio of Ultimate Moment Capacity versus Change in ϵ_{cu}

5.2 Proposed Stress-Strain Relationship for HSC

For many years, numerous researchers have investigated the stress-strain relationship of concrete. Various researchers proposed different relationships with complicated equations and coefficients. Some researchers suggested two different equations to simulate the ascending and the descending parts of the relationship such as Hognestad (1951), CEB-FIB Model Code (1990), Muguruma (1991) and Van Gysel and Taerwe (1996). Several researchers predicted both the ascending and the descending parts of the stress-strain relationship using only one equation such as a parabola or a curved shape (Sargin and Handa (1969), Popovics (1973), Thorenfeldt et al. (1987), Collins and Porasz (1989) and Attard and Setunge 1996). For the purpose of this research, only one relationship that characterizes both the ascending and the descending branches of stress-strain relationship of HSC was evaluated for comparison purposes. The simplest equation was selected since complicated equations for stress-strain relationships are not preferred by design engineers. Adapting those complicated relationships into design softwares is a significantly difficult process.

5 Analytical Work

The following stress-strain relationship originally proposed by Popovics (1973) and modified by Thorenfeldt et al. (1987) was found to be the most suitable relationship to define the test results of the eccentric bracket specimens tested in this research.

$$f_c = f'_c \frac{\epsilon_c}{\epsilon_{co}} \left[\frac{n}{n-1 + (\epsilon_c/\epsilon_{co})^{nk}} \right] \quad \text{Equation 5-6}$$

where f_c is the stress in general, f'_c is the maximum concrete stress, ϵ_c is the strain in general, ϵ_{co} is the strain at maximum stress and n and k is the coefficients for concrete behavior. Note that, this relationship consists of only one equation that characterizes both the ascending and the descending branches of stress-strain relationship of HSC.

As discussed earlier, using the stress-strain relationships, only the generalized stress block parameters, k_1 and k_2 , can be obtained where k_1 is defined as the ratio of the average compressive stress to the maximum compressive stress and k_2 is the ratio of the depth of the resultant compressive force to the depth of neutral axis. The actual values of these parameters, k_1 and k_2 , were calculated for each of the eccentric bracket specimens and compared to that of obtained from the proposed stress-strain relationship shown in Equation 5-6. The ratios of the calculated and actual parameters were used to determine the equations for the parameters n and k by minimizing the deviations of these ratios from 1. Note that when this ratio is equal to 1, the proposed relationship perfectly matches the actual stress-strain relationship. Actual values of maximum stress, strain at maximum stress and ultimate compressive strain of the stress-strain relationships of concrete obtained from eccentric bracket specimens were used in these calculations. Based on the test results of this research, the following equations producing the minimum deviations from 1 are suggested for the parameters n and k :

$$n = 0.31f'_c + 0.78 \quad (f'_c \text{ in ksi}) \quad \text{Equation 5-7}$$

$$k = 0.10f'_c + 1.20 \quad (f'_c \text{ in ksi}) \quad \text{Equation 5-8}$$

The summary of the actual strain at maximum stress (ϵ_{co}) and actual ultimate compressive strain of concrete (ϵ_{cu}) are shown in Table 5-4. This table also contains the calculated

parameters (k_{1calc} , k_{2calc}) obtained from the proposed stress-stress relationship using the proposed n and k and the actual ϵ_{co} and ϵ_{cu} in comparison to actual parameters (k_{1EBS} , k_{2EBS}) obtained from the eccentric bracket specimens tested in this research. The comparison of the ratios of the calculated to actual values of the stress block parameters k_1 and k_2 are also shown in Figure 5-22. The stress-strain relationships obtained in this research are presented in Appendix F. Sample comparisons of the calculated and the actual stress-strain relationships for eccentric bracket specimens with 10, 14 and 18 ksi target concrete strength are shown in Figure 5-23 to Figure 5-25.

Table 5-4 – Concrete Strain Obtained from Stress-Strain Relationships

Specimen ID	Maximum Stress, f_c (ksi)	Actual Strain at Maximum Stress, ϵ_{co} ($\mu\epsilon$)	Actual Ultimate Strain, ϵ_{cu} ($\mu\epsilon$)	$\frac{k_{1calc}}{k_{1EBS}}$	$\frac{k_{2calc}}{k_{2EBS}}$	$\left(1 - \frac{k_{1calc}}{k_{1EBS}}\right)^2$	$\left(1 - \frac{k_{1calc}}{k_{1EBS}}\right)^2$
10EB1	11.4	3495	3675	0.979	0.935	0.0005	0.0042
10EB2	12.8	3148	3148	1.000	0.949	0.0000	0.0026
10EB3	13.7	3204	3403	0.984	0.962	0.0003	0.0015
10EB4	12.4	2764	3100	1.002	0.997	0.0000	0.0000
10EB5	12.7	2764	2988	1.023	0.985	0.0005	0.0002
14EB1	14.7	2900	3316	0.990	0.988	0.0001	0.0001
14EB2	15.5	2983	3162	1.028	0.971	0.0008	0.0009
14EB3	16.0	2900	3177	1.015	0.996	0.0002	0.0000
14EB4	16.6	2900	3032	1.036	0.980	0.0013	0.0004
14EB5	17.5	2868	2868	1.061	0.990	0.0037	0.0001
14EB6	17.1	2954	2954	1.047	0.978	0.0022	0.0005
18EB1	13.1	2963	3531	0.916	0.989	0.0070	0.0001
18EB2	13.8	3200	3327	0.943	0.936	0.0033	0.0041
18EB3	12.6	2641	2872	1.019	0.972	0.0004	0.0008
18EB4	14.1	3109	3300	0.982	0.960	0.0003	0.0016
18EB5	13.7	3027	3094	0.955	0.949	0.0020	0.0026
18EB6	13.7	3027	3388	0.965	0.975	0.0012	0.0006
18EB7	15.9	3012	3586	0.957	1.023	0.0018	0.0005
18EB8	14.9	3012	3503	0.949	1.010	0.0026	0.0001
18EB9	15.8	3277	3487	0.984	0.961	0.0002	0.0015
18EB10	14.2	2963	3520	0.968	0.987	0.0010	0.0002
Deviation from 1 ($\sqrt{\frac{\sum \left(1 - \frac{k_{calc}}{k_{EBS}}\right)^2}{21}}$)						0.0375	0.0328

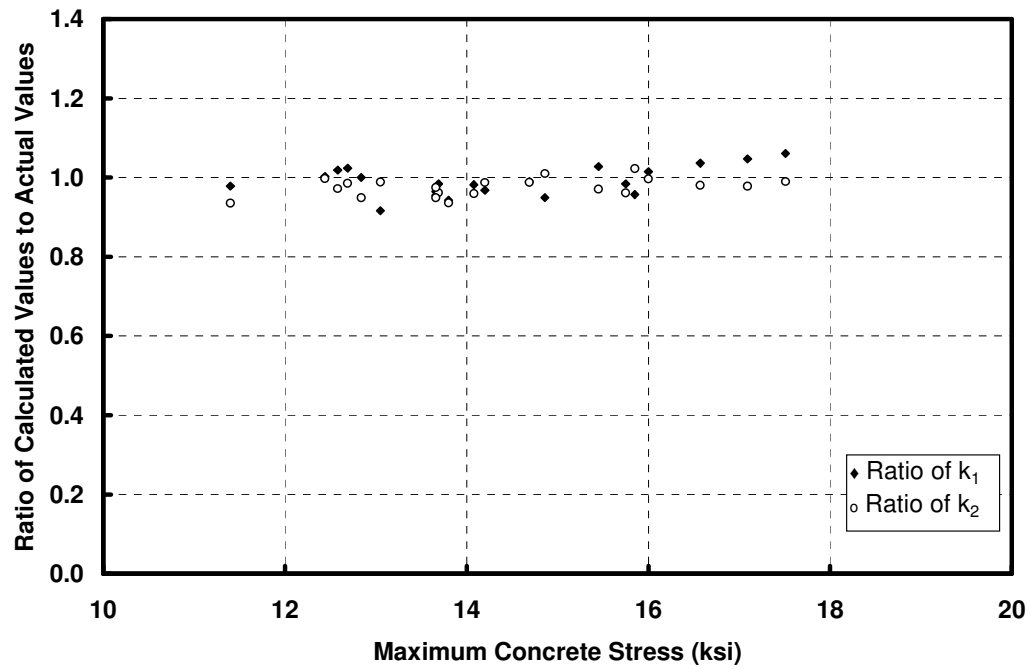


Figure 5-22 – Comparison of the Ratio of Calculated and Actual Values using Actual Values for ϵ_{co} and ϵ_{cu}

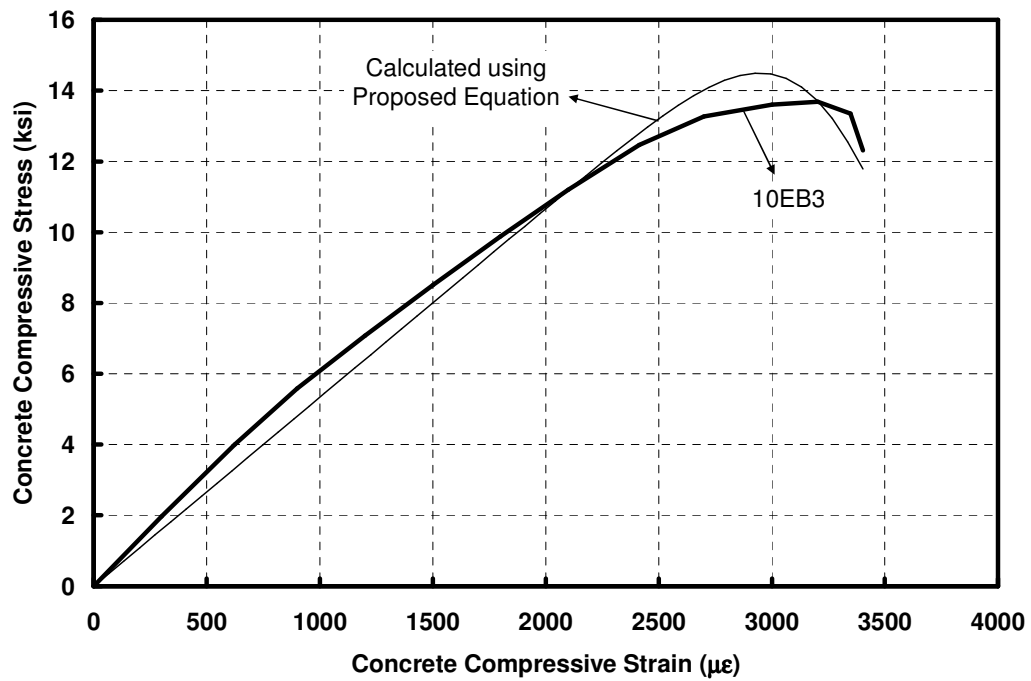


Figure 5-23 – Comparison of Calculated and Actual Stress-Strain Relation of 10EB3

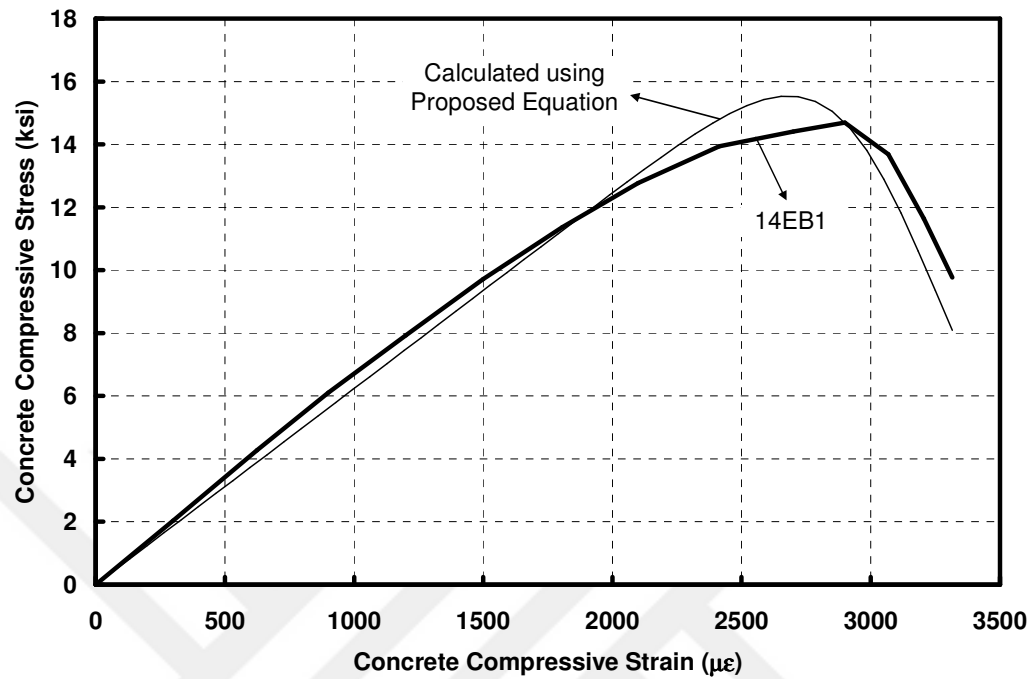


Figure 5-24 – Comparison of Calculated and Actual Stress-Strain Relation of 14EB1

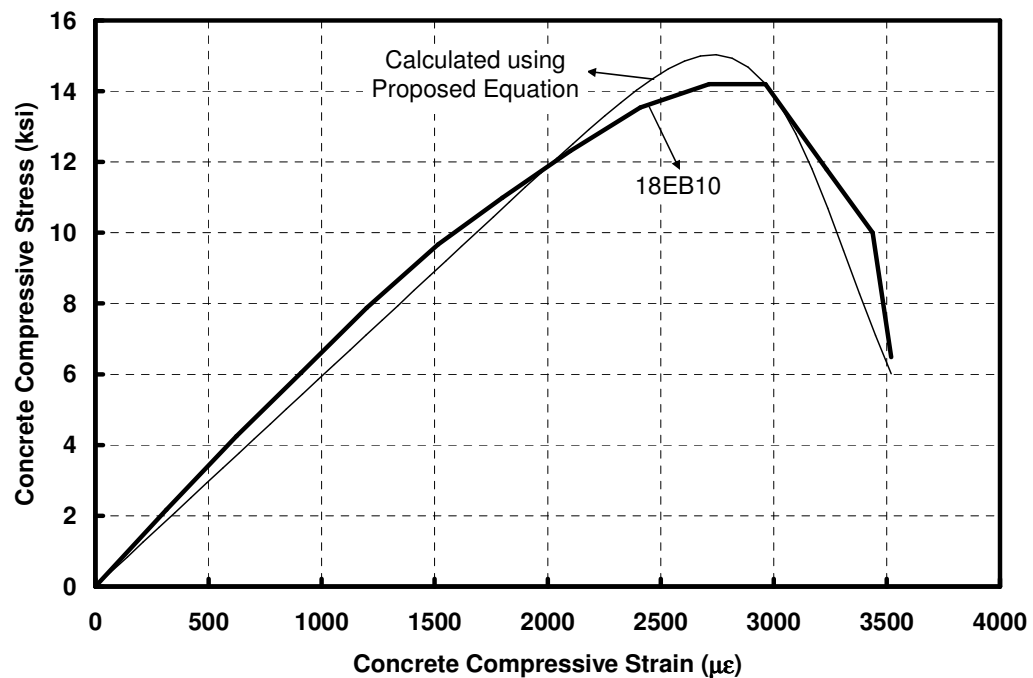


Figure 5-25 – Comparison of Calculated and Actual Stress-Strain Relation of 18EB10

Note that, Collins and Porasz (1989) also proposed n and k relationships based on their research for this stress-strain relationship. The ratios of the calculated parameters obtained from this stress-stress relationship using n and k proposed by Collins and Porasz (1989) and the actual ϵ_{co} and ϵ_{cu} in comparison to actual parameters obtained from the eccentric bracket specimens tested in this research are also evaluated for comparison purposes. The deviations of calculated to actual ratios of k_1 and k_2 from 1 were calculated as 0.0543 and 0.0398, respectively. The comparison of deviations shows that the proposed relationships for n and k in this research provide better predictions for the test results of this research.

A regression analysis was performed on the results shown in Table 5-4 to propose relationships for ultimate compressive strain and strain at maximum stress. The following relationships are suggested as the best fit linear approximation to the test data obtained in this research. These equations are shown in Figure 5-26 and Figure 5-27.

$$\epsilon_{co} = 0.0033 - 2f'_c \times 10^{-5} \quad (f'_c \text{ in ksi}) \quad \text{Equation 5-9}$$

$$\epsilon_{cu} = 0.0038 - 4f'_c \times 10^{-5} \quad (f'_c \text{ in ksi}) \quad \text{Equation 5-10}$$

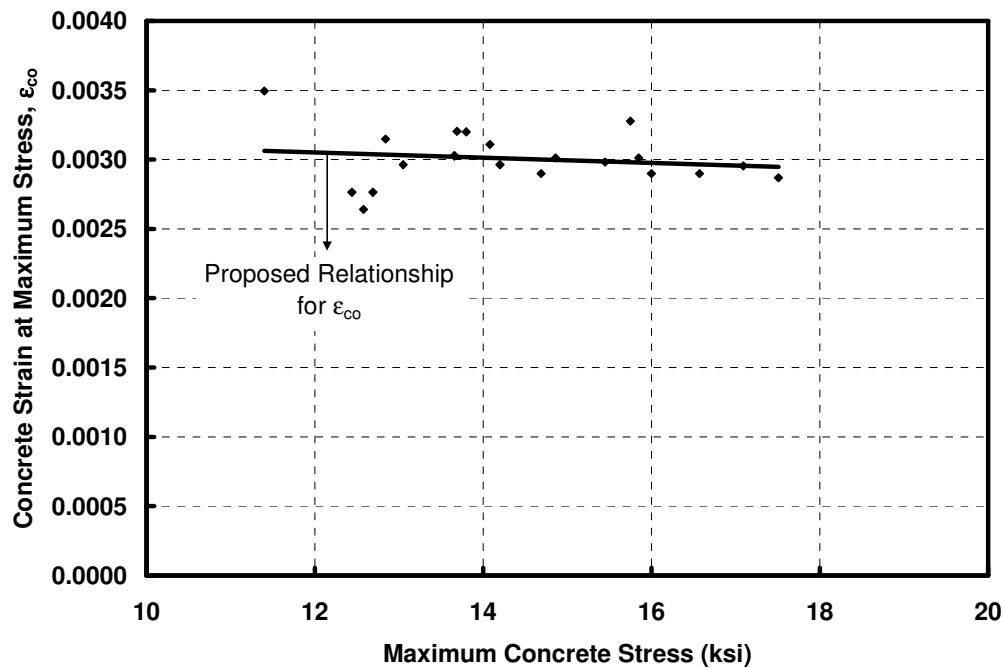


Figure 5-26 – Strain at Maximum Stress versus Concrete Maximum Stress

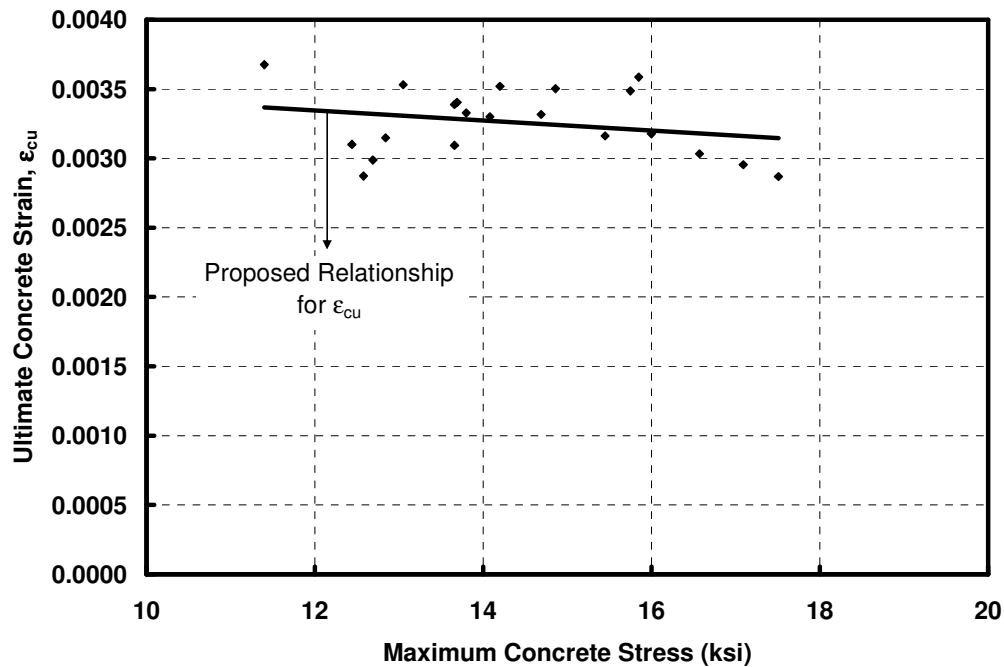


Figure 5-27 – Ultimate compressive strain of concrete versus Concrete Maximum Stress

The summary of the numerical data for the proposed equations for strain at maximum stress (ϵ_{co}) and ultimate strain (ϵ_{cu}) are shown in Table 5-5. This table also contains the comparison of ratios of the calculated values of k_1 and k_2 using the proposed relationships for n , k , ϵ_{co} and ϵ_{cu} in this research to actual values obtained from this research. This comparison is also shown in Figure 5-28. Sample comparisons of the calculated and the actual stress-strain relationships for eccentric bracket specimens with 10, 14 and 18 ksi target concrete strength are shown in Figure 5-29 to Figure 5-31. The proposed relationship yields the minimum deviations from 1 for both of the ratios of k_1 and k_2 . Based on the test results and analysis of this research, the proposed relationship for the stress-strain distribution of HSC can sufficiently be used for design purposes.

Table 5-5 – Concrete Strain Obtained from Stress-Strain Relationships

Specimen ID	Maximum Stress, f_c (ksi)	Calculated Strain at Maximum Stress, ϵ_{co} ($\mu\epsilon$)	Calculated Ultimate Strain, ϵ_{cu} ($\mu\epsilon$)	$\frac{k_{1calc}}{k_{1EBS}}$	$\frac{k_{2calc}}{k_{2EBS}}$	$\left(1 - \frac{k_{1calc}}{k_{1EBS}}\right)^2$	$\left(1 - \frac{k_{1calc}}{k_{1EBS}}\right)^2$
10EB1	11.4	3495	3675	0.902	1.139	0.0097	0.0192
10EB2	12.8	3148	3148	1.083	0.911	0.0069	0.0080
10EB3	13.7	3204	3403	0.944	1.053	0.0031	0.0028
10EB4	12.4	2764	3100	1.060	0.882	0.0036	0.0140
10EB5	12.7	2764	2988	1.127	0.846	0.0163	0.0238
14EB1	14.7	2900	3316	0.952	0.997	0.0023	0.0000
14EB2	15.5	2983	3162	1.035	0.962	0.0013	0.0015
14EB3	16.0	2900	3177	1.001	0.987	0.0000	0.0002
14EB4	16.6	2900	3032	1.077	0.928	0.0059	0.0052
14EB5	17.5	2868	2868	1.179	0.881	0.0321	0.0142
14EB6	17.1	2954	2954	1.137	0.908	0.0188	0.0085
18EB1	13.1	2963	3531	0.849	1.050	0.0229	0.0025
18EB2	13.8	3200	3327	0.932	0.992	0.0046	0.0001
18EB3	12.6	2641	2872	1.168	0.792	0.0281	0.0434
18EB4	14.1	3109	3300	0.966	0.999	0.0011	0.0000
18EB5	13.7	3027	3094	1.027	0.893	0.0007	0.0114
18EB6	13.7	3027	3388	0.919	1.022	0.0065	0.0005
18EB7	15.9	3012	3586	0.851	1.197	0.0222	0.0388
18EB8	14.9	3012	3503	0.865	1.128	0.0181	0.0163
18EB9	15.8	3277	3487	0.894	1.165	0.0112	0.0274
18EB10	14.2	2963	3520	0.889	1.065	0.0123	0.0043
Deviation from 1 ($\sqrt{\frac{\sum \left(1 - \frac{k_{calc}}{k_{EBS}}\right)^2}{21}}$)						0.1042	0.1073

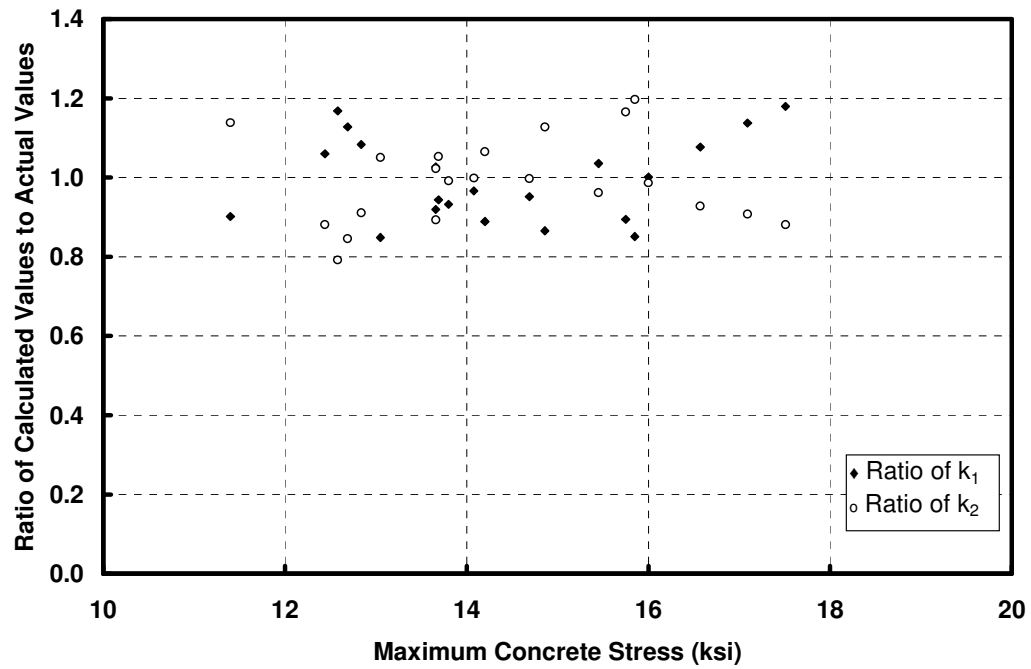


Figure 5-28 – Comparison of the Ratio of Calculated and Actual Values using Proposed Relationships for ϵ_{co} and ϵ_{cu}

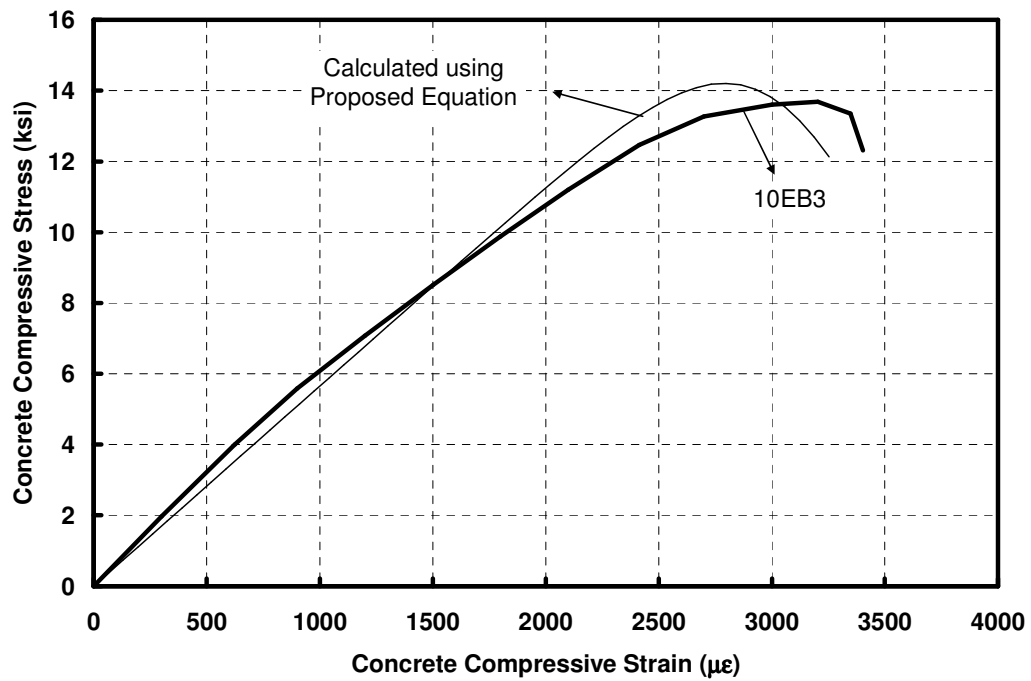


Figure 5-29 – Comparison of Calculated and Actual Stress-Strain Relation of 10EB3

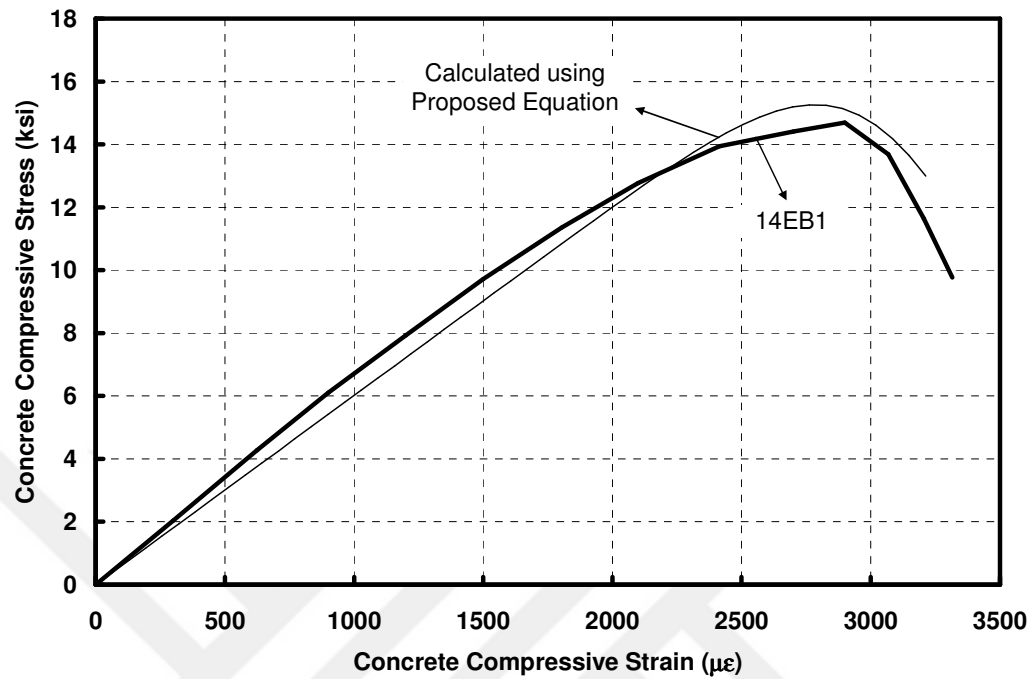


Figure 5-30 – Comparison of Calculated and Actual Stress-Strain Relation of 14EB1

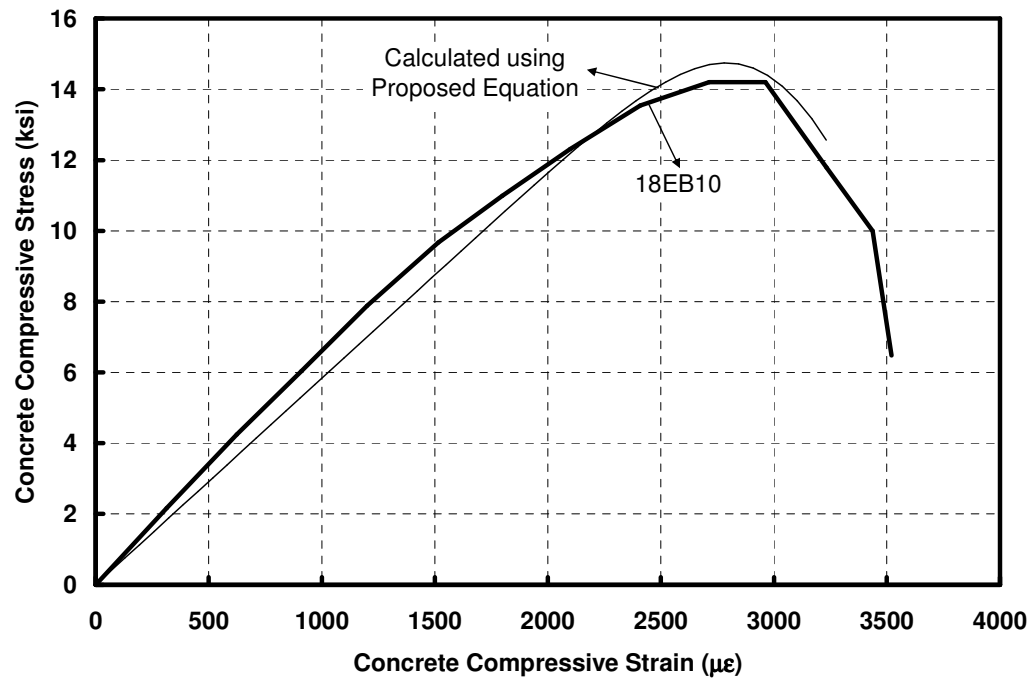


Figure 5-31 – Comparison of Calculated and Actual Stress-Strain Relation of 18EB10

The ratios of the calculated parameters obtained from the proposed stress-stress relationship using n and k proposed by Collins and Porasz (1989) and the proposed ϵ_{co} and ϵ_{cu} relationship in this research in comparison to actual parameters obtained from the eccentric bracket specimens tested in this research are also evaluated for comparison purposes. The deviations for the ratios from 1 of calculated to actual values of k_1 and k_2 were calculated as 0.1084 and 0.1077, respectively. The comparison of deviations shows that the proposed relationships for n and k in this research provide better predictions for the test results of this research.

5.3 Proposed Poisson's Ratio for HSC

Test results in the literature indicate that the Poisson's ratio is between 0.15 to 0.25 with an average value close to 0.2 for concrete compressive strengths up to 18 ksi. Based on the test results of this research and other researchers in the literature, Poisson's ratio of 0.2 was found to be adequately used for concrete compressive strengths up to 18 ksi for design purposes. The comparison of the proposed relationship with the test results of this research and other researchers in the literature is shown in Figure 5-32.

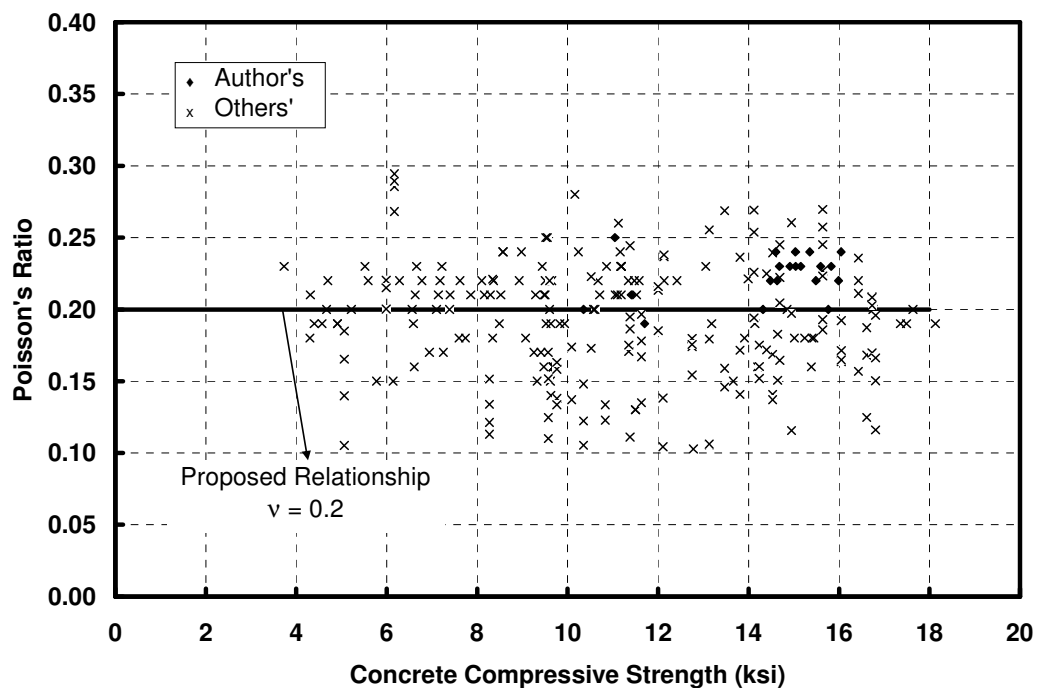


Figure 5-32 – Proposed Relationship for Poisson's Ratio

5.3.1 Regression Analysis for Poisson's Ratio

Similar procedure explained in Appendix H was used to obtain the linear regression relationship for the Poisson's ratio, ν . Details about the regression analysis are presented in Appendix J.

A total of 246 test results with concrete compressive strengths up to 20 ksi were investigated using regression analysis technique to develop the relationship between Poisson's ratio, ν , and concrete compressive strength, f'_c . The collected data consists of test results obtained by Komendant et al. (1978), Perenchio and Klieger (1978), Carrasquillo et al. (1981), Swartz et al. (1985), Jerath and Yamane (1987), Radain et al. (1993), and Iravani (1996) and Logan (2005) and this research. Ninety four of the test results have concrete compressive strengths below 10 ksi. The concrete compressive strengths of other test results are over 10 ksi. The data was analyzed for three different concrete compressive strength ranges: below 10 ksi to evaluate the results for NSC, over 10 ksi to evaluate the results for HSC and up to 20 ksi to combine the results to evaluate for NSC and HSC. The tabulated results of the regression analysis for Poisson's ratio are presented in Table 5-6. Note that the values given in this table are for 50 percent regression lines corresponding to the medians of the test results. The regression analysis graphs of Poisson's ratio for three concrete compressive strength ranges are shown in Figure 5-33 to Figure 5-35. These graphs include the 50 percent regression lines, 90 percent regression lines corresponding to the lower bounds of the 90 percent of the test results and the proposed relationship in this research (Section 5.3) for ultimate compressive strain of concrete.

Table 5-6 – Tabulated Results of the Regression Analysis for Poisson's Ratio

Concrete Compressive Strength Range (ksi)	Regression Line for ν		
	Slope (h)	Intercept (j)	Standard Deviation
0 – 10	-0.00477	0.22957	0.0376
10 – 20	0.00075	0.18345	0.0404
0 – 20	-0.00019	0.19556	0.0397

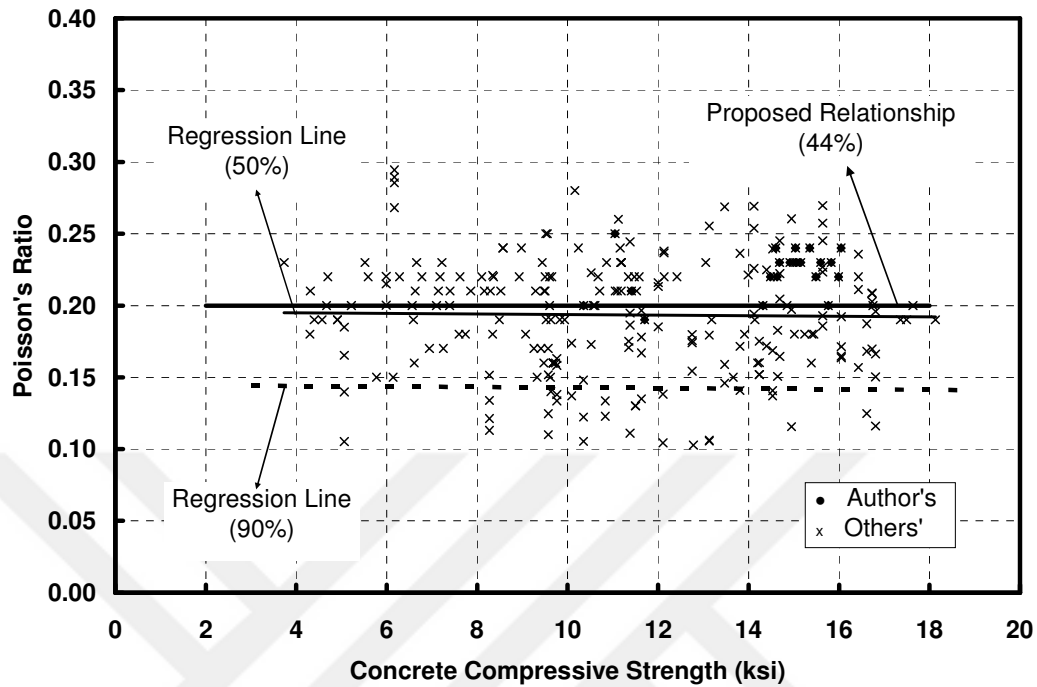


Figure 5-33 – Regression Analysis of ν for Concrete Compressive Strengths up to 20 ksi

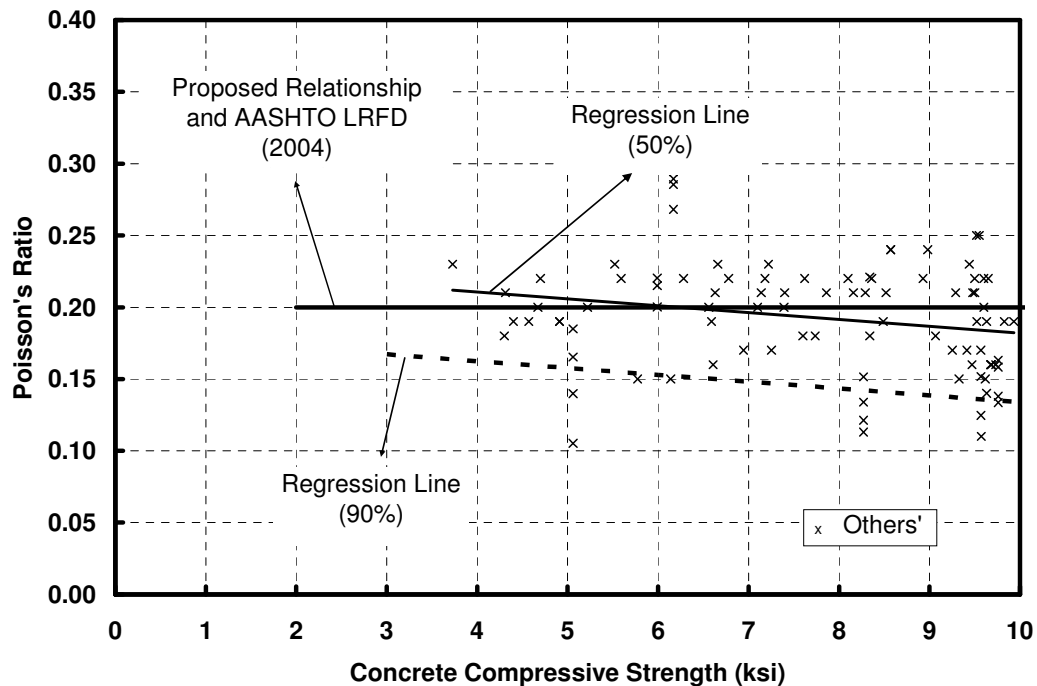


Figure 5-34 – Regression Analysis of ν for Concrete Compressive Strengths below 10 ksi

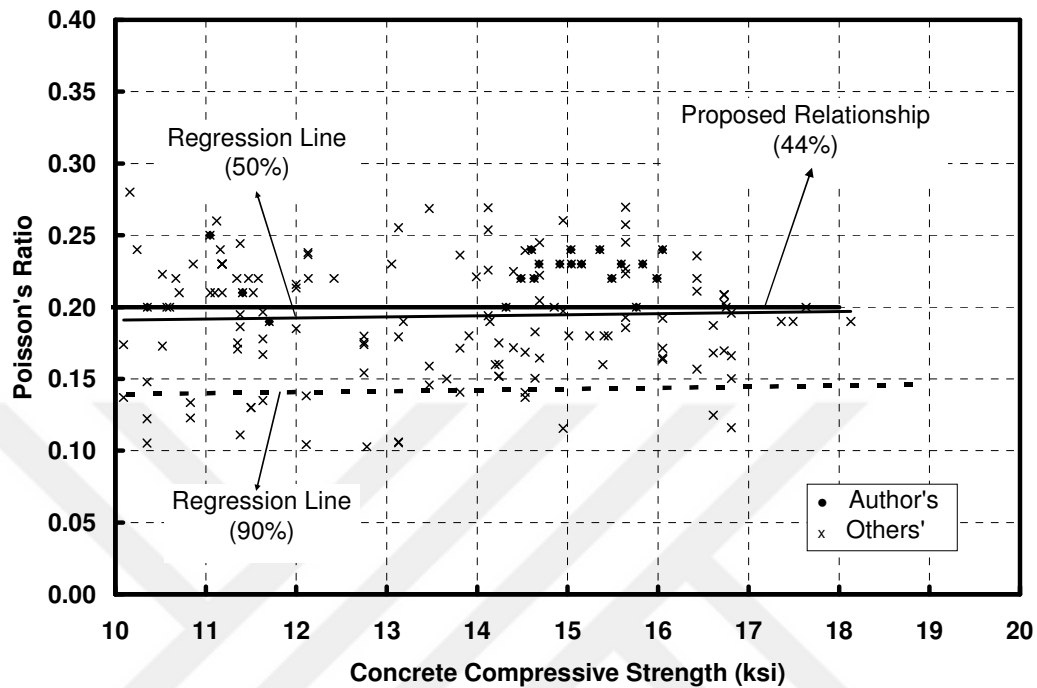


Figure 5-35 – Regression Analysis of ν for Concrete Compressive Strengths over 10 ksi

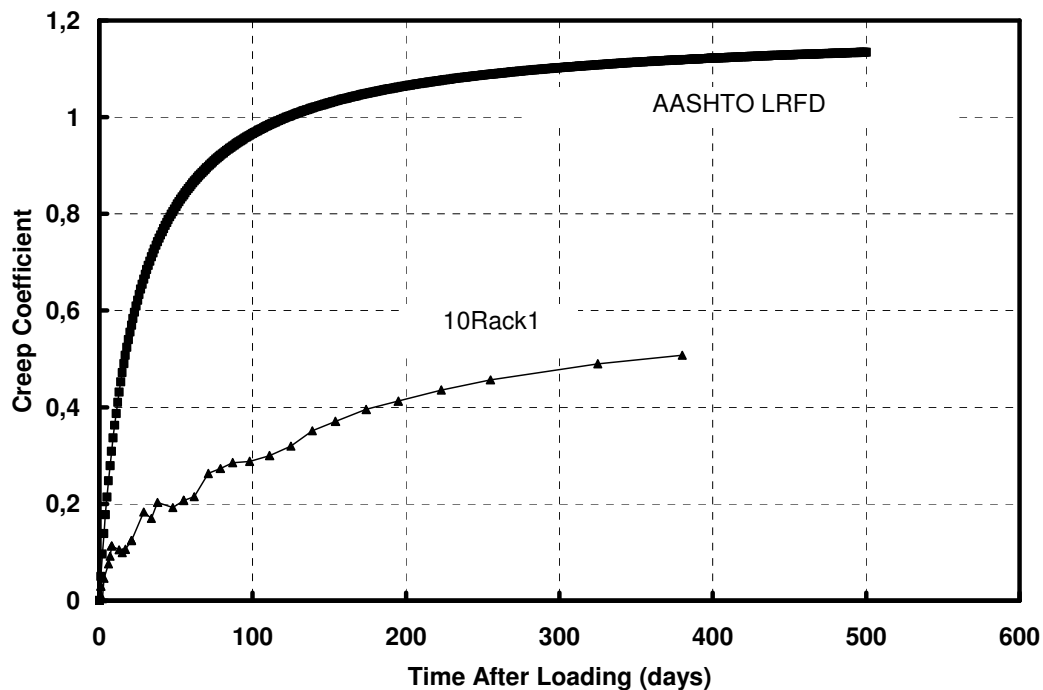
The standard deviations of Poisson's ratio, ν , show similar variations for three of the concrete compressive strength ranges (below 10 ksi, over 10 ksi and up to 20 ksi). However, slope of the regression analysis of the test results of ν with concrete compressive strengths below 10 ksi is decreasing whereas, that of with concrete compressive strengths over 10 ksi is increasing. When only test results of specimens with concrete compressive strengths over 10 ksi are considered, there is an increase in the Poisson's ratio as concrete compressive strength increases. The proposed relationship for Poisson's ratio establishes the lower bound limit of 44 percent of the test results. These results indicate that the proposed equation for ν would seem appropriate for concrete compressive strengths up to 18 ksi.

5.4 Proposed Creep and Shrinkage Relationships for HSC

Based on the results of the literature review, it is evident that creep and shrinkage of HSC is much less than NSC and as concrete strength increases, both the creep and and shrinkage of concrete decreases. Test results obtained from this research were compared

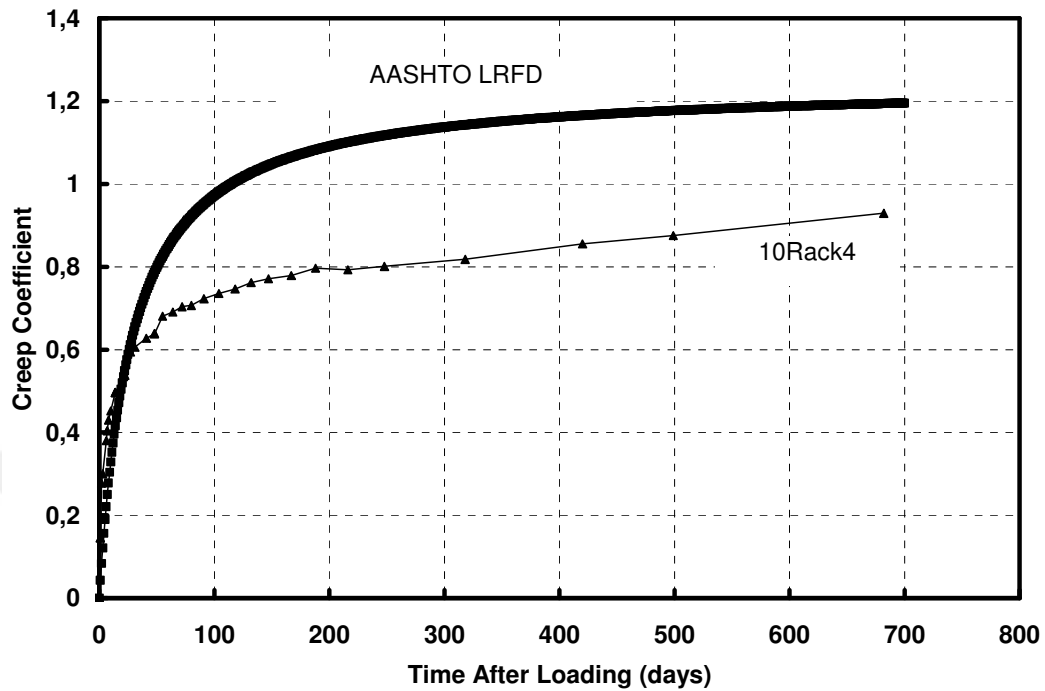
to the predictions specified by different design codes and documents. The relationships specified by AASHTO LRFD Bridge Design Specifications (2004) were found to be the most appropriate relationship to predict creep and shrinkage of HSC with compressive strengths less than or equal to 15 ksi.

Comparisons of adjusted creep measurements to the creep equations specified by AASHTO LRFD Bridge Design Specification (2004) are shown in Figure 5-36 to Figure 5-47. Since this relationship is designed to predict the creep coefficient, which is the ratio of the creep strain at a given time to the initial elastic strain of the specimen at the initial loading stage, the comparisons are only presented in terms of the creep coefficients. The creep coefficient specified by AASHTO LRFD Bridge Design Specifications (2004) can be found in Section 2.4.2.3.



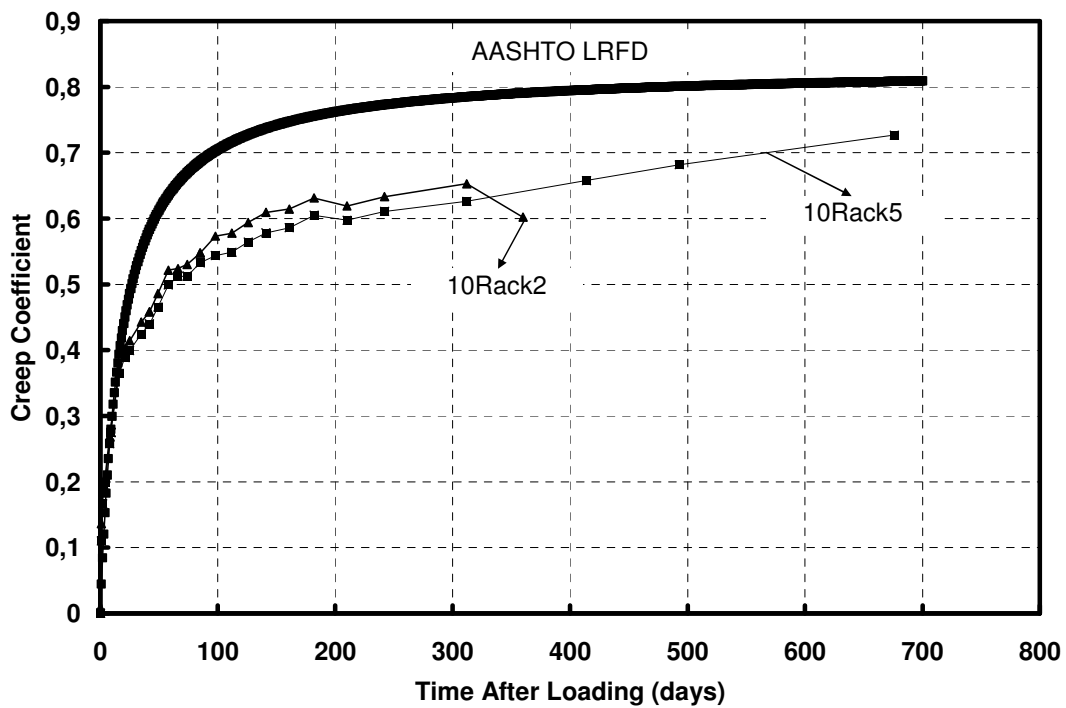
(10 ksi Target Concrete Compressive Strength, Heat Cured, Loaded on the 1st Day)

Figure 5-36 – Comparison of Creep Coefficient of 10Rack1



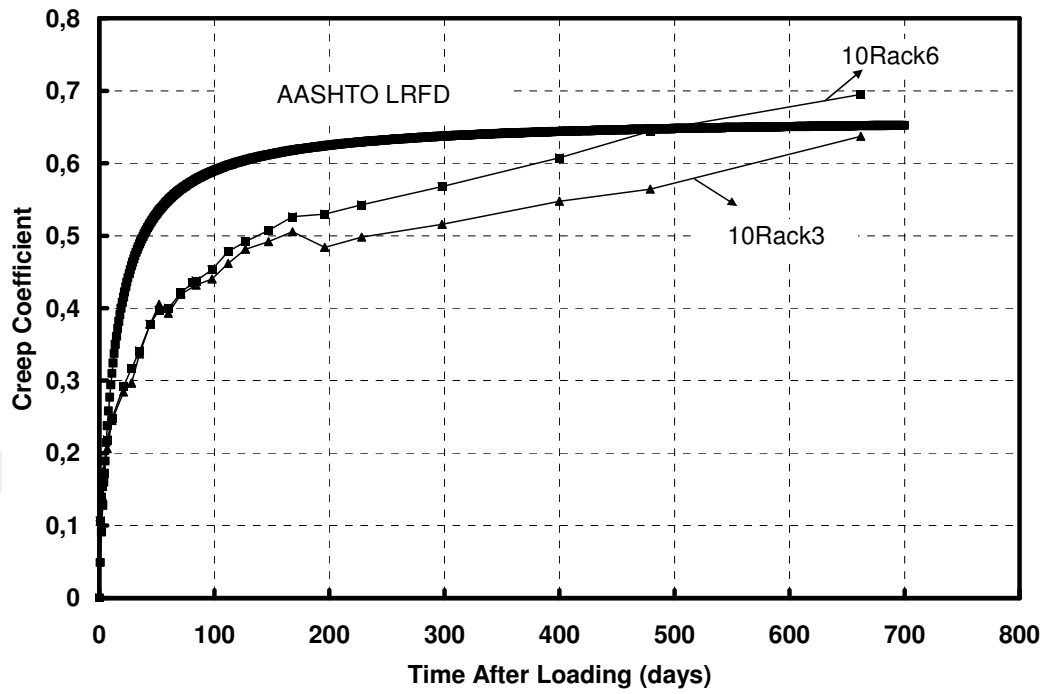
(10 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 8th Day)

Figure 5-37 – Comparison of Creep Coefficient of 10Rack4



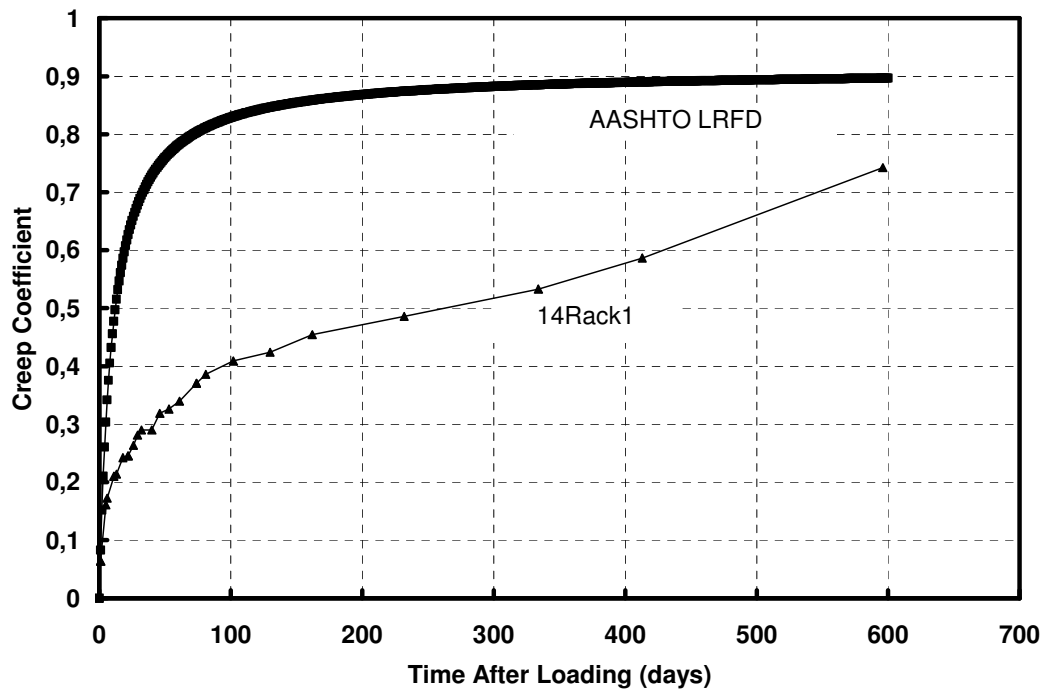
(10 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 14th Day)

Figure 5-38 – Comparison of Creep Coefficient of 10Rack2 and 10Rack 5



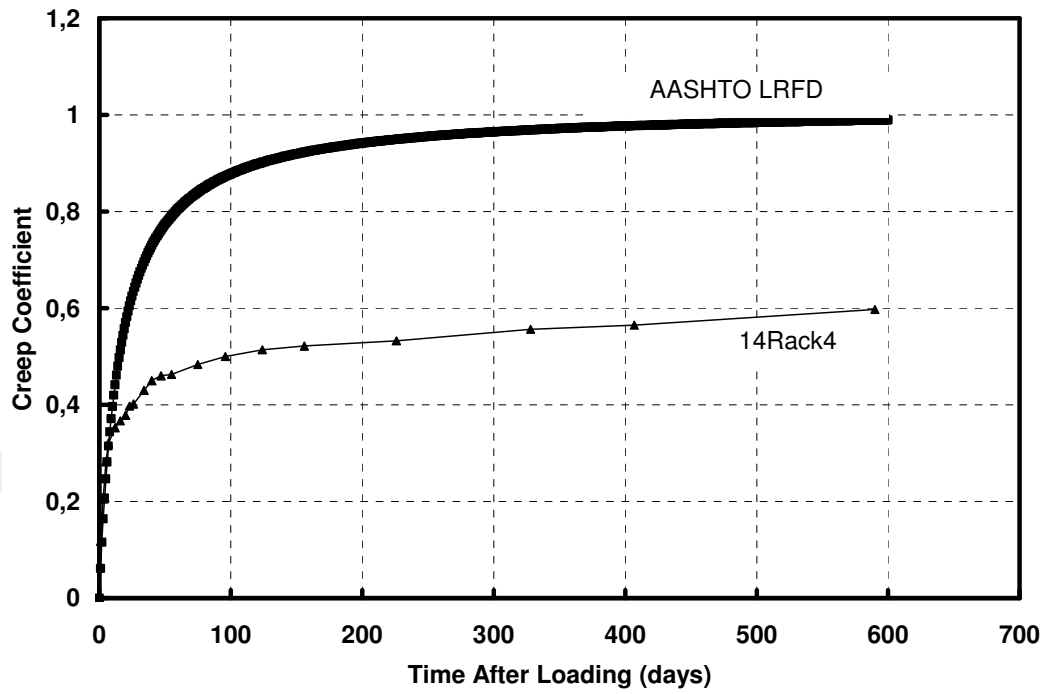
(10 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 28th Day)

Figure 5-39 – Comparison of Creep Coefficient of 10Rack3 and 10Rack6



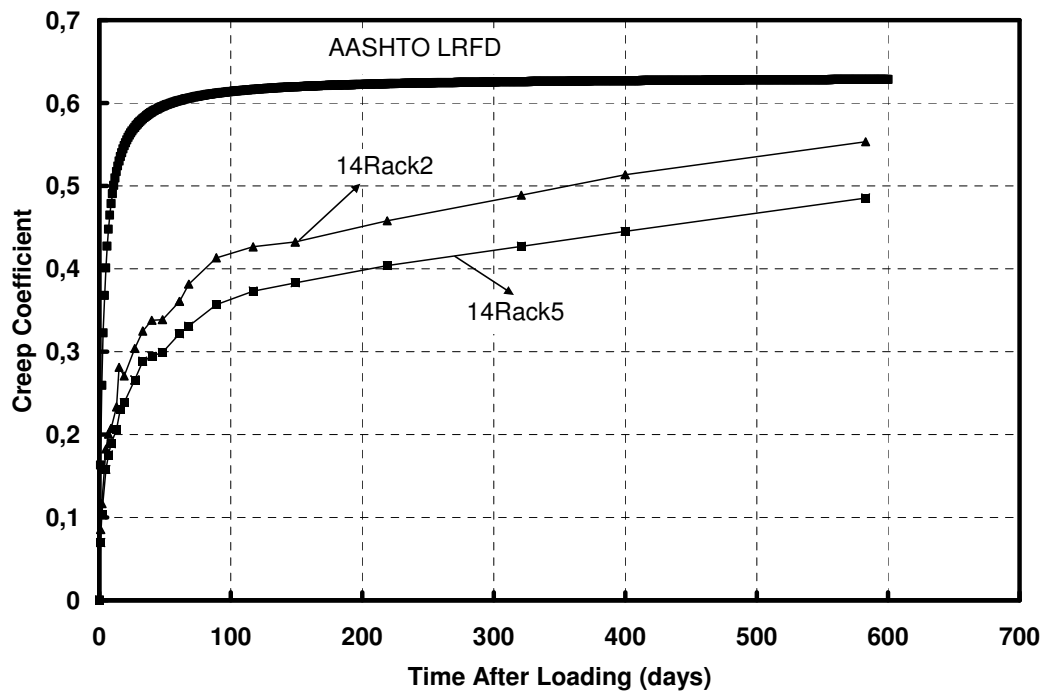
(14 ksi Target Concrete Compressive Strength, Heat Cured, Loaded on the 1st Day)

Figure 5-40 – Comparison of Creep Coefficient of 14Rack1



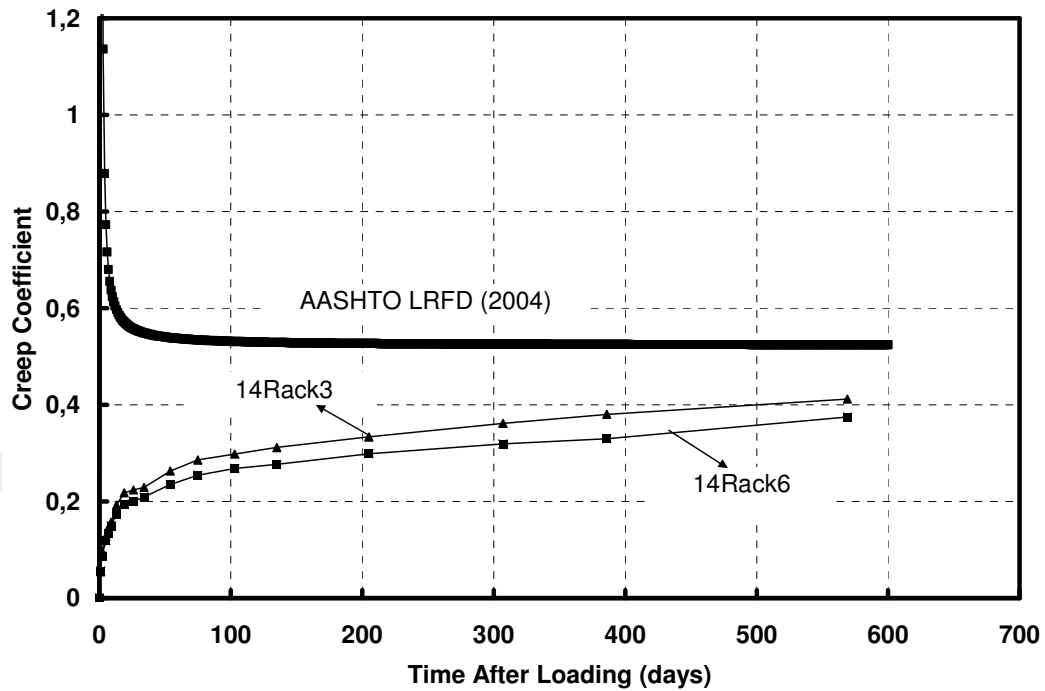
(14 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 7th Day)

Figure 5-41 – Comparison of Creep Coefficient of 14Rack4



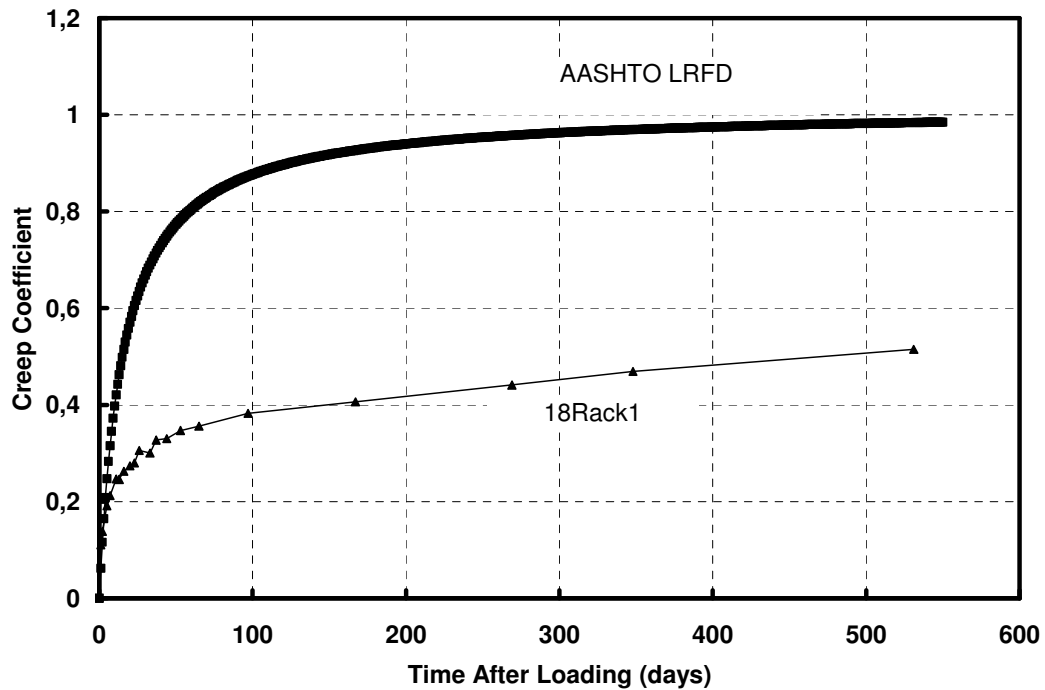
(14 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 14th Day)

Figure 5-42 – Comparison of Creep Coefficient of 14Rack2 and 14Rack5



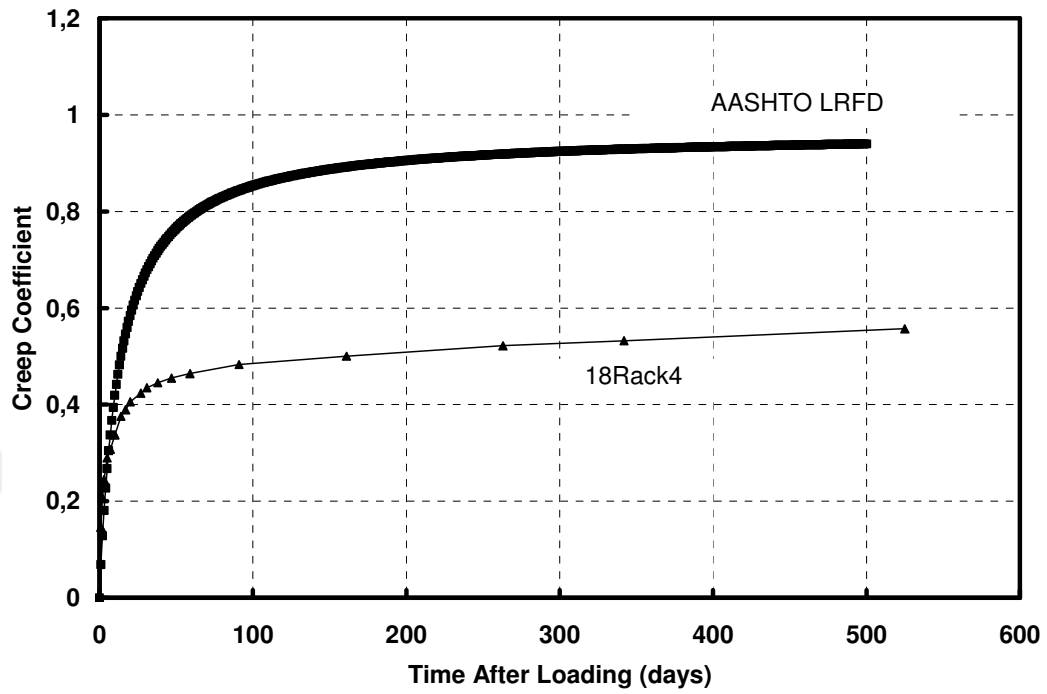
(14 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 28th Day)

Figure 5-43 – Comparison of Creep Coefficient of 14Rack3 and 14Rack6



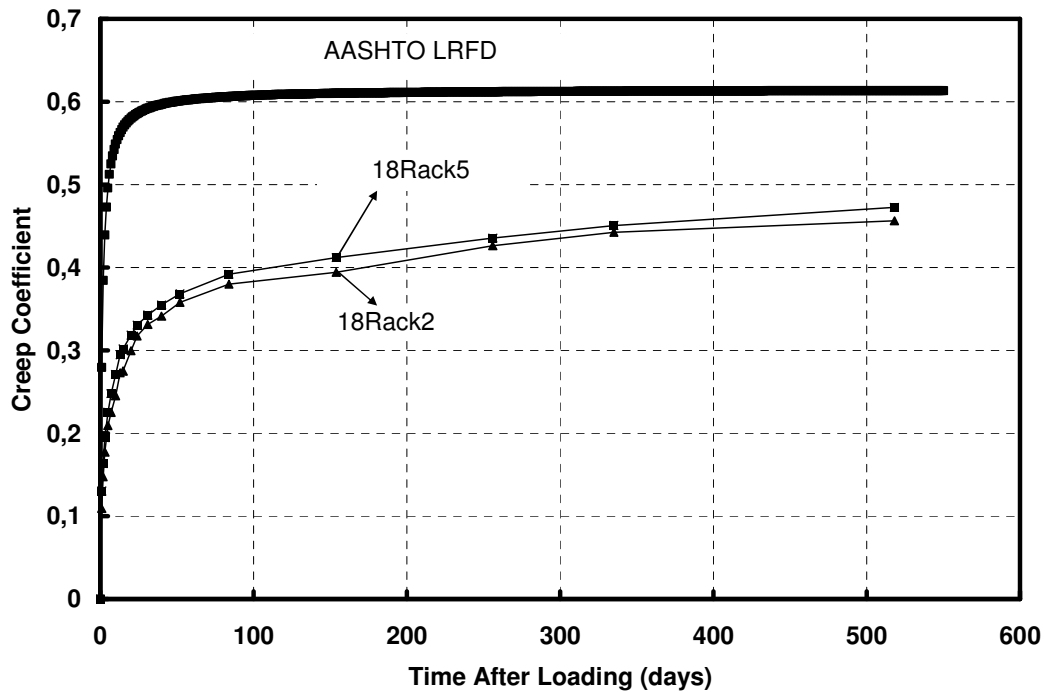
(18 ksi Target Concrete Compressive Strength, Heat Cured, Loaded on the 1st Day)

Figure 5-44 – Comparison of Creep Coefficient of 18Rack1



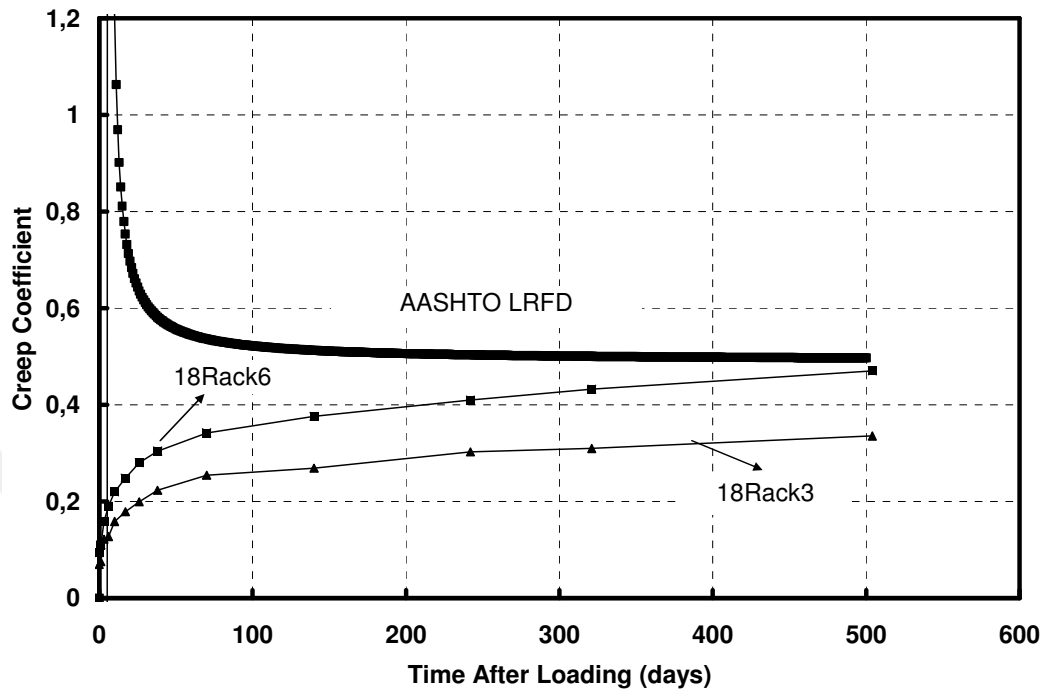
(18 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 7th Day)

Figure 5-45– Comparison of Creep Coefficient of 18Rack4



(18 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 14th Day)

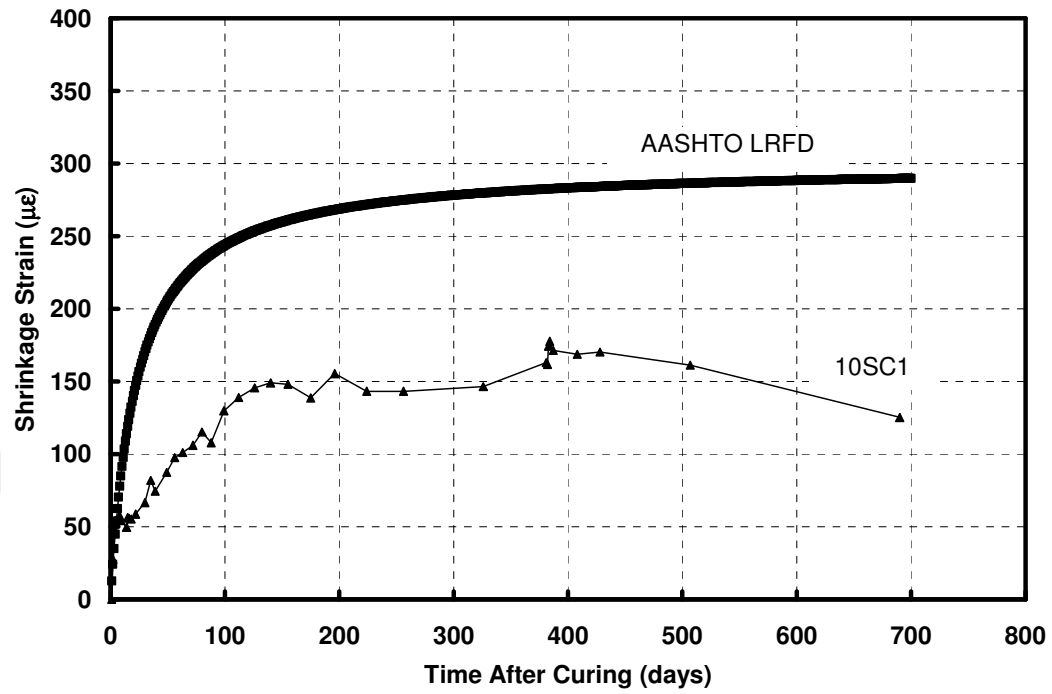
Figure 5-46 – Comparison of Creep Coefficient of 18Rack2 and 18Rack5



(18 ksi Target Concrete Compressive Strength, Moist Cured, Loaded on the 28th Day)

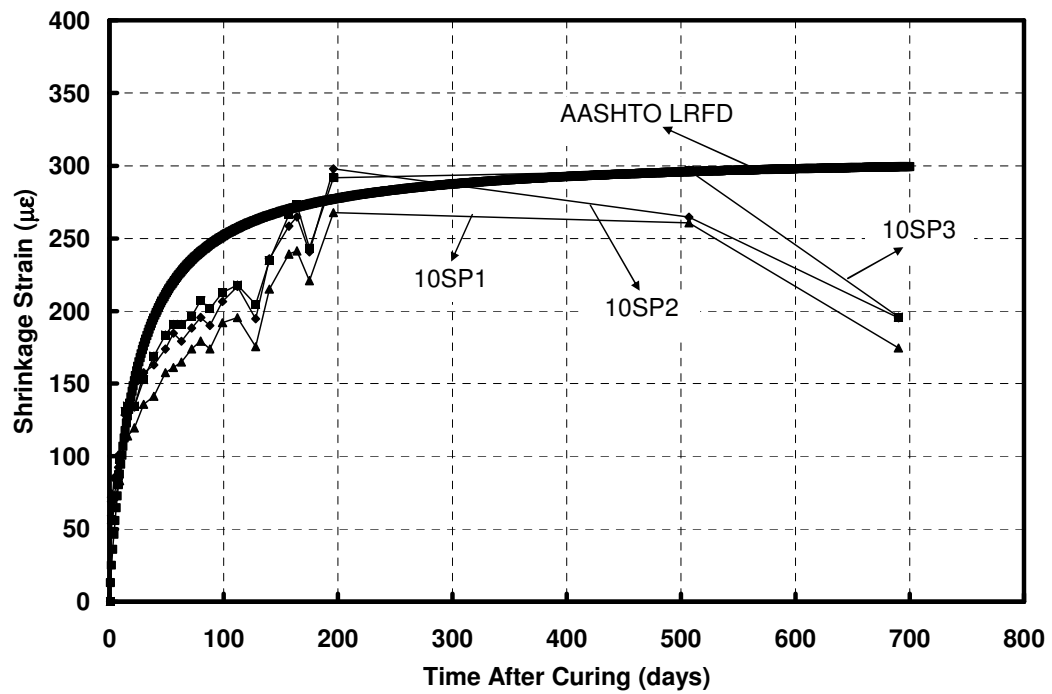
Figure 5-47 – Comparison of Creep Coefficient of 18Rack3 and 18Rack6

Comparisons of adjusted shrinkage strain measurements to the shrinkage relationships specified by AASHTO LRFD Bridge Design Specification (2004) are shown in Figure 5-48 to Figure 5-59. The shrinkage strain specified by ASSHTO LRFD Bridge Design Specifications (2004) can be found in Section 2.4.2.3.



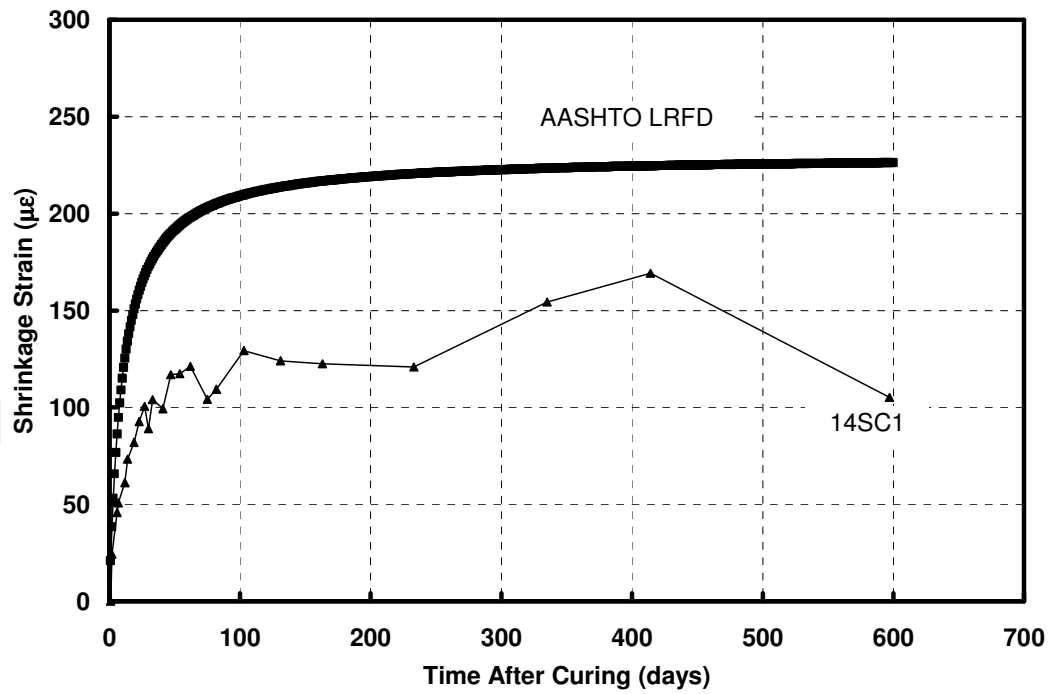
(10 ksi Target Concrete Compressive Strength, Heat Cured Cylinder)

Figure 5-48 – Comparison of Shrinkage Strain of 10SC1



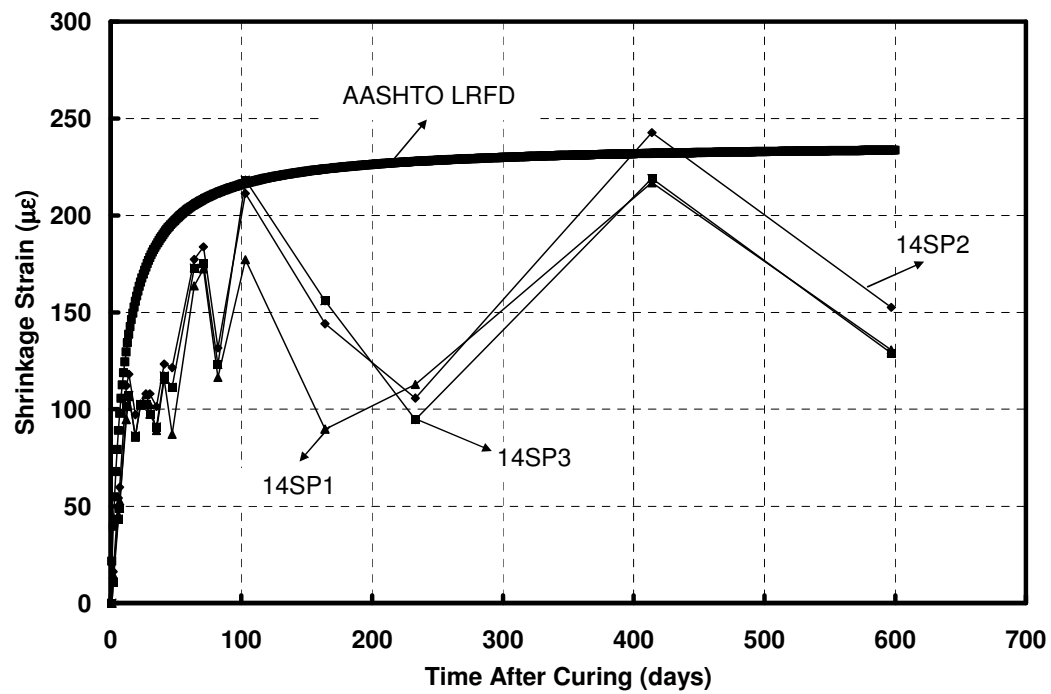
(10 ksi Target Concrete Compressive Strength, Heat Cured Prisms)

Figure 5-49 – Comparison of Shrinkage Strain of 10SP1, 10SP2 and 10SP3



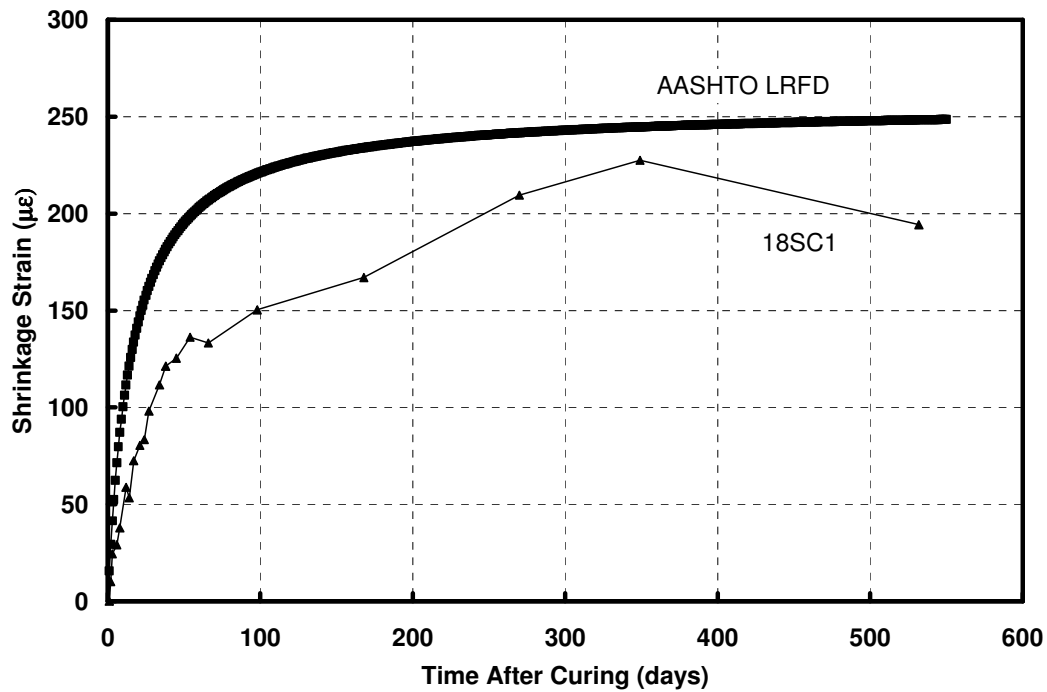
(14 ksi Target Concrete Compressive Strength, Heat Cured Cylinder)

Figure 5-50 – Comparison of Shrinkage Strain of 14SC1



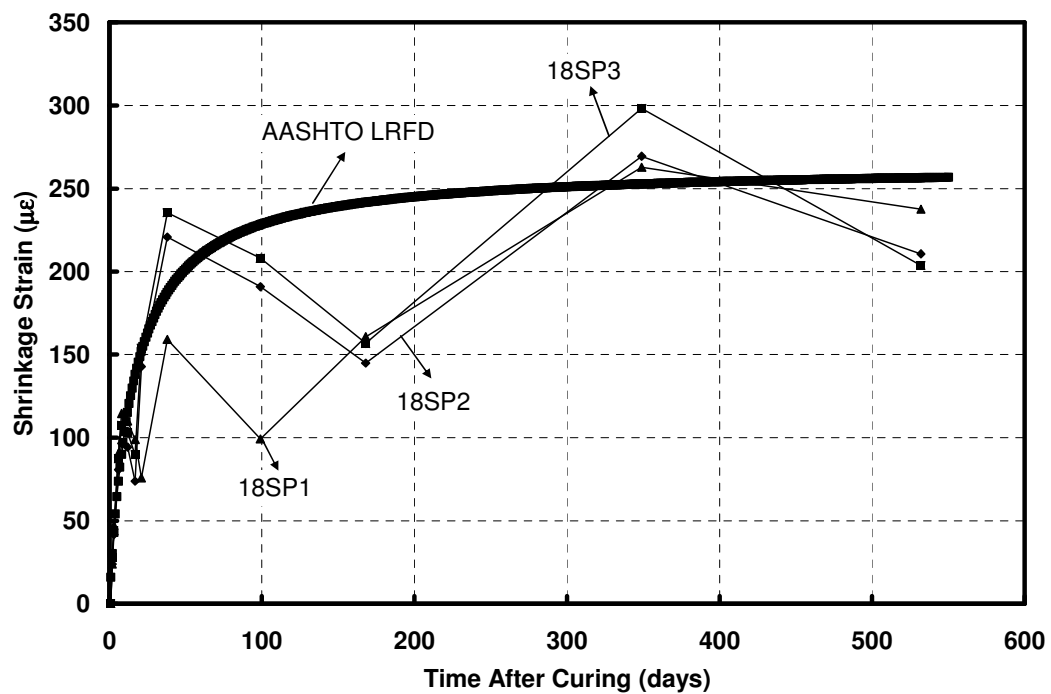
(14 ksi Target Concrete Compressive Strength, Heat Cured Prisms)

Figure 5-51 – Comparison of Shrinkage Strain of 14SP1, 14SP2 and 14SP3



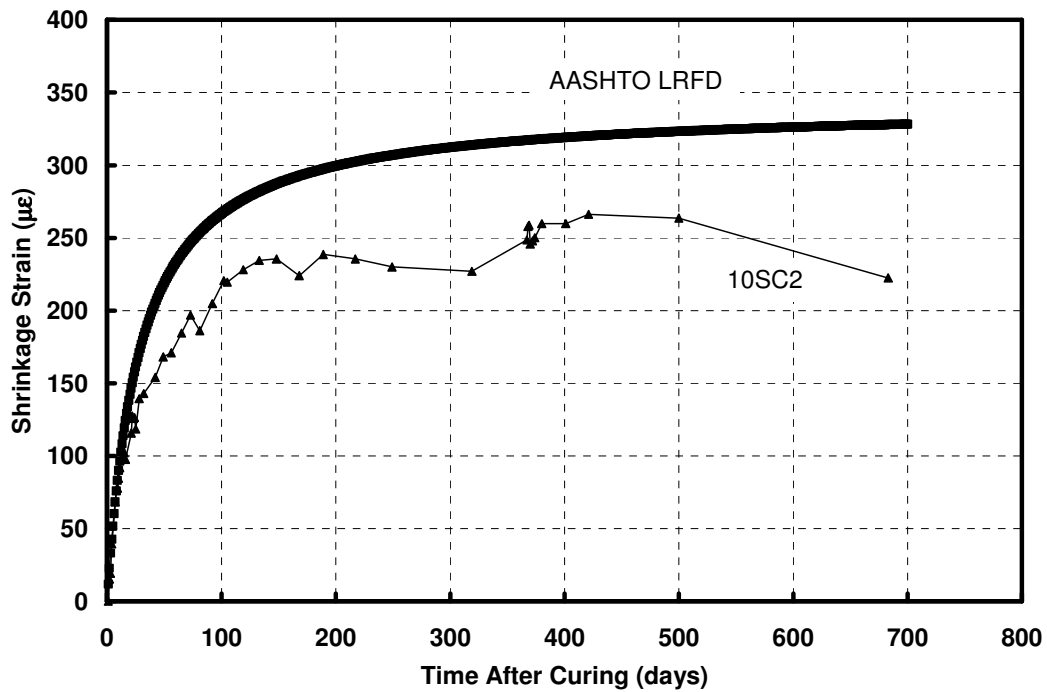
(18 ksi Target Concrete Compressive Strength, Heat Cured Cylinder)

Figure 5-52 – Comparison of Shrinkage Strain of 18SC1



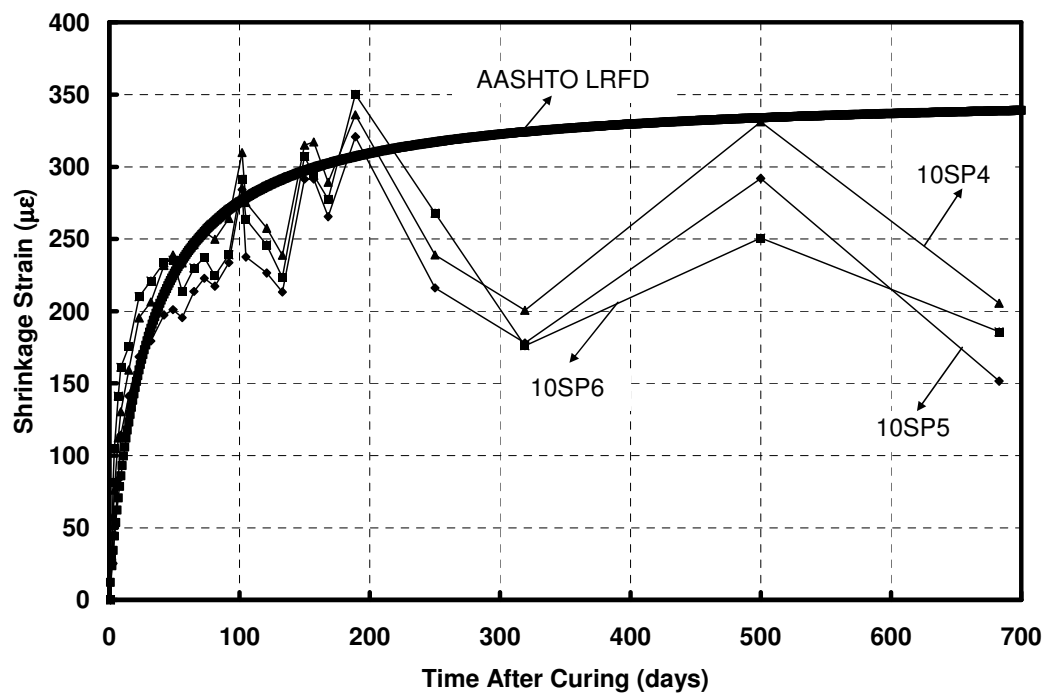
(18 ksi Target Concrete Compressive Strength, Heat Cured Prisms)

Figure 5-53 – Comparison of Shrinkage Strain of 18SP1, 18SP2 and 18SP3



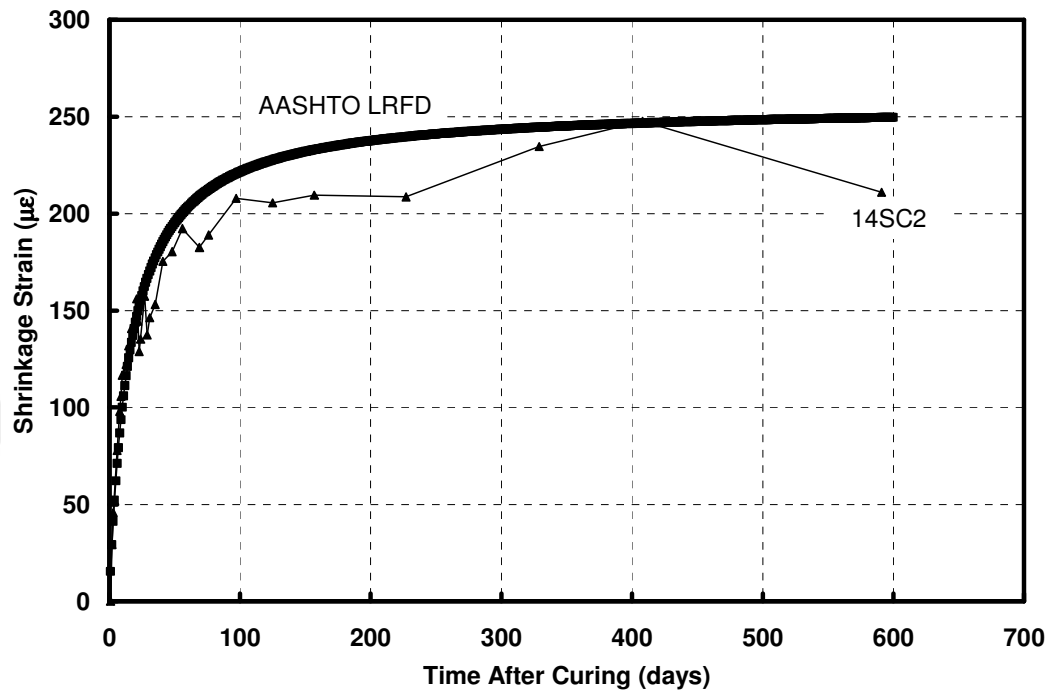
(10 ksi Target Concrete Compressive Strength, Moist Cured Cylinder)

Figure 5-54 – Comparison of Shrinkage Strain of 10SC2



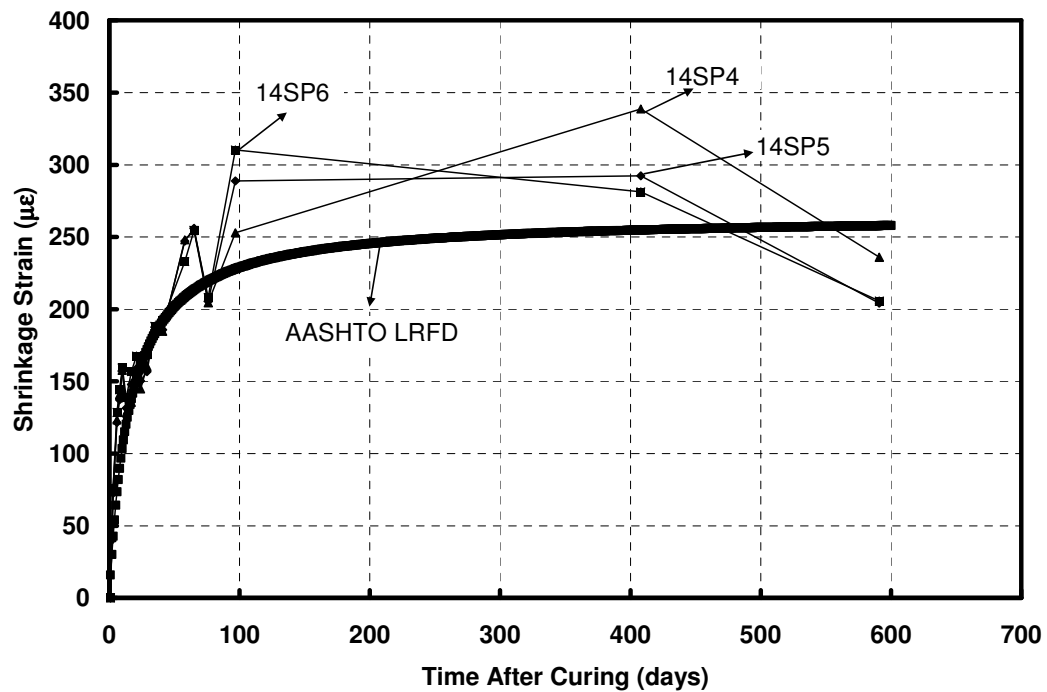
(10 ksi Target Concrete Compressive Strength, Moist Cured Prisms)

Figure 5-55 – Comparison of Shrinkage Strain of 10SP4, 10SP5 and 10SP6



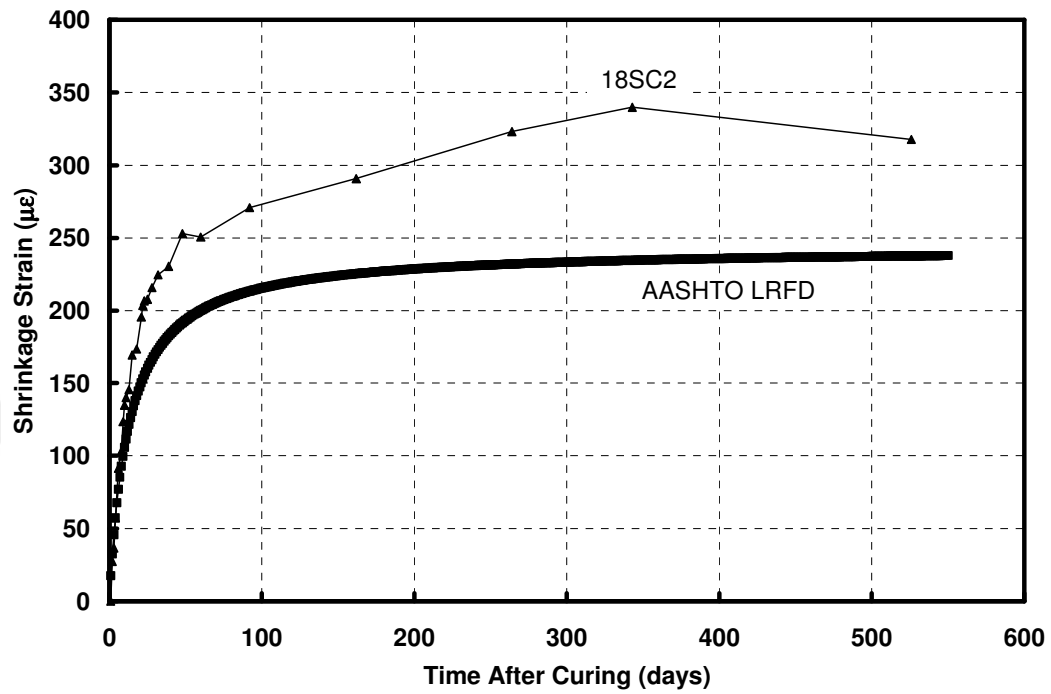
(14 ksi Target Concrete Compressive Strength, Moist Cured Cylinder)

Figure 5-56 – Comparison of Shrinkage Strain of 14SC2



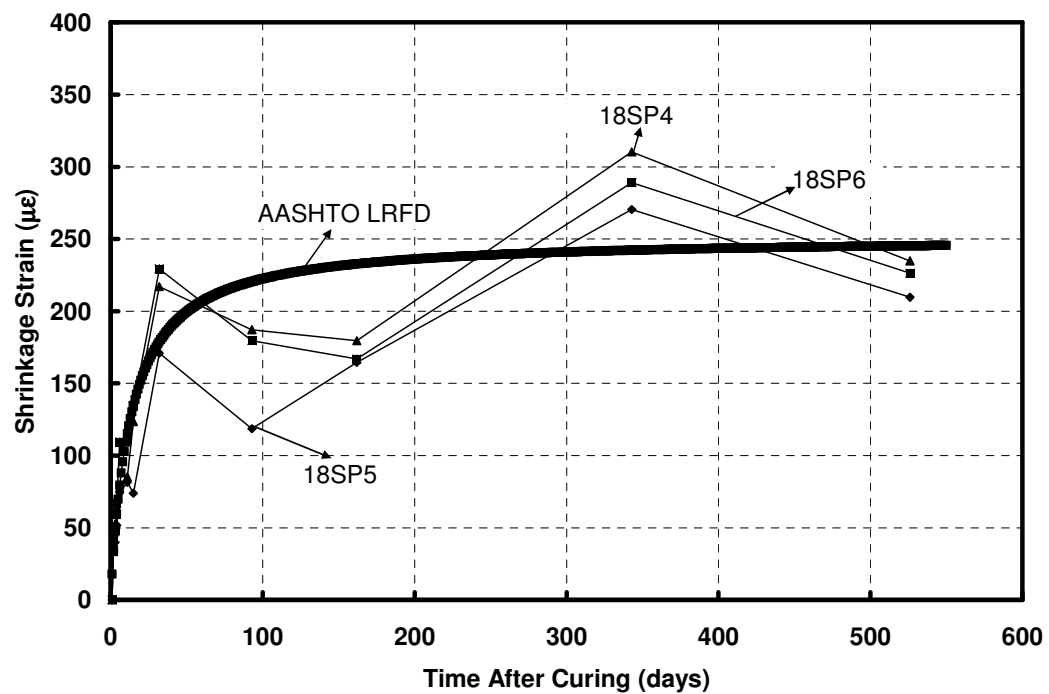
(14 ksi Target Concrete Compressive Strength, Moist Cured Prisms)

Figure 5-57 – Comparison of Shrinkage Strain of 14SP4, 14SP5 and 14SP6



(18 ksi Target Concrete Compressive Strength, Moist Cured Cylinder)

Figure 5-58 – Comparison of Shrinkage Strain of 18SC2



(18 ksi Target Concrete Compressive Strength, Moist Cured Prisms)

Figure 5-59 – Comparison of Shrinkage Strain of 18SP4, 18SP5 and 18SP6

The test results of this research indicate that the creep and shrinkage prediction relationships specified by AASHTO LRFD Bridge Design Specifications (2004) are approximately predicting the creep and shrinkage behavior of HSC. However, the creep and shrinkage prediction relationships contain the following development correction factor which must be investigated in details since it yields negative results in the first few days after loading for concrete strength greater than 15.0 ksi:

$$k_{td} = \frac{t}{61 - 4f'_{ci} + t}$$

where t is the age of concrete after loading in days, f'_{ci} is the specified compressive strength in ksi at prestress transfer for prestressed members or 80 percent of the strength at service for non-prestressed members.

The equation also gives abrupt changes in the slope of the predicted creep in the first few days for concrete strengths more than 12 ksi. This equation was developed by Tadros et al (2003) based on research data with concrete strengths up to 12 ksi and was extended to include strengths up to 15 ksi. It was recognized that concrete compressive strength of 18 ksi would not be used for releasing strength for pretensioned members. However, for cast in place columns and post-tensioning girders, the concrete strength at the time of loading could be as high as 18 ksi. The results of this research indicate that the equation should be revised to provide better predictions for concrete strengths up to 18 ksi.

After a detailed examination of test results and parametric studies, the following time-development correction factor is proposed:

$$k_{td} = \frac{t}{12 \left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20} \right) + t}$$

In Figure Figure 5-60 to Figure 5-66, the proposed time development correction factor and the current expression are compared for different concrete compressive strengths up

to 18 ksi. In these figures, the dotted and the solid lines represent the current and the proposed time development correction factors, respectively. It can be seen that for concrete strength greater than 12 ksi, the proposed time development correction factor eliminates the unreasonable prediction values given by the current time development correction factor.

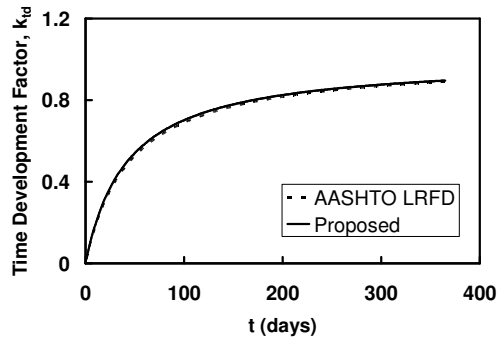


Figure 5-60 – k_{td} for $f'_{ci} = 4$ ksi

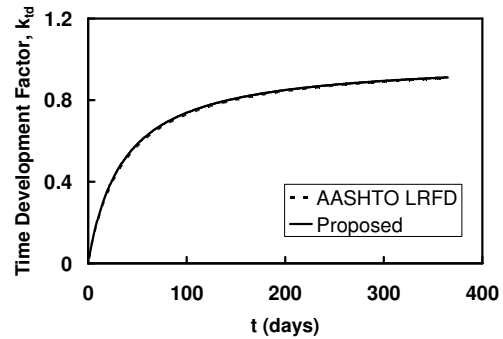


Figure 5-61 – k_{td} for $f'_{ci} = 6$ ksi

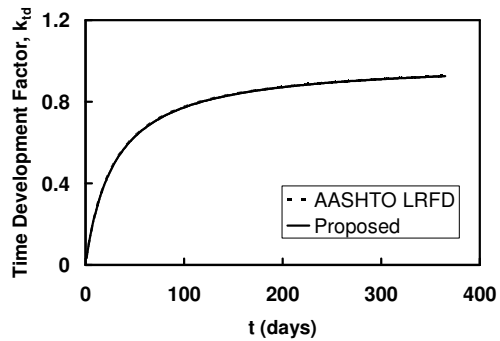


Figure 5-62 – k_{td} for $f'_{ci} = 8$ ksi

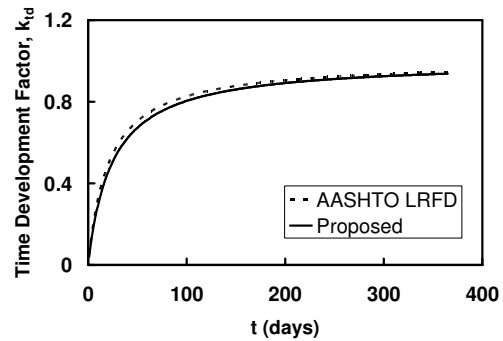


Figure 5-63 – k_{td} for $f'_{ci} = 10$ ksi

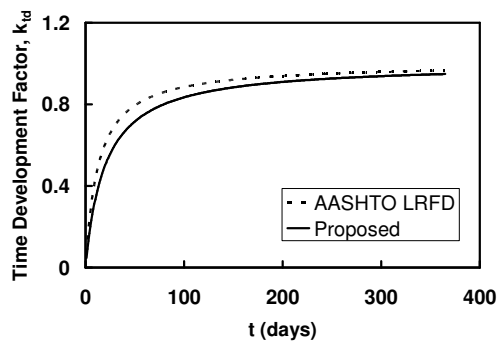


Figure 5-64 – k_{td} for $f'_{ci} = 12$ ksi

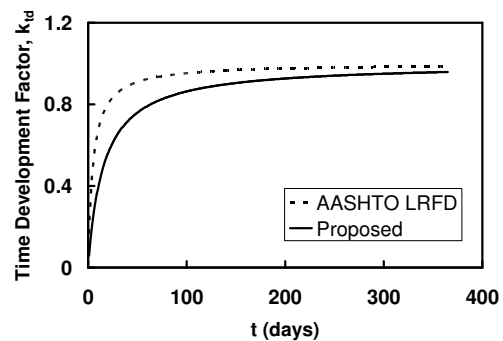


Figure 5-65 – k_{td} for $f'_{ci} = 14$ ksi

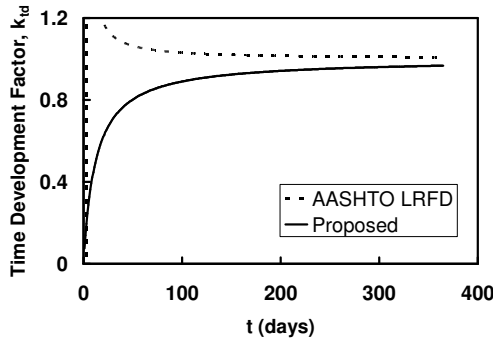


Figure 5-66 – k_{td} for $f'_{ci} = 16$ ksi

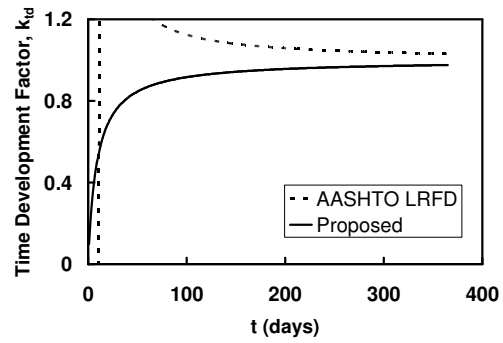


Figure 5-67 – k_{td} for $f'_{ci} = 18$ ksi

5.5 Proposed Limits for Reinforcement for Compression Members

AASHTO LRFD Bridge Design Specifications (2004) have two relationships to limit the maximum reinforcement and one criterion to limit the minimum reinforcement for compression members. The maximum area of prestressed and non-prestressed longitudinal reinforcement for non-composite compression components according to AASHTO LRFD Bridge Design Specifications (2004) is limited by the two following equations:

$$\frac{A_s}{A_g} + \frac{A_{ps}f_{pu}}{A_gf_y} \leq 0.08 \quad (\text{AASHTO LRFD Eqn. 5.7.4.2-1}) \quad \text{Equation 5-11}$$

and

$$\frac{A_{ps}f_{pe}}{A_gf'_c} \leq 0.30 \quad (\text{AASHTO LRFD Eqn. 5.7.4.2-2}) \quad \text{Equation 5-12}$$

where f'_c is the concrete compressive strength, A_{ps} is the area of prestressing steel, f_{pu} is the specified tensile strength of prestressing steel, f_{pe} is the effective prestress after losses, A_s and f_y are the area and yield strength of mild compression steel, respectively, and A_g is the gross area of the section.

The minimum area of prestressed and non-prestressed longitudinal reinforcement for non-composite compression components according to AASHTO LRFD Bridge Design Specifications (2004) is:

$$\frac{A_s f_y}{A_g f'_c} + \frac{A_{ps} f_{pu}}{A_g f'_c} \geq 0.135 \quad (\text{AASHTO LRFD Eqn. 5.7.4.2-3}) \quad \text{Equation 5-13}$$

The upper limits were initially established based on practical considerations of concrete placement and have since been maintained for all ranges of concrete compressive strengths. Therefore no changes are proposed to AASHTO LRFD Bridge Design Specifications (2004) for the maximum reinforcement ratio for compression members to extend the applicability of the specifications up to 18 ksi.

However, the analysis conducted to examine current AASHTO LRFD Bridge Design Specifications (2004) indicate a requirement of 4.05 percent as the minimum reinforcement ratio for 18 ksi concrete compressive strength and Grade 60 steel, in the absence of any prestressing steel in the section as shown in Figure 5-68. Such very high level of minimum reinforcement ratio is quite unusual and should be examined for HSC. In order to evaluate this reinforcement limit, it is necessary to review the basis and historical development of the current requirement of the minimum reinforcement.

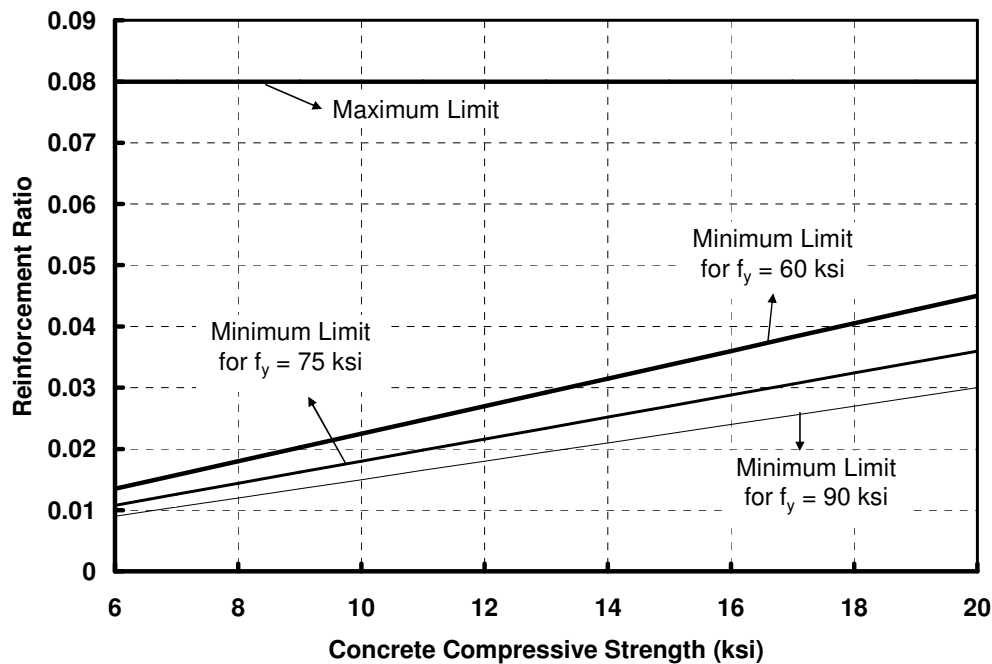


Figure 5-68 – Reinforcement Limits for Compression Members with Only Mild Steel
According to the Current AASHTO LRFD Bridge Design Specifications

For non-prestressed sections, the minimum limit for longitudinal reinforcement for compression members originated from the early column tests by Richart et al. (1931a, 1931b, 1931c and 1932) at the University of Illinois in the 1930's. When a column is tested under sustained service loads, the stress distribution between steel and concrete changes over time due to creep and shrinkage of concrete. With creep and shrinkage increasing progressively, concrete relieves itself from its initial share of the axial load. As a result, longitudinal steel reinforcement gradually carries a larger portion of the sustained load over time. Therefore, it is theoretically possible that in columns with small amounts of longitudinal reinforcement, the reinforcing steel could yield, resulting in creep rupture of the column. Tests by Richart et al. (1931a, 1931b, 1931c and 1932) showed the increase of stress in the steel reinforcement is inversely proportional to the percentage of the longitudinal steel. Results from their tests carried out with concrete strengths between 2 and 8 ksi, suggested a minimum reinforcement ratio of 1%. The application of this limit was later extended by AASHTO LRFD Bridge Design Specifications (2004) for concrete compressive strengths up to 10 ksi without any further tests or analysis, and certainly without any consideration for HSC above 10 ksi.

Three types of strain are normally developed in the longitudinal reinforcement under the effect of sustained loading: initial elastic strain, strain developed due to shrinkage of concrete and strain developed due to creep of concrete.

When a sustained load is applied to a reinforced concrete column, initial elastic strain, ϵ_1 , is observed immediately as shown in Figure 5-69.

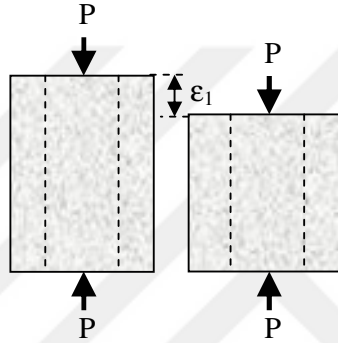


Figure 5-69 – Initial Elastic Strain due to Applied Sustained Load

At this stage the applied load is resisted by both concrete and steel as follows:

$$P = E_c \epsilon_1 (1 - \rho_l) A_g + E_s \epsilon_1 \rho_l A_g \quad \text{Equation 5-14}$$

where P is the applied axial load, E_c is the modulus of elasticity of concrete, ρ_l is the longitudinal reinforcement ratio, A_g is the gross area of concrete and E_s is the modulus of elasticity of steel. The initial elastic strain of concrete and steel can be obtained from the above equilibrium equation, i.e.,

$$\epsilon_1 = \frac{P}{A_g} \frac{1}{(E_c (1 - \rho_l) + E_s \rho_l)} \quad \text{Equation 5-15}$$

Following the initial elastic deformation, the time dependent deformations will occur in the concrete due to creep and shrinkage. Since the column contains reinforcement, the shrinkage strain, ϵ_{sh} , will be restrained by the longitudinal reinforcement of the column causing an increase in the load carried by the reinforcement and a decrease in the load

carried by the concrete. The same behavior holds for the creep strain of concrete, ϵ_{cr} . The behavior of reinforced concrete column due to shrinkage and creep is presented in Figure 5-70.

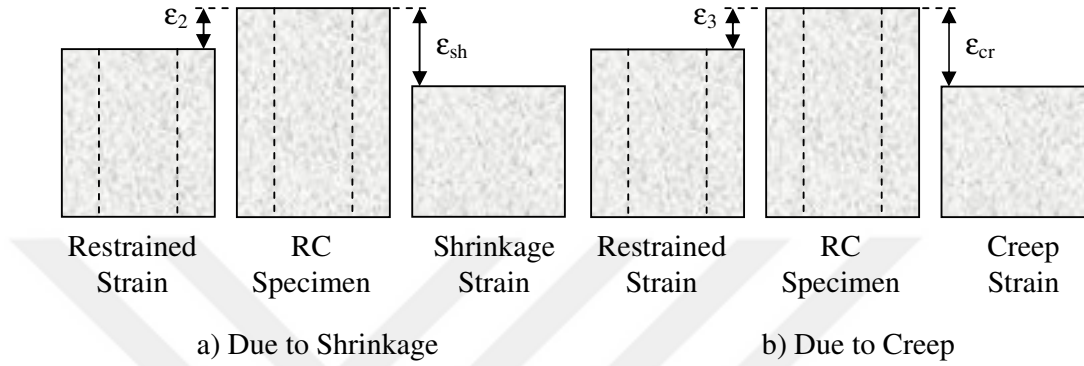


Figure 5-70 – Behavior of Reinforced Concrete Column due to Shrinkage and Creep

From equilibrium of forces due to shrinkage,

$$E_s \epsilon_2 \rho_l A_g = E_c (\epsilon_{sh} - \epsilon_2) (1 - \rho_l) A_g \quad \text{Equation 5-16}$$

where ϵ_2 is the strain in the reinforcement due to shrinkage of concrete. Thus the strain developed in the longitudinal reinforcement due to shrinkage of concrete can be determined as:

$$\epsilon_2 = \frac{(1 - \rho_l) \epsilon_{sh} E_c}{(\rho_l E_s + (1 - \rho_l) E_c)} \quad \text{Equation 5-17}$$

Similarly, from equilibrium of forces due to creep,

$$E_s \epsilon_3 \rho_l A_g = E_c (\epsilon_{cr} - \epsilon_3) (1 - \rho_l) A_g \quad \text{Equation 5-18}$$

where ϵ_3 is the strain in the reinforcement due to creep of concrete, ϵ_{cr} is the creep strain of unrestrained concrete. Then the strain developed in the longitudinal reinforcement due to creep of concrete can be determined as:

$$\varepsilon_3 = \frac{(1 - \rho_l)\varepsilon_{cr}E_c}{(\rho_l E_s + (1 - \rho_l)E_c)} \quad \text{Equation 5-19}$$

To prevent yielding of the longitudinal reinforcement, the summation of the initial elastic strain and the strains due to shrinkage and creep should not reach the yield strain of the longitudinal reinforcement. Thus,

$$\varepsilon_{total} = \varepsilon_1 + \varepsilon_2 + \varepsilon_3 \leq \text{yield strain of longitudinal reinforcement} \quad \text{Equation 5-20}$$

Note that for Grade 60 steel reinforcement, the yield strain is assumed to be 0.002.

The procedure used to calculate the minimum longitudinal reinforcement ratio for compression members was an iterative procedure which was modeled using Microsoft Excel. The amount of reinforcement was determined for a reinforced concrete column under sustained load which would lead to a total strain of 0.002 after specified period of time. The procedure used was as follows:

1. The range of concrete compressive strength used in this study varies between 6 ksi and 18 ksi.
2. The modulus of elasticity of steel was used as 30,000 ksi. For modulus of elasticity of concrete, the relationships proposed by NCHRP Project 12-64 to extend the applicability of AASHTO LRFD Bridge Design Specifications (2004) as well as the current AASHTO LRFD Bridge Design Specifications (2004) were used for HSC. However, most critical conditions were established using the one proposed by NCHRP 12-64. The unit weight of concrete (w_c) used in the analysis was 0.150 kcf since HSC is more compact and denser than NSC. The modulus of elasticity (E_c) equation proposed by NCHRP 12-64 used in the analysis is:

$$E_c = 310,000 K_1 w_c^{2.5} (f_c')^{0.33} \quad \text{Equation 5-21}$$

where K_I is the correction factor for source of aggregate (taken as 1.0) and f'_c is the concrete compressive strength.

The analysis was repeated using the current equation specified by AASHTO LRFD Bridge Design Specifications (2004):

$$E_c = 33,000 K_1 w_c^{1.5} \sqrt{f'_c} \quad (\text{AASHTO LRFD Eqn. 5.4.2.4-1}) \quad \text{Equation 5-22}$$

3. The shrinkage strain relationship (ϵ_{sh}) specified by AASHTO LRFD Bridge Design Specifications (2004) was used to calculate the shrinkage strain of concrete (Section 2.4.2.3).
4. The creep coefficient relationship (ψ) specified by AASHTO LRFD Bridge Design Specifications (2004) was used to calculate the creep of concrete (Section 2.4.2.3).
5. The relative humidity used in the calculation of the creep and shrinkage was 10 percent, since lower relative humidity would produce more critical results.
6. The volume to surface ratio used in the calculation of creep and shrinkage was 3. Note that, the volume to surface ratio for a circular column with 12 in. diameter is 3. Also for a 12×12 in. square column, the volume to surface ratio is 3.
7. The time considered in the calculation of the creep and shrinkage was 10 years which is equal to 3650 days.
8. The age of loading in the calculation of the creep coefficient was 28 days.
9. The sustained load level on the reinforced concrete column considered in this investigation was 50 percent. ($P/f'_c A_g = 0.5$). The unfactored permanent load on columns do not exceed $0.5A_g f'_c$, which is typically the case encountered in design.

10. The effects associated with stress relief for both creep and shrinkage due to creep of concrete in tension are neglected in the formulation of the equilibrium conditions. By neglecting such effects, the results are more conservative.
11. The initial value for the longitudinal reinforcement ratio (ρ_l) for a reinforced concrete column was established. The initial elastic strain and strains due to creep and shrinkage were calculated based on the previous discussions in this section. The sum of all three strain values, the total strain (ϵ_{total}), was calculated and compared to the yield strain of steel reinforcement. By changing the initial value of the longitudinal reinforcement ratio, the reinforcement ratio for which the total strain was equal to the yield strain of steel was determined. This reinforcement ratio was used as the minimum amount of longitudinal reinforcement ratio for compression members to prevent creep rupture.
12. Step 11 was performed for all the concrete compressive strengths in the considered range.

Note that, the most critical conditions were evaluated in the calculation of minimum longitudinal reinforcement ratio for compression members. Based on the performed analysis using the proposed equation and the current relationship specified by AASHTO LRFD Bridge Design Specifications (2004) for E_c , a new relationship is proposed for minimum reinforcement ratio for compression members as follows:

$$\frac{A_s}{A_g} + \frac{A_{ps}f_{pu}}{A_gf_y} \geq 0.135 \frac{f'_c}{f_y} \text{ but not greater than } 0.0225. \quad \text{Equation 5-23}$$

For concrete compressive strengths up to 10 ksi, the proposed relationship for minimum longitudinal reinforcement ratio requires the same amount as that of AASHTO LRFD Bridge Design Specifications (2004). For concrete compressive strengths greater than 10 ksi, the proposed equation requires the same amount of 0.0225 for concrete compressive strengths up to 18 ksi. Furthermore, the proposed minimum reinforcement limitation is similar in format with the maximum reinforcement limitation specified by AASHTO LRFD Bridge Design Specifications (2004).

The minimum longitudinal reinforcement ratio required by the current AASHTO LRFD Bridge Design Specifications (2004), the proposed provision, and based on the above procedure considering the effects of creep and shrinkage, for the stress level $P/f'_c A_g = 0.5$ are tabulated in Table 5-7 and shown in Figure 5-71. The figure clearly indicates that for concrete strength greater than 10 ksi, the required minimum longitudinal reinforcement ratio by the proposed equation is greatly reduced from that called for by the current AASHTO LRFD Bridge Design Specifications (2004), but the proposed equation still provides substantial margin against what is needed to prevent creep rupture.

Table 5-7 – Comparison of the A_s/A_g Ratio for $P/f'_c A_g = 0.5$

f'_c (ksi)	$P/f'_c A_g = 0.5$		
	A_s/A_g (AASHTO LRFD)	A_s/A_g (Proposed)	A_s/A_g (Calculated)
6	0.0135	0.0135	1.109
7	0.01575	0.01575	0.892
8	0.018	0.018	0.764
9	0.02025	0.02025	0.707
10	0.0225	0.0225	0.712
11	0.02475	0.0225	0.770
12	0.027	0.0225	0.874
13	0.02925	0.0225	1.020
14	0.0315	0.0225	1.204
15	0.03375	0.0225	1.422
16	0.036	0.0225	1.672
17	0.03825	0.0225	1.952
18	0.0405	0.0225	2.259

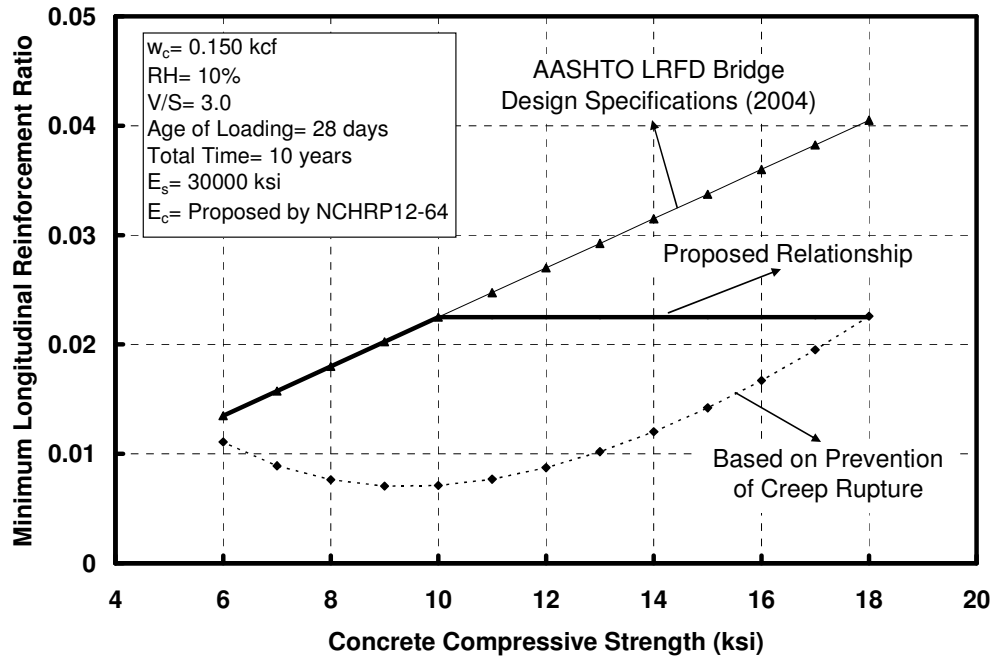


Figure 5-71 – Comparison of the A_s/A_g Ratio for $P/f'_c A_g = 0.5$

The calculated values for minimum reinforcement ratio for compression members based on prevention of creep rupture for $P/f'_c A_g = 0.5$ are tabulated in Table 5-8. Note that the summation of the initial elastic, shrinkage and creep strains are equal to the yield strain of Grade 60 steel reinforcement, 0.002. It is clear from the table that the creep and shrinkage strains of concrete decreases as concrete compressive strength increases. However, the initial elastic strain increases as concrete compressive strength increases since the same stress level was applied on each column with different concrete compressive strengths. When columns with 6 ksi and 18 ksi concrete compressive strengths are compared under $P/f'_c A_g = 0.5$, the load applied on the column with 18 ksi concrete compressive strength is 3 times that of applied on the column with 6 ksi concrete compressive strength. However, the modulus of elasticity of the column with 18 ksi concrete compressive strength is only 1.44 times that of the column with 6 ksi concrete compressive strength. Therefore, the minimum reinforcement ratio for compression members can not be reduced for HSC compared to NSC, although HSC creeps and shrinks less.

Table 5-8 – Calculated Values for $P/f'_c A_g = 0.5$

f'_c (ksi)	ρ_l (%)	E_c (ksi)	Initial Elastic Strain (ϵ_l)	Shrinkage Strain (ϵ_s)	Creep Strain (ϵ_3)
6	1.109	4880	0.000582	0.000633	0.000785
7	0.892	5134	0.000653	0.000563	0.000784
8	0.764	5365	0.000720	0.000505	0.000775
9	0.707	5578	0.000782	0.000457	0.000761
10	0.712	5776	0.000841	0.000415	0.000744
11	0.770	5960	0.000895	0.000380	0.000725
12	0.874	6134	0.000946	0.000350	0.000704
13	1.020	6298	0.000994	0.000323	0.000683
14	1.204	6454	0.001039	0.000299	0.000662
15	1.422	6602	0.001081	0.000278	0.000641
16	1.672	6744	0.001122	0.000259	0.000619
17	1.952	6881	0.001159	0.000242	0.000599
18	2.259	7012	0.001195	0.000227	0.000578

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

The characteristics of HSC for combined flexural and axial compression were investigated in this research. The experimental program consisted of two parts. The first part included tests of twenty-one 9×9×40 unreinforced HSC members under combined axial load and flexure to evaluate the stress-strain distribution in the compression zone of flexural members. The test concept was based on the test method developed by Hognestad et al. (1955) to simulate the compression zone of a flexural member. The two axial loads were adjusted during the test to maintain zero strain (which is at the neutral axis for a flexural member) at one face of the specimen and the maximum compressive strain at the opposite face of the cross-section. The main parameter was the concrete compressive strength ranging from 10.4 to 16 ksi. Stress-strain relationships, ultimate compressive strain, stress block parameters and Poisson's ratios for HSC were obtained, evaluated and compiled with the data available in the literature.

The second part of the experimental program consisted of tests performed to evaluate the creep and shrinkage behavior of HSC. A total of forty-two 4×12 in. cylindrical specimens and eighteen 3×3×11¼ in. prism specimens were monitored for one year. The variables considered in this investigation were the concrete compressive strength ranging from 10.4 to 16.7 ksi, specimen size (cylindrical or prism), curing type (moist or heat curing), age of concrete at loading (1, 7, 14, 28 days) and loading stress level ($0.2f'_c$ or $0.4f'_c$). The creep strains, specific creep values, creep coefficients and shrinkage strains were obtained for the range of concrete compressive strength, evaluated and compiled with the available relationships in the literature.

Test results of this research combined with other researches in the literature were analytically evaluated using regression analyses and parametric studies. These analyses were used to evaluate the validity of the provisions of AASHTO LRFD Bridge Design Specifications (2004) for HSC. If necessary, new relationships were proposed to modify AASHTO LRFD Bridge Design Specifications (2004) to extend the current limitation of 10 ksi concrete compressive strength up to 18 ksi.

6.2 Observations and Conclusions

The observations made and conclusions drawn from this study are based on the test results of this research and other researches in the literature unless otherwise stated. Therefore, the proposed equations can appropriately be used for design purposes for concrete compressive strengths up to 18 ksi.

1. Similar behaviors were observed for all the eccentric bracket specimens regardless of concrete compressive strength. The eccentric bracket specimens showed an explosive failure of the compression face. It was observed that the failure surface was passing through the aggregates indicating the cement paste and the interface between the cement paste and aggregate were stronger than the aggregate itself.
2. The assumption of plane sections remain plane after deformation is valid for high strength concrete.
3. The stress-strain distributions obtained from the eccentric bracket specimens tested in this research indicated that as concrete compressive strength increased, the strain at the peak stress increased and was closer to 0.003 strain. The shape of the ascending branch of the stress-strain relationship became more linear and steeper, and the slope of the descending part also became steeper. The general shape of the stress-strain relationship became more likely to be a triangle with little or no descending parts.
4. The ultimate concrete compressive strain of 0.003 is proposed to be used for design purposes for concrete compressive strengths up to 18 ksi.
5. Poisson's ratio of 0.2 was found to be an appropriate value for concrete compressive strengths up to 18 ksi.
6. The test results of this research and other researches in the literature indicate that the majority of the collected data for the generalized stress block parameter k_1 for HSC is higher than 0.58 for concrete compressive strengths ranging between 10 and 18 ksi. Therefore, using the lower bound of 0.58 is proposed for k_1 parameter for concrete compressive strengths beyond 15 ksi.

7. The test results of this research and other researches in the literature indicate that the stress block parameter k_2 for HSC between 8 ksi and 18 ksi can be assumed to be 0.33.
8. The test results of this research and other researches in the literature indicate that the generalized stress block parameter k_3 for HSC is similar to NSC. Hence, using the same value of k_3 parameter, 0.85, for concrete compressive strengths up to 18 ksi is found to be appropriate for design purposes.
9. The test results, confirmed by other data in the literature, indicate that the stress block parameter α_l should be reduced when the compressive strength of concrete is increased beyond 10 ksi. The recommended value for the parameter α_l for concrete compressive strengths up to 18 ksi is:

$$\alpha_l = \begin{cases} 0.85 & \text{for } f'_c \leq 10 \text{ ksi} \\ 0.85 - 0.02(f'_c - 10) \geq 0.75 & \text{for } f'_c > 10 \text{ ksi} \end{cases} \quad \text{Equation 6-1}$$

where f'_c is in ksi.

10. The following relationship currently used by AASHTO LRFD Bridge Design Specifications is found to be acceptable for the rectangular stress block parameter β_l for concrete compressive strengths up to 18 ksi:

$$\beta_l = \begin{cases} 0.85 & \text{for } f'_c \leq 4 \text{ ksi} \\ 0.85 - 0.05(f'_c - 4) \geq 0.65 & \text{for } f'_c > 4 \text{ ksi} \end{cases} \quad \text{Equation 6-2}$$

where f'_c is in ksi.

11. The results of the sensitivity analysis for flexural under-reinforced concrete sections indicate that the change in the rectangular stress block parameters and the ultimate compressive strain of concrete have very little or no effect on the ultimate moment capacity of the section. On the other hand the results of the sensitivity analysis for the

6 Conclusions and Recommendations

flexural over-reinforced concrete sections indicate that the change in the rectangular stress block parameters affects the ultimate moment capacity of the sections significantly. However, the change in the ultimate compressive strain of concrete has very little effect on the ultimate moment capacity of the section for flexural over-reinforced sections.

12. Based on the test results of this research, the following stress-strain relationship was found to be the most appropriate equation to predict the stress-strain distribution of concrete for compressive strengths up to 18 ksi.

$$f_c = f'_c \frac{\epsilon_c}{\epsilon_{co}} \frac{n}{n-1 + (\epsilon_c/\epsilon_{co})^{nk}} \quad \text{Equation 6-3}$$

with

$$n = 0.31f'_c + 0.78 \quad \text{Equation 6-4}$$

$$k = 0.10f'_c + 1.20 \quad \text{Equation 6-5}$$

and

$$\epsilon_{co} = 0.0033 - 0.00002f'_c \quad \text{Equation 6-6}$$

$$\epsilon_{cu} = 0.0038 - 0.00004f'_c \quad \text{Equation 6-7}$$

where f'_c is in ksi.

13. In general, test results of this research and other researches in the literature indicate that as concrete ages, the creep of HSC decreases. The creep behavior of heat cured cylinders is less than that of the moist cured cylinders. Creep for HSC is proportional to the applied stress provided that the applied stress was less than the proportional limit.

6 Conclusions and Recommendations

14. Test results of this research on shrinkage of HSC indicate that heat cured specimens had less shrinkage compared to moist cured specimens. There was little difference in shrinkage of HSC specimens with target concrete strengths of 10, 14 and 18 ksi. The average weight loss of the specimens was directly proportional to shrinkage of HSC.
15. The relationships specified by the current AASHTO LRFD Bridge Design Specifications (2004) were found to be acceptable to predict creep and shrinkage of HSC except the time-development correction factor. The time-development correction factor which yields negative results in the first few days after loading for concrete strengths greater than 15.0 ksi. Accordingly, the following time-development correction factor was proposed to eliminate the unreasonable prediction values given by the current time development correction factor:

$$k_{td} = \frac{t}{12 \left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20} \right) + t} \quad \text{Equation 6-8}$$

where f'_{ci} is in ksi.

16. The relationships for the maximum reinforcement ratio for compression members specified by AASHTO LRFD Bridge Design Specifications (2004) are applicable for concrete compressive strengths up to 18 ksi.
17. The current AASHTO LRFD Bridge Design Specifications (2004) requires extremely high levels of minimum reinforcement ratio for compression members for concrete compressive strengths over 10 ksi. Based on an analysis of creep buckling behavior using the proposed equation and the current relationship specified by AASHTO LRFD Bridge Design Specifications (2004) for E_c , a new relationship is developed for minimum reinforcement ratio for compression members as follows:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \geq 0.135 \frac{f'_c}{f_y} \text{ but not greater than } 0.0225. \quad \text{Equation 6-9}$$

For concrete compressive strengths up to 10 ksi, the proposed relationship for minimum longitudinal reinforcement ratio requires the same amount as that of AASHTO LRFD Bridge Design Specifications (2004). For concrete compressive strengths greater than 10 ksi, the proposed equation requires the same amount of 0.0225 for concrete compressive strengths up to 18 ksi.

6.3 Recommendations

Due to the practical production reasons, the number of tests performed to evaluate the stress-strain distribution and shrinkage and creep behavior of HSC was limited for concrete compressive strength over 16 ksi. More tests are needed for concrete compressive strengths beyond 16 ksi to provide additional data to fully validate the proposed relationships.

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APPENDICES



APPENDIX A – ECCENTRIC BRACKET TEST DATA BY DIFFERENT RESEARCHERS

Hognestad (1951)					
Specimen ID	f'_c (ksi)	ϵ_{cu}	Specimen ID	f'_c (ksi)	ϵ_{cu}
A1a*	5.28	0.0034	C7a*	1.97	0.00372
A1b*	5.66	0.0038	C7b*	1.52	0.005
B1a*	4.25	0.0049	A8a*	5.52	0.0044
B1b*	4.07	0.00456	A8b*	5.81	0.00402
C1a*	2.27	0.0036	B8a*	4.7	0.00386
C1b*	2.02	0.00344	B8b*	4.26	0.0034
A2a*	5.28	0.00334	C8a*	1.82	0.00464
A2b*	5.83	0.00326	C8b*	1.82	0.00556
B2a*	4.25	0.00456	B9b*	4.37	0.00336
B2b*	4.07	0.0046	C9a*	1.88	0.00432
C2a*	2.27	0.003	C9b*	1.73	0.00552
C2b*	1.97	0.0036	B12a*	4.3	0.00348
A3a*	5.66	0.003	B12b*	4.01	0.00336
A3b*	5.83	0.00382	C12a*	2.3	0.00382
B3a*	4.63	0.00316	C12b*	2.2	0.00448
B3b*	4.29	0.0032	B13a*	3.58	0.00404
C3a*	1.88	0.0038	B13b*	4.29	0.00412
C3b*	1.69	0.0036	C13a*	2.3	0.00436
C4a*	1.69	0.00312	C13b*	2.07	0.0038
C4b*	1.73	0.00368	A14b*	5.1	0.0037
C5a*	2.31	0.0044	B14a*	3.58	0.0033
C5b*	1.77	0.00368	C14a*	1.95	0.0043
A7a*	5.24	0.00412	C14b*	2.07	0.00392
A7b*	5.81	0.0044	C15a*	1.95	0.0034
B7a*	4.08	0.00376	C15b*	2.07	0.00428
B7b*	4.04	0.00324			

* column specimens subjected to eccentrically applied axial load

Hognestad et al. (1955)*					
Specimen ID	f'_c (ksi)	$k_1 k_3$	k_2	α_1	β_1
1	0.8	1.05	0.49	1.08	0.97
2	1.4	0.91	0.48	0.94	0.97
3	1.7	0.95	0.48	0.99	0.97
4	1.7	0.90	0.47	0.95	0.95
5	2.2	0.88	0.48	0.91	0.96
6	2.9	0.82	0.44	0.93	0.88
7	3.0	0.81	0.46	0.89	0.91
8	3.1	0.90	0.46	0.99	0.91
9	3.7	0.74	0.48	0.78	0.95
10	3.9	0.80	0.48	0.83	0.97
11	4.6	0.76	0.43	0.88	0.86
12	5.2	0.73	0.45	0.81	0.90
13	5.4	0.63	0.44	0.71	0.88
14	5.5	0.72	0.47	0.76	0.94
15	5.7	0.67	0.41	0.81	0.83
16	6.1	0.64	0.42	0.77	0.83
17	6.5	0.65	0.39	0.82	0.79
18	6.5	0.65	0.42	0.78	0.84
19	6.8	0.65	0.41	0.79	0.81
20	7.6	0.63	0.37	0.84	0.75
21	7.6	0.65	0.41	0.80	0.81
22	10.3	0.57	0.39	0.72	0.79
23	10.4	0.56	0.38	0.73	0.76

*Obtained from Ibrahim (1994)

Sargin et al. (1971)			
Specimen ID	f'_c (ksi)	k_3	ϵ_{cu}
PE-1	4.35	0.93	0.0035
PE-2	4.68	1.02	0.0028
PE-3	4.18	0.97	0.0039

Nedderman (1973)						
Specimen ID	f'_c (ksi)	$k_1 k_3$	k_2	α_1	β_1	ϵ_{cu}
2	13.85	0.59	0.37	0.80	0.74	0.0032
4	13.85	0.55	0.39	0.71	0.78	0.0029
5	14.01	0.61	0.37	0.82	0.74	0.0034
6	14.16	0.59	0.38	0.78	0.76	0.0032
7	11.50	0.57	0.34	0.84	0.68	0.0030
9	12.97	0.58	0.36	0.81	0.72	0.0033
10	12.43	0.58	0.36	0.81	0.72	0.0032
11	12.05	0.56	0.38	0.74	0.76	0.0028
12	12.20	0.55	0.38	0.72	0.76	0.0030

Kaar et al. (1978a)							
Specimen ID	f'_c (ksi)	k_1	k_2	k_3	α_1	β_1	ϵ_{cu}
A6	6.50	0.75	0.45	1.01	0.84	0.90	0.0045
A6P	6.90	0.73	0.45	0.98	0.80	0.90	0.0038
A8	8.47	0.70	0.39	0.93	0.83	0.78	0.0034
A10	10.41	0.68	0.37	0.92	0.85	0.74	0.0036
A10P	9.38	0.64	0.36	1.00	0.89	0.72	0.003
A12	11.34	0.62	0.36	1.12	0.96	0.72	0.0031
A14	14.00	0.56	0.34	1.03	0.85	0.68	0.0033
A14P	13.28	0.65	0.37	1.09	0.96	0.74	0.0038
D6	6.57	0.75	0.42	0.98	0.86	0.84	0.0028
D6P	7.10	0.74	0.42	0.89	0.79	0.84	0.0033
D8	8.42	0.64	0.40	0.95	0.76	0.80	0.0024
D10	9.81	0.66	0.38	0.94	0.82	0.76	0.0031
D10P	10.10	0.63	0.41	0.98	0.76	0.82	0.0032
D12	11.18	0.59	0.36	1.20	0.99	0.72	0.0032
D14	12.87	0.55	0.38	1.08	0.78	0.76	0.0028
D14P	14.85	0.60	0.36	1.03	0.86	0.72	0.0035
F10	11.29	0.65	0.37	0.99	0.86	0.74	0.0032
F12	12.69	0.57	0.36	1.12	0.89	0.72	0.0029
F14	13.70	0.65	0.38	0.91	0.78	0.76	0.0035
E4	4.24	0.66	0.35	1.16	1.09	0.70	0.0024
E6	6.44	0.61	0.47	0.92	0.70	0.94	0.0052
E8	8.21	0.67	0.36	0.99	0.92	0.72	0.0038
E8P	8.45	0.68	0.46	0.84	0.62	0.92	0.0037
E10	11.33	0.58	0.33	1.06	0.92	0.66	0.0037
E12	12.49	0.63	0.37	0.75	0.64	0.74	0.0031
E12P	11.95	0.60	0.34	0.97	0.85	0.68	0.0036
C4	3.63	0.75	0.43	1.03	0.90	0.86	0.0036
C4P	3.56	0.77	0.44	0.95	0.84	0.88	0.0051
C6	6.01	0.64	0.42	0.96	0.73	0.84	0.0035
C8	8.21	0.61	0.34	0.94	0.84	0.68	0.003
C8P	8.15	0.58	0.38	0.96	0.74	0.76	0.003
C10	9.57	0.57	0.34	1.06	0.90	0.68	0.0032
C12	9.68	0.53	0.39	1.20	0.82	0.78	0.0029
C12P	9.99	0.45	0.43	1.19	0.62	0.86	0.0029

Kaar et al. (1978b)							
Specimen ID	f'_c (ksi)	k_1	k_2	k_3	α_1	β_1	ϵ_{cu}
1	3.33	0.74	0.44	1.07	0.90	0.88	0.0030
20	3.20	0.75	0.44	1.23	1.05	0.88	0.0030
17	6.75	0.74	0.45	0.95	0.78	0.90	0.0040

Swartz et al. (1985)							
Specimen ID	f'_c (ksi)	k_1	k_2	k_3	α_1	β_1	ϵ_{cu}
1	8.40	0.77	0.41	1.11	1.04	0.82	0.0020
2	9.40	0.73	0.37	0.90	0.89	0.74	0.0031
3B	9.69	0.73	0.41	0.96	0.85	0.82	0.0027
3C	9.69	0.73	0.41	0.94	0.84	0.82	0.0026
4A	11.18	0.81	0.43	0.92	0.87	0.86	0.0033
4B	11.18	0.72	0.37	0.97	0.94	0.74	0.0031
4C	11.18	0.68	0.37	1.06	0.97	0.74	0.0028
4D	11.18	0.63	0.34	0.97	0.90	0.68	0.0024

Pastor (1986)							
Specimen No	f'_c (ksi)	k_1	k_2	k_3	α_1	β_1	ϵ_{cu}
SP-1	2.60	0.88	0.48	0.96	0.88	0.96	0.0043
SP-2	2.70	0.86	0.48	0.93	0.83	0.96	0.0036
SP-3	5.35	0.81	0.44	0.90	0.83	0.88	0.0036
SP-4	6.17	0.79	0.41	0.91	0.88	0.82	0.0038
SP-8	8.65	0.64	0.38	1.06	0.89	0.76	0.0032
SP-9	9.19	0.67	0.37	0.97	0.88	0.74	0.0034
SP-10	9.20	0.69	0.41	0.99	0.83	0.82	0.0035
SP-11	9.37	0.70	0.39	0.98	0.88	0.78	0.0031
SP-12	11.00	0.65	0.33	0.93	0.92	0.66	0.0031
SP-13	11.57	0.63	0.36	0.96	0.84	0.72	0.0032

Schade (1992)							
Specimen ID	f'_c (ksi)	k_1	k_2	k_3	α_1	β_1	ϵ_{cu}
col 1	15.36	0.56	0.36	1.00	0.78	0.71	0.0030
col 2	15.36	0.62	0.36	0.97	0.85	0.71	0.0034
col 3	15.36	0.60	0.36	0.96	0.81	0.71	0.0033
col 4	15.36	0.65	0.37	0.94	0.83	0.73	0.0036
col 5	15.36	0.65	0.36	0.92	0.83	0.72	0.0035
col 6	15.36	0.67	0.37	0.89	0.83	0.73	0.0039
col 7	15.90	0.68	0.35	0.94	0.91	0.70	0.0032
col 8	15.90	0.67	0.35	0.94	0.89	0.70	0.0032
col 9	15.90	0.70	0.37	0.93	0.88	0.74	0.0037
col 10	15.90	0.70	0.36	0.91	0.89	0.72	0.0030
col 11	15.90	0.70	0.36	0.91	0.88	0.72	0.0032
col 12	15.90	0.65	0.37	0.90	0.80	0.73	0.0034

Ibrahim (1994)							
Specimen ID	f'_c (ksi)	k_1	k_2	k_3	α_1	β_1	ϵ_{cu}
V1	10.25	0.50	0.26	0.92	0.87	0.53	0.00331
V2	12.01	0.64	0.35	0.97	0.88	0.70	0.00458
V4	10.56	0.71	0.39	0.90	0.81	0.79	0.00348
V5	18.10	0.59	0.34	0.92	0.78	0.69	0.00396
V6	14.33	0.63	0.37	0.89	0.75	0.75	0.00378
V7	12.28	0.67	0.36	0.87	0.81	0.72	0.00369
V5	18.75	0.55	0.35	1.00	0.79	0.69	0.00396
V10	12.97	-	-	-	-	-	-
V11	18.49	0.57	0.35	0.82	0.68	0.69	0.00389
V12	17.56	0.58	0.35	0.91	0.74	0.71	0.00408
V13	10.52	0.71	0.39	0.87	0.80	0.78	0.00372
V14	18.09	0.58	0.34	0.88	0.74	0.68	0.00398
V15	18.10	0.57	0.34	0.98	0.81	0.69	0.004
V16	8.60	0.67	0.37	1.08	0.98	0.74	0.00363
V17	18.52	0.57	0.36	0.92	0.74	0.71	0.00433
T1 ^x	11.75	0.71	0.53	1.01	0.48	1.06	-
T4 ^x	18.72	0.64	0.52	1.09	0.43	1.03	-
T2 ^x	12.97	0.70	0.55	1.07	0.48	1.10	-
T5 ^x	18.94	0.64	0.50	0.99	0.41	1.00	-
T3 ^x	12.18	0.74	0.55	1.01	0.50	1.11	-
T6 ^x	17.09	0.67	0.53	0.92	0.38	1.07	-

^x are the triangular specimens.

Tan et al. (2005)						
Specimen ID	f'_c (ksi)	$k_1 k_3$	k_2	α_1	β_1	ϵ_{cu}
S40-A-N2	6.74	0.69	0.40	0.86	0.80	0.0029
S40-A-N3	7.01	0.65	0.38	0.86	0.76	0.0029
S40-B-N3	7.01	0.76	0.40	0.95	0.80	0.0040
S40-B-N4	7.11	0.72	0.39	0.92	0.78	0.0037
S40-B-N5	7.11	0.75	0.37	1.01	0.74	0.0033
S40-C-N1	6.60	0.83	0.41	1.01	0.82	0.0051
S40-C-N2	7.01	0.80	0.40	1.00	0.80	0.0041
S70-A-N	10.36	0.54	0.38	0.71	0.76	0.0030
S70-B-N1	10.05	0.55	0.39	0.71	0.78	0.0041
S70-B-N2	11.04	0.59	0.39	0.76	0.78	0.0044
S70-C-N	10.05	0.58	0.40	0.73	0.80	0.0042
S90-B-N	12.98	0.57	0.41	0.70	0.82	0.0038
S90-B-N1	12.98	0.42	0.36	0.58	0.72	0.0032
S90-B-N2	14.59	0.44	0.36	0.61	0.72	0.0031
S90-E-N1	13.79	0.49	0.37	0.66	0.74	0.0031
S90-E-N2	13.40	0.52	0.37	0.70	0.74	0.0033
S90-E-N3	14.59	0.52	0.34	0.76	0.68	0.0030
S40-B-E20/2*	7.11	0.84	0.44	0.95	0.88	0.0039
S40-B-E40/1*	7.11	0.77	0.37	1.04	0.74	0.0042
S40-B-E40/2*	7.11	0.85	0.41	1.04	0.82	0.0039
S40-B-E60/1*	7.11	0.71	0.35	1.01	0.70	0.0037
S40-B-E60/2*	7.11	0.77	0.41	0.94	0.82	0.0040
S70-B-E20*	11.04	0.55	0.35	0.79	0.70	0.0038
S70-B-E40*	11.04	0.57	0.36	0.79	0.72	0.0037
S70-B-E60*	11.04	0.57	0.39	0.73	0.78	0.0035

* column specimens subjected to eccentrically applied axial load

APPENDIX B – MATERIAL PROPERTIES**PROPERTIES OF FINE AGGREGATE (NATURAL SAND)****Sieve Analysis of Natural Sand (Perform at NCSU CFL)**

Sieve Size	Cumulative Passing
3/8 in	100.0
1/4 in	100.0
#4	98.2
#8	90.1
#16	72.7
#50	12.8
#100	1.6

PROPERTIES OF FINE AGGREGATE (MANUFACTURED SAND)

Provided by Carolina Sunrock Corporation

Carolina Sunrock Corporation

Physical Properties for 2MS Concrete Sand
Butner Quarry #178

May 12, 2003

ASTM C 136 Method for Sieve Analysis of Fine and Coarse Aggregates

Average Gradation from 1/1/03 to 4/29/03		Specifications		
		NCDOT	AASHTO M 6	ASTM C 33
3/8" (9.5mm)	100.0	100	100	→
#4 (4.75mm)	99.7	95-100	95-100	→
#8 (2.36mm)	88.7	84-100	80-100	→
#16 (1.18mm)	66.2	45-95	50-85	→
#30 (.600mm)	48.8	25-75	25-60	→
#40 (.425mm)	39.4			
#50 (.300mm)	30.9	8-35	10-30	→
#80 (.180mm)	18.1			
#100 (.150mm)	13.3	0.5-20	2-10	→
Fineness Modulus	2.53	± 0.20	± 0.20	→

ASTM C117 Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing

		Specifications		
		NCDOT	AASHTO M 6	ASTM C 33
#200	% Finer 5.0%	0-8	<5%	→

ASTM C 88 Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate (5 cycles)

		Specifications		
		NCDOT	AASHTO M 6	ASTM C 33
Grading Size Combined	Weighted % Loss 0.5%	<15%	<10%	→

AASHTO T 21 Organic Impurities in Fine Aggregates for Concrete

		Specifications		
		NCDOT	AASHTO M 6	ASTM C 33
	% Organic 0.0%			

ASTM C 142 Clay Lumps and Friable Particles in Aggregates

		Specifications		
		NCDOT	AASHTO M 80	ASTM C 33
	% Deleterious 0.0%	<1.0%	<3.0%	<2.0%

Carolina Sunrock CorporationPhysical Properties for 2MS Concrete Sand
Butner Quarry #178

May 12, 200

ASTM C 29**Unit Weight and Voids in Aggregate**

Dry Rodded Unit Weight	122.0 lb/ft ³	%Voids	33.9%
Loose Unit Weight	112.9 lb/ft ³	%Voids	38.9%

ASTM C 128**Specific Gravity and Absorption of Fine Aggregate**

Bulk Specific Gravity (Gsb)	2.959	% Absorption	0.64%
Bulk Specific Gravity SSD	2.981		
Apparent Specific Gravity (Gsa)	3.003		

ASTM C 1252**Fine Aggregate Angularity**
(Standard Graded Sample)

Uncompacted Air Voids	46.6%

ASTM D 2419**Sand Equivalent Value of Soils and Fine Aggregate**

Sand Equivalent	84

PROPERTIES OF COARSE AGGREGATE

Provided by Carolina Sunrock Corporation

Carolina Sunrock Corporation

Physical Properties for #78M Crushed Stone
Butner Quarry #178

May 12, 2003

ASTM C 136 Method for Sieve Analysis of Fine and Coarse Aggregates

Average Gradation from
1/1/01 to 4/25/03

	Average Gradation from 1/1/01 to 4/25/03	Specifications
		NCDOT
3/4"	100.0	100
1/2"	100.0	98-100
3/8"	98.7	75-100
#4	26.8	20-45
#8	3.7	0-15

ASTM C117 Test Method for Materials Finer than 75- μ m (No. 200) Sieve in Mineral Aggregates by Washing

#200

% Finer
0.2%

Specifications		
NCDOT	AASHTO M 80	ASTM C 33
< 0.6%	< 1.5%	→

ASTM C 131 Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine

Grading Size
"B"

% Wear
18%

Specifications		
NCDOT	AASHTO M 80	ASTM C 33
<55%	<50%	→

ASTM C 88 Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate (5 cycles)

Grading Size
> 3/8"
> #4
Combined

Weighted
% Loss
0.4%
0.1%
0.5%

Specifications		
NCDOT	AASHTO M 80	ASTM C 33
<15%	<12%	→

ASTM C 142 Clay Lumps and Friable Particles in Aggregates

% Deleterious
0.0%

Specifications		
NCDOT	AASHTO M 80	ASTM C 33
<3.2%	<2.0%	→

Test results indicate that #78 stone meets or exceeds the requirements of ASTM C 33, Class 5S and AASHTO M 80, Class A Coarse Aggregate in Portland Cement Concrete.

Carolina Sunrock CorporationPhysical Properties for #78M Crushed Stone
Butner Quarry #178

May 12, 2003

ASTM C 29 Unit Weight and Voids in Aggregate

Dry Rodded Unit Weight	109.7 lb/ft ³	%Voids	40.6%
Loose Unit Weight	101.9 lb/ft ³	%Voids	44.8%

ASTM C 127 Specific Gravity and Absorption of Coarse Aggregate

Bulk Specific Gravity (Gsb)	2.958	% Absorption	0.42%
Bulk Specific Gravity SSD	2.970		
Apparent Specific Gravity (Gsa)	3.006		

PaDOT Test Method 621 Determining the Percentage of Crushed Fragments in Gravel

One Fractured Face	100%
Two Fractured Faces	100%

ASTM D 4791 Flat or Elongated Particles in Coarse Aggregate

3:1	3.4%
5:1	0.2%

Test results indicate that #78 stone meets or exceeds the requirements of NCDOT Standard Specifications 2002 and Superpave Mix Design SP-2 for use in Asphalt Concrete.

PROPERTIES OF SILICA FUME

Provided by Elkem Materials

Product Specification November 2005*

Elkem Microsilica[®] 971

<u>Analysis/Properties</u>		<u>Limits</u>
SiO ₂	(Silicon dioxide, amorphous)	min. 97.5 %
C	(Carbon)	max. 0.8 %
Fe ₂ O ₃	(Iron oxide)	max. 0.3 %
Al ₂ O ₃	(Aluminium oxide)	max. 0.7 %
CaO	(Calcium oxide)	max. 0.3 %
MgO	(Magnesium oxide)	max. 0.5 %
K ₂ O	(Potassium oxide)	max. 0.6 %
Na ₂ O	(Sodium oxide)	max. 0.3 %
P ₂ O ₅	(Phosphorus pentoxide)	max. 0.10 %
SO ₃	(Sulphur trioxide)	max. 0.4 %
Cl	(Chloride)	max. 0.1 %
H ₂ O	(Moisture, when packed)	max. 0.8 %
Loss on Ignition (L.O.I.) @ 975 °C		max. 1.0 %
Coarse Particles; >45 µm (325 mesh)		max. 0.6 %
pH-value (fresh)		4.5 – 7.5
Undensified 971-U		
Bulk Density (when packed)		300-450 kg/m ³
Densified 971-D		
Bulk Density (when packed)		500-600 kg/m ³

Material data given in this Product Specification refer to analysis according to Elkem Standard Test Methods for Microsilica (ESTMS). Test methods are available upon request.

*For info. on latest update, pls. contact microsilica.materials@elkem.no (discard outdated versions)



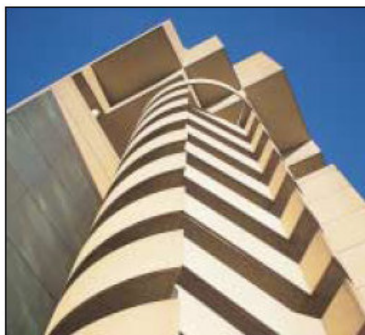
PROPERTIES OF SILICA FUME

Provided by Boral Material Technologies

BORAL FLY ASH

Class F fly ash

For **improved** durability
and ASR resistance.



Features

*In plastic concrete,
Boral Fly Ash:*

- Reduces bleeding and segregation

*In hardened concrete,
depending on mixture
proportions used,
Boral Fly Ash:*

- Increases ultimate strength
- Reduces drying shrinkage
- Decreases permeability
- Lowers heat of hydration
- Reduces creep

Product Description

Boral Fly Ash is a pozzolan for concrete, consisting of the “finely divided residue that results from the combustion of ground or powdered coal” as defined by ASTM C 618. A pozzolan, as defined by ASTM, reacts chemically with calcium hydroxide produced by the hydration of portland cement to form additional cementitious compounds.

Boral Class F fly ash significantly increases the ability of concrete to resist attack from sulfates in soil or ground water. Additionally, Class F fly ash has been proven through extensive research and field experience to be highly effective in mitigating the deleterious effects of expansive alkali-silica reactions (ASR) in concrete. It is produced from the combustion of pulverized bituminous or Texas lignite coal.

When correctly proportioned, concrete which contains fly ash can have equivalent or greater 28-day compressive strengths when compared to plain Portland cement concrete. Due to the pozzolanic reaction fly ash concrete will continue to gain strength beyond 28 days exceeding that of plain Portland cement concrete.

Major Benefits

- Makes for easier placement.
- Improves pumpability.
- Improves finishability.
- Reduces water requirements.
- Improves durability.

Applications

Boral Class F fly ash can be used as a pozzolan in virtually any concrete application. When correctly proportioned Class F fly ash will add many benefits such as increased strength, increased durability and reduced permeability. Class F fly ash is particularly beneficial in high performance concrete applications where high compressive strengths are required or where severe exposure conditions demand highly durable concrete. Class F fly ash is also very effective at mitigating problems associated with alkali-silica reactions. In mass concrete placements where low heats of hydration are required class F fly ash is very advantageous in controlling temperature rise.

Boral
Material
Technologies



BORAL FLY ASH

Class F fly ash

Enhance concrete performance.

Quality & Specifications

Boral Class F fly ash which is provided for use in concrete applications will meet or exceed the performance requirements of ASTM C 618. To ensure compliance with ASTM C 618 our on-site and central quality control laboratories carefully sample and test the fly ash according to the requirements of ASTM C 311. In addition independent commercial testing laboratories provide additional compliance testing.

Shipping and Delivery

Boral Fly Ash is normally shipped, stored and batched in the same manner as portland cement. Your trained Boral representative can suggest the most appropriate and economical procedure for given conditions.

Boral Material Technologies

Boral Material Technologies is a major processor and marketer of coal combustion products in the United States. With over 40 years of marketing experience, Boral is committed to supplying quality products broadly supported with skilled technical sales professionals. To meet both our customer's present and future needs with coal combustion products Boral continues its commitment to customer based research and development and broad based marketing programs.

For more information on our complete line of products, contact your local Boral representative, corporate office or visit us online at www.boralmti.com.



Boral Class F fly ash provides enhanced characteristics in plastic and in hardened concrete. This pozzolan improves finishability and achieves higher ultimate strengths compared to plain portland cement concrete.

Regional Sales Offices

Phoenix, AZ 800 528-6630
Fontana, CA 800 864-4521
Denver, CO 800 255-0663
Auburndale, FL 800 329-6337
Atlanta, GA 800 241-4943
Greensboro, NC 800 822-8896
San Antonio, TX 800 292-5354

Corporate Office

(210) 349-4069
(800) 964-0951
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www.boralmti.com



Because Boral Material Technologies Inc. cannot control the final use of its products, there are no warranties expressed or implied regarding a product's use or performance in any given circumstance. Persons receiving this information should make their own tests to determine suitability for their particular use.

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PROPERTIES OF HIGH RANGE WATER REDUCING AGENT

Provided by Degussa Admixtures, Inc.



GLENIUM® 3030 NS

Full-Range Water-Reducing Admixture

3 4	Product Data	
	03 30 00	Cast-In-Place Concrete
	03 40 00	Precast Concrete
	03 70 00	Mass Concrete
	04 05 16	Masonry Grouting

Description

Glenium 3030 NS ready-to-use full-range water-reducing admixture is a patented new generation of admixture based on polycarboxylate chemistry. Glenium 3030 NS admixture is very effective in producing concretes with different levels of workability including applications that require the use of Rheodynamic® self-consolidating concrete (SCC). Glenium 3030 NS admixture meets ASTM C 494 requirements for Type A, water-reducing, and Type F, high-range water-reducing, admixtures.

Applications

Recommended for use in:

- Concrete where high flowability, high-early and ultimate strengths and increased durability are needed
- Production of Rheodynamic SCC mixtures
- Concrete where normal, mid-range, or high-range water-reduction is desired
- Concrete where normal setting times are required
- 4x4™ Concrete for fast track construction

Features

- Reduced water content for a given slump
- Dosage flexibility for normal, mid and high-range water reduction
- Produces cohesive and non-segregating concrete mixture
- Increased compressive strength and flexural strength performance at all ages
- Providing faster setting times and strength development
- Enhanced finishability and pumpability

Benefits

- Providing economic benefits to the entire construction team through higher productivity and reduced variable costs

Performance Characteristics

Mixture Data: 600 lb/yd³ of Type I cement (360 kg/m³); slump, 8.5-9.25 in. (210-235 mm); non-air-entrained concrete; dosage rate adjusted to obtain 25-30% water reduction.

Setting Time

Mixture	Initial Set (h:min)	Difference (h:min)
Plain	4:24	–
Conventional Superplasticizer	6:00	+ 1.36
Glenium 3030 NS	5:00	+0.36

Compressive Strength

Mixture	1 day		7 days	
	psi	MPa	psi	MPa
Plain	1700	12	4040	28
Conventional Superplasticizer	3460	24	6380	44
Glenium 3030 NS	4120	28	7580	52

Slump Retention - In. (mm)

Mixture	Minutes		
	15	30	45
Plain:	8.5 (215)	8.5 (215)	7.5 (200)
Conventional Superplasticizer	8.5 (215)	4.25 (110)	3.5 (90)
Glenium 3030 NS	9.25 (235)	9.25 (235)	8.25 (210)

GLENIUM® 3030 NS *continued*

Rate of Hardening: Glenium 3030 NS admixture is formulated to produce normal setting characteristics throughout its recommended dosage range. Setting time of concrete is influenced by the chemical and physical composition of the basic ingredients of the concrete, temperature of the concrete and ambient conditions. Trial mixtures should be made with actual job materials to determine the dosage required for a specified setting time and a given strength requirement.

Guidelines for Use

Dosage: Glenium 3030 NS admixture has a recommended dosage range of up to 3 fl oz/cwt (195 mL/100 kg) for Type A applications, 3-6 fl oz/cwt (195-390 mL/100 kg) for mid-range use and up to 18 fl oz/cwt (1,170 mL/100 kg) for Type F applications. The dosage range is applicable to most concrete mixtures using typical concrete ingredients. However, variations in job conditions and concrete materials, such as silica fume, may require dosages outside the recommended range. In such cases, contact your local Degussa Admixtures, Inc. representative.

Mixing: Glenium® 3030 NS admixture can be batched with the initial mixing water or as a delayed addition. However, optimum water reduction is generally obtained with a delayed addition.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: Glenium 3030 NS admixture will neither initiate nor promote corrosion of reinforcing steel embedded in concrete, prestressed concrete or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of Glenium 3030 NS admixture.

Compatibility: Glenium 3030 NS admixture is compatible with most admixtures used in the production of quality concrete, including normal, mid-range and high-range water-reducing admixtures, air-entrainers, accelerators, retarders, extended set control admixtures, corrosion inhibitors, and shrinkage reducers.

Do not use Glenium 3030 NS admixture with admixtures containing beta-naphthalene-sulfonate. Erratic behaviors in slump, slump flow, and pumpability may be experienced.

For directions on the proper evaluation of Glenium 3030 NS admixture in specific applications, contact your Degussa Admixtures, Inc. representative.

Storage and Handling

Storage Temperature: If Glenium 3030 NS admixture freezes, thaw at 45 °F (7 °C) or above and completely reconstitute by mild mechanical agitation.

Do not use pressurized air for agitation.

Shelf Life: Glenium 3030 NS admixture has a minimum shelf life of 12 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your Degussa Admixtures, Inc. representative regarding suitability for use and dosage recommendations if the shelf life of Glenium 3030 NS admixture has been exceeded.

Packaging

Glenium 3030 NS admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Material Safety Data Sheets: Glenium 3030 NS admixture.

Additional Information

For additional information on Glenium 3030 NS admixture or on its use in developing concrete mixes with special performance characteristics, contact your Degussa Admixtures, Inc. representative.

Degussa Admixtures, Inc. is a leading provider of innovative chemical admixtures and silica fume for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets in the United States and Canada. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.



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PROPERTIES OF RETARDING AGENT

Provided by Degussa Admixtures, Inc.

degussa.
creating essentials

DELVO® STABILIZER

Hydration Controlling Admixture

3	Product Data	
	03 30 00	Cast-In-Place Concrete
	03 40 00	Precast Concrete
	03 70 00	Mass Concrete

Description

DELVO Stabilizer is a ready-to-use, liquid admixture for making more uniform and predictable high-performance concrete. DELVO Stabilizer admixture retards setting time by controlling the hydration of portland cement and other cementitious materials while facilitating placing and finishing operations. It can be used to stabilize returned plastic concrete and concrete washwater to reduce waste and increase profits. DELVO Stabilizer admixture meets ASTM C 494 requirements for Type B, retarding, and Type D, water-reducing and retarding, admixtures.

Applications

Recommended for use in:

- Stabilization of concrete washwater
- Stabilization of returned plastic concrete
- Stabilization of freshly batched concrete for long hauls
- 4x4™ Concrete
- Pumped concrete, shotcrete (wet mix) and conventionally-placed concretes
- Plain, reinforced, precast, prestressed, lightweight and normal weight concrete.

Features

- Reduced water content required for a given workability
- Retarded-setting time characteristics
- Improved workability
- Reduced segregation

Benefits

- Provides flexibility in the scheduling of placing and finishing operations
- Offsets the effects of slump loss during extended delays between mixing and placing
- Reduces waste associated with concrete washwater and returned concrete
- Increased strength – compressive and flexural

Performance Characteristics

Rate of Hardening: The temperature of a concrete mixture and the ambient temperature (forms, earth, air, etc.) affect the hardening rate of concrete. At higher temperatures, concrete hardens more rapidly which may cause problems with placing and finishing.

One of the functions of DELVO Stabilizer admixture is to retard the set of concrete. Within the normal dosage range, it will generally extend the working and setting times of concrete containing normal portland cement, fly ash, slag and silica fume approximately 1 hour to 5 hours compared to a plain concrete mixture. This depends on job materials and temperatures. Trial mixes should be made under approximate job conditions to determine the dosage required.

Compressive Strength: Concrete produced with Delvo Stabilizer admixture will develop higher early (within 24 hours) and higher ultimate strengths than plain concrete when used within the recommended dosage range and under normal, comparable curing conditions. When DELVO Stabilizer admixture is used in heat-cured concrete, the length of the preheating period should be increased until the initial set of the concrete is achieved. The actual heat-curing period is then reduced accordingly to maintain existing production cycles without sacrificing early or ultimate strengths.

Guidelines for Use

Dosage: DELVO Stabilizer admixture is recommended for use at a dosage of 4 + 1 fl oz/100 lb (260 + 65 mL/100 kg) of cementitious materials for most concrete mixtures using average concrete ingredients. Because of variations in job conditions and concrete materials, dosages other than the recommended amounts may be required. In such cases, contact your Degussa Admixtures, Inc. representative. For concrete washwater and returned concrete stabilization, utilize Delvo charts or the Delvomatic™ software to determine the appropriate dosage rates.

DELVO® STABILIZER *continued*

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: DELVO Stabilizer admixture will neither initiate nor promote corrosion of reinforcing steel in concrete. This admixture does not contain intentionally-added calcium chloride or other chloride-based ingredients.

Compatibility: DELVO Stabilizer admixture may be used in combination with any Degussa Admixtures, Inc. admixture. When used in conjunction with another admixture, each admixture must be dispensed separately into the mix.

Storage and Handling

Storage Temperature: If DELVO Stabilizer admixture freezes, thaw at 35 °F (2 °C) or above and completely reconstitute by mild mechanical agitation. **Do not use pressurized air for agitation.**

Shelf Life: Delvo Stabilizer admixture has a minimum shelf life of 12 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your Degussa Admixtures, Inc. representative regarding suitability for use and dosage recommendations if the shelf life of Delvo Stabilizer admixture has been exceeded.

Packaging

Delvo Stabilizer admixture material is supplied in a specially designed 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Material Safety Data Sheets: Delvo Stabilizer admixture.

Additional Information

For more information on DELVO Stabilizer admixture, contact your Degussa Admixtures, Inc. representative.

Degussa Admixtures, Inc. is a leading provider of innovative chemical admixtures and silica fume for specialty concrete used in the ready mix, precast, manufactured concrete products, underground construction and paving markets in the United States and Canada. The Company's respected Master Builders brand products are used to improve the placing, pumping, finishing, appearance and performance characteristics of concrete.



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APPENDIX C – CYLINDER COMPRESSION TESTS FOR EACH SPECIMEN

Specimen ID	Age of Specimens at Testing Day (days)	Strength of Cylinder 1 (ksi)	Strength of Cylinder 2 (ksi)	Strength of Cylinder 3 (ksi)	Strength of Cylinder 4 (ksi)	Average f'_c at Testing Day (ksi)	Coefficient of Variation	Control Type
10EB1	63	10.914	10.847	11.373	11.053	11.046	2.12	Excellent
10EB2	109	10.980	11.305	12.063	11.281	11.407	4.05	Tight
10EB3	111	12.190	11.513	11.419	-	11.708	3.59	Tight
10EB4	63	10.273	9.937	10.852	-	10.354	4.47	Tight
10EB5	62	11.341	11.405	9.958	-	10.901	7.50	Tight
14EB1	49	14.748	14.824	14.317	-	14.630	1.87	Excellent
14EB2	51	14.370	13.939	14.646	-	14.318	2.49	Excellent
14EB3	52	14.542	14.901	14.604	-	14.682	1.31	Excellent
14EB4	57	14.707	15.320	15.099	-	15.042	2.06	Excellent
14EB5	100	15.604	15.368	15.098	-	15.357	1.65	Excellent
14EB6	101	14.796	15.642	15.020	-	15.152	2.89	Tight
18EB1	76	15.912	15.995	15.581	-	15.829	1.38	Excellent
18EB2	77	16.014	15.935	16.012	-	15.987	0.28	Excellent
18EB3	81	15.229	15.868	15.686	-	15.594	2.11	Excellent
18EB4	82	15.424	16.188	15.670	-	15.761	2.48	Excellent
18EB5	83	16.424	15.807	15.915	-	16.049	2.05	Excellent
18EB6	84	16.007	15.055	15.404	-	15.489	3.11	Tight
18EB7	96	15.045	15.036	15.028	-	15.036	0.06	Excellent
18EB8	97	13.808	14.682	14.960	-	14.483	4.15	Tight
18EB9	99	14.578	14.678	15.149	-	14.802	2.06	Excellent
18EB10	102	13.975	15.188	-	-	14.582	5.88	Tight

$$\text{Coefficient of Variation (CV)} = \frac{\text{Standard Deviation}}{\text{Average Value}}$$

CV < 2.5 → Excellent Control

CV < 2.5 → Tight Control

APPENDIX D – TESTS RESULTS FOR POISSON’S RATIO

Komendant et al. (1978)		Perenchio and Klieger (1978)		Carrasquillo et al. (1981)		Swartz et al. (1985)	
f'_c (ksi)	ν	f'_c (ksi)	ν	f'_c (ksi)	ν	f'_c (ksi)	ν
6.28	0.22	11.16	0.24	4.91	0.19	9.68	0.16
6.78	0.22	11.58	0.22	4.70	0.22	9.68	0.16
6.66	0.23	11.18	0.21	4.67	0.20	9.68	0.16
7.62	0.22	10.16	0.28	8.52	0.21	9.68	0.16
9.50	0.22	11.12	0.26	8.29	0.21	11.18	0.23
6.59	0.19	10.60	0.2	8.57	0.24	11.18	0.23
7.18	0.22	9.55	0.25	10.67	0.22	11.18	0.23
8.93	0.22	10.86	0.23	11.05	0.21	11.18	0.23
6.56	0.20	9.48	0.21	11.10	0.21	11.5	0.13
7.40	0.21	8.98	0.24	4.91	0.19	11.5	0.13
9.65	0.22	10.24	0.24	4.40	0.19		
		9.44	0.23	4.57	0.19		
		8.10	0.22	7.86	0.21		
		9.60	0.22	7.10	0.20		
		8.57	0.24	7.14	0.21		
		7.22	0.23	9.29	0.21		
		8.34	0.221	9.52	0.25		
		7.39	0.2	9.50	0.21		
				10.57	0.20		

Jerath and Yamane (1987)		Radain et al. (1993)		Iravani (1996)		Logan (2005)	
f'_c (ksi)	ν	f'_c (ksi)	ν	f'_c (ksi)	ν	f'_c (ksi)	ν
9.52	0.19	5.77	0.15	9.33	0.15	9.57	0.12
8.36	0.22	6.14	0.15	9.47	0.16	9.57	0.15
8.34	0.18	6.61	0.16	9.57	0.11	5.06	0.11
8.16	0.21	6.94	0.17	9.60	0.20	5.06	0.18
6.63	0.21	7.25	0.17	9.62	0.15	5.06	0.17
7.60	0.18	7.74	0.18	9.63	0.19	5.06	0.14
5.59	0.22	8.49	0.19	9.63	0.14	10.09	0.17
5.52	0.23	9.07	0.18	9.73	0.16	10.09	0.14
5.22	0.20	9.25	0.17	13.18	0.19	8.27	0.13
4.31	0.21	9.41	0.17	13.66	0.15	8.27	0.12
4.30	0.18	9.57	0.17	13.91	0.18	8.27	0.15
3.73	0.23	9.83	0.19	14.14	0.19	8.27	0.11
		9.93	0.19	14.20	0.16	9.76	0.16
		10.71	0.21	15.01	0.18	9.76	0.16
		11.34	0.22	14.85	0.20	9.76	0.14
		11.47	0.22	15.24	0.18	9.76	0.13
		11.52	0.21	15.40	0.18	10.83	0.12
		12.13	0.22	15.45	0.18	10.83	0.13
		12.42	0.22	15.39	0.16	10.35	0.15
		13.05	0.23	16.75	0.20	10.35	0.12
				17.36	0.19	10.35	0.11
				17.49	0.19	12.11	0.10
				17.64	0.20	12.11	0.14
				18.13	0.19	11.63	0.20
						11.63	0.18
						11.63	0.13
						11.63	0.17
						10.52	0.17
						10.52	0.22
						12.78	0.10
						12.13	0.24
						12.13	0.24

Logan (2005)		Logan (2005)	
f'_c (ksi)	ν	f'_c (ksi)	ν
12.75	0.17	11.38	0.24
12.75	0.18	11.38	0.19
12.75	0.18	11.38	0.19
12.75	0.15	5.99	0.22
6.17	0.27	5.99	0.20
6.17	0.29	5.99	0.22
6.17	0.29	13.13	0.11
6.17	0.29	13.13	0.11
13.47	0.27	13.13	0.26
13.47	0.15	13.13	0.18
13.47	0.16	12	0.21
11.35	0.17	12	0.22
11.35	0.18	12	0.19
13.99	0.22	13.81	0.24
14.53	0.17	13.81	0.14
14.53	0.14	13.81	0.17
14.53	0.14	14.95	0.20
14.53	0.24	14.95	0.26
14.24	0.15	14.95	0.10
14.24	0.15	14.95	0.12
14.24	0.18	14.4	0.17
14.24	0.16	14.4	0.22
15.64	0.23	16.73	0.17
15.64	0.19	16.73	0.20
15.64	0.25	16.73	0.21
14.12	0.19	16.73	0.21
14.12	0.25	15.64	0.22
14.12	0.23	15.64	0.19
14.12	0.27	15.64	0.26
14.64	0.15	15.64	0.27
14.64	0.18	14.69	0.16
16.61	0.17	14.69	0.25
16.61	0.19	14.69	0.22
16.61	0.12	14.69	0.20
16.05	0.19	16.81	0.15
16.05	0.16	16.81	0.12
16.05	0.17	16.81	0.17
16.05	0.16	16.81	0.20
11.38	0.11	16.43	0.16
16.43	0.24	16.43	0.22
11.38	0.24	16.43	0.21

APPENDIX E – PROGRAM FOR THE DATALOGGER

Reading 64 Two-Wired Strain Gages with Two AM416 Multiplexers and One CR23X

This strain gage program will multiplex 64 full bridge two wired strain gages on two AM416 multiplexer unit by using one CR23X.

This program was specifically developed for the creep racks of NCHRP 12-64 Project, Application of the LRFD Bridge Design Specifications to HSC: Flexure and Compression.

This program is for Micro Measurements electrical resistance strain gage type YFLA-5-3L with a gage factor of $2.11 \pm 2\%$. The gage resistance of YFLA-5-3L is 120.4 ± 0.5 ohm. This program is written for the gages to be in connection with a MR1-350-127 Full Bridge Completion Module. The correction factor -1900 in this program was established using the shunt check device. The shunt resistance values can be found from the website below:

http://www.vishay.com/brands/measurements_group/guide/tn/tn514/514ref1j.htm

The used values in this website were "Active Arms:1", "Gage Resistance:120", "Gage Factor:2.11" and "Leadwire Resistance:0"

Simulated Strain (Mε)	Shunt Resistance	Device Coding
500	113684	112150
2000	28376	028200
4000	14158	014076
5000	11314	011270

Output Format: All strain gauge values are in microstrain.

Wiring for the Program

Wiring for this program must be connected in the following manner:

MR1-350-127 (#1)	AM416 (#1)	CR23X
P+ (left)	H1	EX1
S+		SE1
P- (left)		Ground (near EX1)
S- (left)	L1	SE2

MR1-350-127 (#2)	AM416 (#1)	CR23X
P+ (left)	H2	EX1
S+		SE3
P- (right)		Ground (near EX1)
S- (left)	L2	SE4

AM416 (#1)	AM416 (#2)
H1	H1
L1	L1
H2	H2
L2	L2

CR23X	AM416 (#1)
12V (between 12V and G)	12V
G (between SW12 and 12V)	GRD
EX2	CLK
C1	RES

CR23X	AM416 (#2)
12V (between 12V and G)	12V
G (between SW12 and 12V)	GRD
EX2	CLK
C2	RES

AM416	YFLA-5 (#1)	YFLA-5 (#17)
H1	Wire	
L1	Wire	
H2		Wire
L2		Wire

Summary of the Program

1. CR23X excites the first AM416 multiplexer to read the first 16 channels which are the strain gages connected to H1's and L1's.
2. CR23X excites the first AM416 multiplexer to read the second 16 channels which are the strain gages connected to H2's and L2's.
3. CR23X excites the second AM416 multiplexer to read the first 16 channels which are the strain gages connected to H1's and L1's.
4. CR23X excites the second AM416 multiplexer to read the second 16 channels which are the strain gages connected to H2's and L2's.

CR23X Program

*Table 1 Program

01: 1 Execution Interval (seconds)

; Synchronize measurement cycle to real time (every 5 seconds).

1: If time is (P92)

1: 0000 -- Minutes (Seconds --) into a

2: 5 -- Interval (same units as above)

3: 30 Then Do

; Set control port C1 to high, this turns on the first multiplexer connected to port C1.

2: Do (P86)

1: 41 Set Port 1 High


```
; Loop for 1 measurement.
3: Beginning of Loop (P87)
  1: 0      Delay
  2: 1      Loop Count

; Loop for first 16 channels of the first multiplexer (all L1's and H1's first).
4: Beginning of Loop (P87)
  1: 0      Delay
  2: 16     Loop Count

; Excite the first multiplexer by EX2.
5: Delay w/Opt Excitation (P22)
  1: 2      Ex Channel
  2: 1      Delay W/Ex (units = 0.01 sec)
  3: 1      Delay After Ex (units = 0.01 sec)
  4: 5000   mV Excitation

; Excite the strain gauge for voltage measurements by EX1 and collect data with DIFF1
; (SE1 and SE2). The collected data will be stored in the first 16 locations between
; rawsg_1 and rawsg_16.
6: Full Bridge (P6)
  1: 1      Reps
  2: 21     10 mV, 60 Hz Reject, Slow Range
  3: 1      DIFF Channel
  4: 1      Excite all reps w/Exchan 1
  5: 2500   mV Excitation
  6: 1      -- Loc [ rawsg_1 ]
  7: 1.0    Mult
  8: 0.0    Offset

; Convert the full bridge reading from mv/v to microstrain. The converted data will be
; stored in the fifth 16 locations between strn_1 and strn_16.
7: Z=X*F (P37)
  1: 1      -- X Loc [ rawsg_1 ]
  2: -1900   F
  3: 65     -- Z Loc [ strn_1 ]

8: End (P95)

; Set control port C1 to low, this turns off the first multiplexer connected to port C1.
9: Do (P86)
  1: 51     Set Port 1 Low

10: End (P95)

; Set control port C1 to high, this turns on the first multiplexer connected to port C1.
11: Do (P86)
  1: 41     Set Port 1 High

; Loop for 1 measurement.
```

```
12: Beginning of Loop (P87)
1: 0    Delay
2: 1    Loop Count

; Loop for second 16 channels of the first multiplexer (all L2's and H2's).
13: Beginning of Loop (P87)
1: 0    Delay
2: 16   Loop Count

; Excite the first multiplexer by EX2.
14: Delay w/Opt Excitation (P22)
1: 2    Ex Channel
2: 1    Delay W/Ex (units = 0.01 sec)
3: 1    Delay After Ex (units = 0.01 sec)
4: 5000 mV Excitation

; Excite the strain gauge for voltage measurements by EX1 and collect data with DIFF2
; (SE3 and SE4). The collected data will be stored in the second 16 locations between
; rawsg_17 and rawsg_32.
15: Full Bridge (P6)
1: 1    Reps
2: 21   10 mV, 60 Hz Reject, Slow Range
3: 2    DIFF Channel
4: 1    Excite all reps w/Exchan 1
5: 2500 mV Excitation
6: 17   -- Loc [ rawsg_17 ]
7: 1.0   Mult
8: 0.0   Offset

; Convert the full bridge reading from mv/v to microstrain. The converted data will be
; stored in the sixth 16 locations between strn_17 and strn_32.
16: Z=X*F (P37)
1: 17   -- X Loc [ rawsg_17 ]
2: -1900 F
3: 81   -- Z Loc [ strn_17 ]

17: End (P95)

; Set control port C1 to low, this turns off the first multiplexer connected to port C1.
18: Do (P86)
1: 51   Set Port 1 Low

19: End (P95)

; Set control port C2 to high, this turns on the second multiplexer connected to port C2.
20: Do (P86)
1: 42   Set Port 2 High

; Loop for 1 measurement.
21: Beginning of Loop (P87)
```

1: 0 Delay
2: 1 Loop Count

; Loop for first 16 channels of the second multiplexer (all L1's and H1's first).

22: Beginning of Loop (P87)

1: 0 Delay
2: 16 Loop Count

; Excite the second multiplexer by EX2.

23: Delay w/Opt Excitation (P22)

1: 2 Ex Channel
2: 1 Delay W/Ex (units = 0.01 sec)
3: 1 Delay After Ex (units = 0.01 sec)
4: 5000 mV Excitation

; Excite the strain gauge for voltage measurements by EX1 and collect data with DIFF1

; (SE1 and SE2). The collected data will be stored in the third 16 locations between

; rawsg_33 and rawsg_48.

24: Full Bridge (P6)

1: 1 Reps
2: 21 10 mV, 60 Hz Reject, Slow Range
3: 1 DIFF Channel
4: 1 Excite all reps w/Exchan 1
5: 2500 mV Excitation
6: 33 -- Loc [rawsg_33]
7: 1.0 Mult
8: 0.0 Offset

; Convert the full bridge reading from mv/v to microstrain. The converted data will be

; stored in the seventh 16 locations between strn_33 and strn_48.

25: $Z=X \cdot F$ (P37)

1: 33 -- X Loc [rawsg_33]
2: -1900 F
3: 97 -- Z Loc [strn_33]

26: End (P95)

; Set control port C2 to low, this turns off the second multiplexer connected to port C2.

27: Do (P86)

1: 52 Set Port 2 Low

28: End (P95)

; Set control port C2 to high, this turns on the second multiplexer connected to port C2.

29: Do (P86)

1: 42 Set Port 2 High

; Loop for 1 measurement.

30: Beginning of Loop (P87)

1: 0 Delay

```

2: 1    Loop Count

; Loop for second 16 channels of the second multiplexer (all L2's and H2's).
31: Beginning of Loop (P87)
1: 0    Delay
2: 16   Loop Count

; Excite the second multiplexer by EX2.
32: Delay w/Opt Excitation (P22)
1: 2    Ex Channel
2: 1    Delay W/Ex (units = 0.01 sec)
3: 1    Delay After Ex (units = 0.01 sec)
4: 5000 mV Excitation

; Excite the strain gauge for voltage measurements by EX1 and collect data with DIFF2
; (SE3 and SE4). The collected data will be stored in the forth 16 locations between
; rawsg_49 and rawsg_64.
33: Full Bridge (P6)
1: 1    Reps
2: 21   10 mV, 60 Hz Reject, Slow Range
3: 2    DIFF Channel
4: 1    Excite all reps w/Exchan 1
5: 2500 mV Excitation
6: 49   -- Loc [ rawsg_49 ]
7: 1.0  Mult
8: 0.0  Offset

; Convert the full bridge reading from mv/v to microstrain. The converted data will be
; stored in the eighth 16 locations between strn_49 and strn_64.
34: Z=X*F (P37)
1: 49   -- X Loc [ rawsg_49 ]
2: -1900 F
3: 113  -- Z Loc [ strn_49 ]

35: End (P95)

; Set control port C2 to low, this turns off the second multiplexer connected to port C2.
36: Do (P86)
1: 52   Set Port 2 Low

37: End (P95)

; Writes the data into final storage.
38: Do (P86)
1: 10   Set Output Flag High (Flag 0)

; Gives an id to your array.
39: Set Active Storage Area (P80)
1: 1    Final Storage Area 1
2: 10   Array ID

```

; This is written as the first four columns of the data line.

40: Real Time (P77)

1: 1111 Year,Day,Hour/Minute,Seconds (midnight = 0000)

; Writes the data stored into the input storage area. Only the 64 strain readings are stored
;in the final storage area.

41: Sample (P70)

1: 64 Repts

2: 65 -- Loc [strn_1]

42: End (P95)

*Table 2 Program

02: 0.0000 Execution Interval (seconds)

*Table 3 Subroutines

End Program

			Data Locations		
1	[rawsg_1]	RW--	1	1	-----
2	[rawsg_2]	----	0	0	-----
3	[rawsg_3]	----	0	0	-----
4	[rawsg_4]	----	0	0	-----
5	[rawsg_5]	----	0	0	-----
6	[rawsg_6]	----	0	0	-----
7	[rawsg_7]	----	0	0	-----
8	[rawsg_8]	----	0	0	-----
9	[rawsg_9]	----	0	0	-----
10	[rawsg_10]	----	0	0	-----
11	[rawsg_11]	----	0	0	-----
12	[rawsg_12]	----	0	0	-----
13	[rawsg_13]	----	0	0	-----
14	[rawsg_14]	----	0	0	-----
15	[rawsg_15]	----	0	0	-----
16	[rawsg_16]	----	0	0	-----
17	[rawsg_17]	RW--	1	1	-----
18	[rawsg_18]	----	0	0	-----
19	[rawsg_19]	----	0	0	-----
20	[rawsg_20]	----	0	0	-----
21	[rawsg_21]	----	0	0	-----
22	[rawsg_22]	----	0	0	-----
23	[rawsg_23]	----	0	0	-----
24	[rawsg_24]	----	0	0	-----
25	[rawsg_25]	----	0	0	-----
26	[rawsg_26]	----	0	0	-----
27	[rawsg_27]	----	0	0	-----
28	[rawsg_28]	----	0	0	-----
29	[rawsg_29]	----	0	0	-----

30	[rawsg_30]	----	0	0	-----
31	[rawsg_31]	----	0	0	-----
32	[rawsg_32]	----	0	0	-----
33	[rawsg_33]	RW--	1	1	-----
34	[rawsg_34]	----	0	0	-----
35	[rawsg_35]	----	0	0	----- End
36	[rawsg_36]	----	0	0	-----
37	[rawsg_37]	----	0	0	-----
38	[rawsg_38]	----	0	0	-----
39	[rawsg_39]	----	0	0	-----
40	[rawsg_40]	----	0	0	-----
41	[rawsg_41]	----	0	0	-----
42	[rawsg_42]	----	0	0	-----
43	[rawsg_43]	----	0	0	-----
44	[rawsg_44]	----	0	0	-----
45	[rawsg_45]	----	0	0	-----
46	[rawsg_46]	----	0	0	-----
47	[rawsg_47]	----	0	0	-----
48	[rawsg_48]	----	0	0	-----
49	[rawsg_49]	-W--	0	1	-----
50	[rawsg_50]	----	0	0	----- End
51	[rawsg_51]	----	0	0	-----
52	[rawsg_52]	----	0	0	-----
53	[rawsg_53]	----	0	0	-----
54	[rawsg_54]	----	0	0	-----
55	[rawsg_55]	----	0	0	-----
56	[rawsg_56]	----	0	0	-----
57	[rawsg_57]	----	0	0	-----
58	[rawsg_58]	----	0	0	-----
59	[rawsg_59]	----	0	0	-----
60	[rawsg_60]	----	0	0	-----
61	[rawsg_61]	----	0	0	-----
62	[rawsg_62]	----	0	0	-----
63	[rawsg_63]	----	0	0	-----
64	[rawsg_64]	----	0	0	-----
65	[strn_1]	RW--	1	1	-----
66	[strn_2]	R---	1	0	-----
67	[strn_3]	R---	1	0	-----
68	[strn_4]	R---	1	0	-----
69	[strn_5]	R---	1	0	-----
70	[strn_6]	R---	1	0	-----
71	[strn_7]	R---	1	0	-----
72	[strn_8]	R---	1	0	-----
73	[strn_9]	R---	1	0	-----
74	[strn_10]	R---	1	0	-----
75	[strn_11]	R---	1	0	-----
76	[strn_12]	R---	1	0	-----
77	[strn_13]	R---	1	0	-----
78	[strn_14]	R---	1	0	-----
79	[strn_15]	R---	1	0	-----

80	[strn_16]	R---	1	0	-----
81	[strn_17]	RW--	1	1	-----
82	[strn_18]	R---	1	0	-----
83	[strn_19]	R---	1	0	-----
84	[strn_20]	R---	1	0	-----
85	[strn_21]	R---	1	0	-----
86	[strn_22]	R---	1	0	-----
87	[strn_23]	R---	1	0	-----
88	[strn_24]	R---	1	0	-----
89	[strn_25]	R---	1	0	-----
90	[strn_26]	R---	1	0	-----
91	[strn_27]	R---	1	0	-----
92	[strn_28]	R---	1	0	-----
93	[strn_29]	R---	1	0	-----
94	[strn_30]	R---	1	0	-----
95	[strn_31]	R---	1	0	-----
96	[strn_32]	R---	1	0	-----
97	[strn_33]	RW--	1	1	-----
98	[strn_34]	R---	1	0	-----
99	[strn_35]	R---	1	0	-----
100	[strn_36]	R---	1	0	-----
101	[strn_37]	R---	1	0	-----
102	[strn_38]	R---	1	0	-----
103	[strn_39]	R---	1	0	-----
104	[strn_40]	R---	1	0	-----
105	[strn_41]	R---	1	0	-----
106	[strn_42]	R---	1	0	-----
107	[strn_43]	R---	1	0	-----
108	[strn_44]	R---	1	0	-----
109	[strn_45]	R---	1	0	-----
110	[strn_46]	R---	1	0	-----
111	[strn_47]	R---	1	0	-----
112	[strn_48]	R---	1	0	-----
113	[strn_49]	RW--	1	1	-----
114	[strn_50]	R---	1	0	-----
115	[strn_51]	R---	1	0	-----
116	[strn_52]	R---	1	0	-----
117	[strn_53]	R---	1	0	-----
118	[strn_54]	R---	1	0	-----
119	[strn_55]	R---	1	0	-----
120	[strn_56]	R---	1	0	-----
121	[strn_57]	R---	1	0	-----
122	[strn_58]	R---	1	0	-----
123	[strn_59]	R---	1	0	-----
124	[strn_60]	R---	1	0	-----
125	[strn_61]	R---	1	0	-----
126	[strn_62]	R---	1	0	-----
127	[strn_63]	R---	1	0	-----
128	[strn_64]	R---	1	0	-----

APPENDIX F – SIMPLIFIED STRESS – STRAIN RELATIONSHIPS FOR TEST SPECIMENS

10EB1		10EB2		10EB3		10EB4		10EB5	
Stress (ksi)	Strain ($\mu\epsilon$)	Stress (ksi)	Strain ($\mu\epsilon$)	Stress (ksi)	Strain ($\mu\epsilon$)	Stress (ksi)	Strain ($\mu\epsilon$)	Stress (ksi)	Strain ($\mu\epsilon$)
0	0	0	0	0	0	0	0	0	0
1.42	279	1.89	290	1.97	300	1.83	300	1.83	300
3.11	600	3.65	600	4	624	3.8	624	3.8	624
4.76	900	5.29	887	5.58	900	5.49	910	5.49	910
6.25	1200	6.89	1206	7.08	1200	7.09	1200	7.09	1200
7.47	1500	8.21	1500	8.5	1500	8.69	1516	8.8	1516
8.53	1800	9.44	1800	9.88	1800	10.05	1800	10.24	1800
9.36	2100	10.17	2012	11.2	2100	11.2	2100	11.42	2100
9.95	2400	10.8	2200	12.46	2414	12.16	2409	12.03	2409
10.38	2700	11.49	2406	13.27	2700	12.3	2600	12.45	2600
10.8	3000	12.06	2620	13.6	3000	12.44	2764	12.69	2764
11.2	3300	12.43	2771	13.69	3204	12.4	2974	12.4	2877
11.4	3495	12.72	2968	13.35	3348	11.03	3090	11.93	2974
9	3675	12.84	3148	12.31	3403	6.48	3100	6.48	2988

14EB1		14EB2		14EB3		14EB4		14EB5	
Stress (ksi)	Strain ($\mu\epsilon$)	Stress (ksi)	Strain ($\mu\epsilon$)	Stress (ksi)	Strain ($\mu\epsilon$)	Stress (ksi)	Strain ($\mu\epsilon$)	Stress (ksi)	Strain ($\mu\epsilon$)
0	0	0	0	0	0	0	0	0	0
2.04	300	2.04	300	2.11	300	2.07	300	2.16	300
4.27	624	4.27	624	4.27	624	4.4	624	4.4	624
6.1	900	6.1	900	6.29	900	6.24	900	6.34	900
7.93	1200	7.93	1200	8.22	1200	8.36	1200	8.36	1200
9.72	1500	9.6	1500	9.91	1500	10.38	1500	10.38	1500
11.36	1806	11.27	1800	11.88	1800	12.16	1800	12.44	1800
12.77	2100	12.58	2100	13.24	2100	13.66	2100	14.32	2100
13.94	2414	13.71	2414	14.6	2414	15.16	2414	15.2	2254
14.4	2700	14.51	2700	15.45	2700	16	2700	16.67	2583
14.69	2900	15.12	2900	16	2900	16.57	2900	16.9	2646
13.69	3068	15.45	2983	15.87	3068	15.87	3000	17.14	2713
11.64	3206	14.51	3080	14.88	3157	11.64	3022	17.42	2805
9.77	3316	13.71	3162	13.57	3177	9.77	3032	17.51	2868

Appendix F

14EB6		18EB1		18EB2		18EB3		18EB4	
Stress (ksi)	Strain (μϵ)	Stress (ksi)	Strain (μϵ)	Stress (ksi)	Strain (μϵ)	Stress (ksi)	Strain (μϵ)	Stress (ksi)	Strain (μϵ)
0	0	0	0	0	0	0	0	0	0
2.07	300	2.07	300	2.07	300	1.97	300	1.97	300
4.4	624	4.4	634	4.4	634	4.13	624	4.14	624
6.24	900	6.06	900	6.06	900	5.77	900	5.77	900
8.36	1200	7.89	1200	7.89	1200	7.6	1200	7.6	1200
10.23	1500	9.48	1500	9.48	1500	9.15	1516	9.39	1516
12.11	1800	10.8	1800	10.8	1800	10.52	1800	10.66	1800
13.66	2100	12.07	2100	12.07	2100	11.74	2100	11.78	2100
14.55	2264	12.4	2264	12.4	2264	12	2264	12.4	2269
15.35	2448	12.96	2448	12.96	2448	12.35	2448	13.6	2757
16.1	2641	13.05	2641	13.57	2641	12.58	2641	13.8	3003
16.43	2728	13.05	2963	13.66	2963	11.13	2781	14.08	3109
16.62	2790	12.8	3327	13.8	3200	10.05	2843	14.04	3206
17.09	2954	8.54	3531	12.11	3327	8.08	2872	13.71	3300

18EB5		18EB6		18EB7		18EB8		18EB9	
Stress (ksi)	Strain (μϵ)	Stress (ksi)	Strain (μϵ)	Stress (ksi)	Strain (μϵ)	Stress (ksi)	Strain (μϵ)	Stress (ksi)	Strain (μϵ)
0	0	0	0	0	0	0	0	0	0
1.97	300	1.97	300	2.17	300	2.17	300	2.17	300
4.23	624	4.23	624	4.4	624	4.4	640	4.29	624
6	900	6	910	6.37	910	6.18	910	6	910
7.89	1200	7.6	1200	8.16	1200	7.88	1200	7.92	1200
9.62	1516	9.39	1516	10.09	1516	9.76	1516	9.53	1516
11.03	1800	10.52	1800	11.51	1800	11.2	1800	11.13	1800
12.35	2100	11.74	2100	13.02	2100	12.59	2100	12.55	2119
13.29	2409	12.72	2409	14.25	2409	13.77	2409	13.68	2409
13.52	2757	13.52	2757	15	2714	14.43	2714	14.56	2714
13.6	2911	13.6	2911	15.85	3012	14.86	3012	15.28	3012
13.66	3027	13.66	3027	14.53	3448	14.53	3470	15.75	3277
13.15	3080	13.66	3332	11.93	3520	11.93	3487	15.6	3476
10.7	3094	7.32	3388	6.48	3586	6.48	3503	6.48	3487

18EB10	
Stress (ksi)	Strain (μϵ)
0	0
2.17	314
4.25	624
6.04	910
7.88	1200
9.67	1516
10.99	1800
12.31	2100
13.54	2409
14.2	2714
14.2	2963
11.79	3233
10	3437
6.48	3520

APPENDIX G – CREEP AND SHRINKAGE TEST DATA

10Rack1 (Initial Elastic Strain = 336 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
1	30	0	0.0000	0.000	0.000
2	30	13	0.0067	0.040	0.030
3	30	2	0.0009	0.005	0.004
4	30	20	0.0102	0.061	0.046
7	30	34	0.0168	0.100	0.076
8	30	41	0.0204	0.122	0.092
9	30	50	0.0250	0.149	0.113
14	30	47	0.0233	0.139	0.105
16	30	44	0.0219	0.131	0.099
18	30	47	0.0235	0.140	0.106
22	30	55	0.0275	0.164	0.124
30	30	81	0.0406	0.242	0.183
35	30	75	0.0377	0.225	0.170
39	30	90	0.0450	0.268	0.203
49	30	85	0.0426	0.254	0.192
56	30	92	0.0459	0.273	0.207
63	30	95	0.0477	0.284	0.215
72	30	117	0.0583	0.347	0.263
80	30	121	0.0605	0.361	0.273
88	30	126	0.0632	0.376	0.285
99	30	128	0.0638	0.380	0.288
112	25	133	0.0665	0.396	0.300
126	45	141	0.0705	0.420	0.320
140	34	155	0.0774	0.461	0.351
155	25	164	0.0818	0.487	0.371
175	33	174	0.0871	0.519	0.395
196	25	182	0.0911	0.543	0.413
224	58	191	0.0953	0.568	0.436
256	60	198	0.0991	0.590	0.457
326	62	210	0.1051	0.626	0.490
381	27	218	0.1091	0.650	0.508

10Rack2 (Initial Elastic Strain = 468 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
14	30	0	0.0000	0.000	0.000
15	30	84	0.0421	0.180	0.136
16	30	114	0.0572	0.244	0.185
17	30	120	0.0601	0.257	0.194
18	30	127	0.0636	0.272	0.206
22	30	166	0.0829	0.354	0.268
30	30	228	0.1142	0.488	0.369
35	30	248	0.1241	0.530	0.402
39	30	256	0.1280	0.547	0.414
49	30	274	0.1368	0.584	0.443
56	30	283	0.1416	0.605	0.458
63	30	300	0.1501	0.641	0.486
72	30	322	0.1612	0.688	0.522
80	30	324	0.1618	0.691	0.524
88	30	328	0.1638	0.700	0.530
99	30	338	0.1691	0.723	0.547
112	25	355	0.1773	0.758	0.573
126	45	357	0.1787	0.763	0.578
140	34	367	0.1835	0.784	0.594
155	25	377	0.1884	0.805	0.609
175	33	380	0.1900	0.812	0.614
196	25	391	0.1953	0.834	0.631
224	58	384	0.1922	0.821	0.619
256	60	391	0.1957	0.836	0.633
326	62	401	0.2006	0.857	0.653
374	71	378	0.1889	0.807	0.602

10Rack3 (Initial Elastic Strain = 352 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
28	30	0	0.0000	0	0.000
29	30	52	0.0259	0.147355164	0.112
29	30	52	0.0259	0.147355164	0.112
30	30	65	0.0326	0.185138539	0.140
31	30	64	0.0321	0.182619647	0.138
32	30	82	0.0412	0.234256927	0.177
35	30	96	0.0479	0.272040302	0.206
39	30	114	0.0569	0.323173804	0.245
49	30	132	0.0661	0.375314861	0.284
56	30	138	0.0689	0.391687657	0.297
56	30	138	0.0689	0.391687657	0.297
63	30	156	0.0782	0.444584383	0.337
72	30	176	0.0878	0.498740554	0.378
80	30	188	0.0942	0.535264484	0.406
88	30	183	0.0913	0.518891688	0.393
99	30	195	0.0973	0.552896725	0.419
112	25	201	0.1004	0.570528967	0.432
126	45	204	0.1022	0.580604534	0.440
140	34	214	0.1071	0.608312343	0.462
155	25	223	0.1117	0.634760705	0.481
175	33	228	0.1142	0.64861461	0.492
196	25	235	0.1175	0.667506297	0.506
224	58	227	0.1133	0.643576826	0.484
256	60	232	0.1159	0.658690176	0.498
326	62	239	0.1193	0.677581864	0.516
428	23	254	0.1270	0.721662469	0.548
507	44	261	0.1306	0.741813602	0.564
690	78	285	0.1425	0.809823678	0.637

10Rack4 (Initial Elastic Strain = 884 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
8	30	0	0.0000	0.000	0.000
9	30	169	0.0422	0.191	0.145
10	30	324	0.0810	0.367	0.278
11	30	350	0.0876	0.396	0.300
14	30	443	0.1108	0.502	0.380
15	30	471	0.1178	0.533	0.404
16	30	501	0.1254	0.567	0.430
18	30	527	0.1318	0.596	0.452
22	30	579	0.1447	0.655	0.496
30	30	628	0.1570	0.710	0.538
35	30	693	0.1733	0.784	0.594
39	30	707	0.1768	0.800	0.606
49	30	732	0.1830	0.828	0.627
56	30	745	0.1863	0.843	0.639
63	30	794	0.1985	0.898	0.681
72	30	806	0.2015	0.912	0.691
80	30	821	0.2053	0.929	0.704
88	30	825	0.2062	0.933	0.707
99	30	844	0.2109	0.954	0.723
112	25	858	0.2146	0.971	0.735
126	45	870	0.2175	0.984	0.746
140	34	888	0.2220	1.005	0.762
155	25	899	0.2247	1.017	0.771
175	33	908	0.2270	1.027	0.779
196	25	929	0.2322	1.051	0.796
224	58	925	0.2313	1.047	0.793
256	60	933	0.2333	1.056	0.801
326	62	949	0.2373	1.074	0.818
428	23	994	0.2486	1.125	0.855
507	44	1016	0.2541	1.150	0.876
690	78	1061	0.2651	1.200	0.929

10Rack5 (Initial Elastic Strain = 799 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
14	30	0	0.0000	0.000	0.000
15	30	116	0.0290	0.145	0.110
16	30	176	0.0439	0.220	0.166
17	30	199	0.0498	0.249	0.189
18	30	210	0.0525	0.263	0.199
22	30	272	0.0679	0.340	0.258
23	30	290	0.0725	0.363	0.275
30	30	384	0.0961	0.481	0.364
35	30	410	0.1025	0.513	0.389
39	30	421	0.1054	0.528	0.400
49	30	447	0.1118	0.560	0.424
56	30	464	0.1159	0.580	0.440
63	30	491	0.1227	0.614	0.465
72	30	526	0.1316	0.659	0.499
80	30	540	0.1350	0.676	0.512
88	30	540	0.1351	0.676	0.512
99	30	562	0.1404	0.703	0.533
112	25	574	0.1434	0.718	0.544
126	45	579	0.1446	0.724	0.549
140	34	595	0.1486	0.744	0.564
155	25	610	0.1524	0.763	0.578
175	33	618	0.1544	0.773	0.586
196	25	638	0.1596	0.799	0.605
224	58	632	0.1579	0.791	0.597
256	60	643	0.1608	0.805	0.611
326	62	656	0.1640	0.821	0.626
428	23	691	0.1728	0.865	0.658
507	44	714	0.1786	0.894	0.682
690	78	748	0.1869	0.936	0.726

10Rack6 (Initial Elastic Strain = 820 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
28	30	0	0.0000	0.000	0.0000
29	30	115	0.0287	0.140	0.11
30	30	151	0.0377	0.184	0.14
31	30	167	0.0418	0.204	0.15
32	30	187	0.0467	0.228	0.17
35	30	236	0.0591	0.288	0.22
39	30	268	0.0671	0.327	0.25
49	30	316	0.0790	0.385	0.29
56	30	343	0.0858	0.418	0.32
63	30	369	0.0923	0.450	0.34
72	30	409	0.1023	0.499	0.38
80	30	430	0.1075	0.524	0.40
88	30	433	0.1082	0.528	0.40
99	30	456	0.1140	0.556	0.42
109	30	471	0.1177	0.574	0.43
112	25	474	0.1185	0.578	0.44
126	45	489	0.1222	0.596	0.45
140	34	516	0.1289	0.629	0.48
155	25	531	0.1328	0.648	0.49
175	33	548	0.1369	0.668	0.51
196	25	568	0.1421	0.693	0.53
224	58	571	0.1429	0.697	0.53
256	60	583	0.1457	0.711	0.54
326	62	605	0.1513	0.738	0.57
428	23	649	0.1624	0.792	0.61
507	44	686	0.1716	0.837	0.64
690	78	725	0.1813	0.884	0.70

14Rack1 (Initial Elastic Strain = 583 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
1	30	0	0.0000	0.000	0.000
2	30	49	0.0176	0.084	0.064
6	30	124	0.0442	0.212	0.161
7	30	133	0.0473	0.228	0.172
12	26	162	0.0580	0.279	0.210
14	38	165	0.0589	0.283	0.214
19	25	188	0.0670	0.322	0.242
23	30	190	0.0678	0.326	0.245
27	30	204	0.0728	0.350	0.264
30	22	218	0.0779	0.374	0.281
33	45	224	0.0801	0.385	0.290
41	49	224	0.0801	0.385	0.290
47	34	246	0.0877	0.422	0.318
54	28	252	0.0899	0.432	0.326
62	25	262	0.0937	0.451	0.340
75	56	282	0.1009	0.485	0.371
75	56	282	0.1009	0.485	0.371
82	33	294	0.1050	0.505	0.386
103	25	312	0.1115	0.536	0.409
131	58	322	0.1150	0.553	0.424
163	60	341	0.1218	0.585	0.454
233	62	360	0.1287	0.619	0.486
335	23	398	0.1422	0.683	0.533
414	44	436	0.1556	0.748	0.586
597	78	521	0.1860	0.894	0.743

14Rack2 (Initial Elastic Strain = 490 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
14	38	0	0.0000	0.000	0.000
15	28	56	0.0200	0.114	0.085
16	22	77	0.0275	0.157	0.117
19	25	121	0.0432	0.247	0.183
21	24	133	0.0475	0.271	0.200
23	30	137	0.0491	0.281	0.207
27	30	154	0.0551	0.315	0.233
29	22	187	0.0667	0.381	0.281
33	45	180	0.0644	0.368	0.271
41	49	200	0.0713	0.407	0.304
47	34	213	0.0760	0.434	0.325
54	28	221	0.0790	0.452	0.338
62	25	222	0.0792	0.452	0.338
75	56	234	0.0834	0.477	0.360
82	33	247	0.0882	0.504	0.381
103	25	268	0.0958	0.548	0.413
131	58	275	0.0983	0.562	0.427
163	60	278	0.0994	0.568	0.432
233	62	292	0.1042	0.595	0.458
335	23	313	0.1116	0.638	0.489
414	44	327	0.1169	0.668	0.514
597	78	345	0.1233	0.705	0.553

14Rack3 (Initial Elastic Strain = 587 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
28	53	0	0.0000	0.000	0.0000
29	22	44	0.0158	0.075	0.05
30	22	79	0.0282	0.134	0.10
33	45	100	0.0358	0.171	0.13
35	50	113	0.0402	0.192	0.15
37	21	122	0.0435	0.208	0.16
41	49	146	0.0523	0.249	0.19
47	34	165	0.0591	0.282	0.22
54	28	170	0.0608	0.290	0.22
62	25	175	0.0624	0.297	0.23
82	33	200	0.0714	0.340	0.26
103	25	219	0.0781	0.372	0.29
131	58	227	0.0809	0.386	0.30
163	60	235	0.0839	0.400	0.31
233	62	249	0.0888	0.423	0.33
335	23	271	0.0969	0.462	0.36
414	44	285	0.1017	0.485	0.38
597	78	302	0.1079	0.514	0.41

14Rack4 (Initial Elastic Strain = 1061 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
7	30	0	0.0000	0.000	0.000
8	30	164	0.0294	0.155	0.117
9	30	233	0.0416	0.220	0.166
12	26	399	0.0712	0.376	0.282
14	38	451	0.0805	0.425	0.321
19	25	497	0.0887	0.468	0.353
23	30	516	0.0922	0.487	0.367
27	30	532	0.0951	0.502	0.378
30	22	561	0.1001	0.528	0.397
33	45	566	0.1011	0.533	0.402
41	49	601	0.1073	0.566	0.430
47	34	629	0.1123	0.592	0.450
54	28	643	0.1148	0.606	0.460
62	25	647	0.1156	0.610	0.463
82	33	675	0.1206	0.636	0.483
103	25	699	0.1248	0.659	0.500
131	58	715	0.1278	0.674	0.514
163	60	724	0.1293	0.682	0.521
233	62	736	0.1315	0.694	0.532
335	23	771	0.1377	0.726	0.556
414	44	782	0.1397	0.737	0.565
597	78	814	0.1454	0.767	0.597

14Rack5 (Initial Elastic Strain = 984 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
14	38	0	0.0000	0.000	0.000
15	28	91	0.0163	0.093	0.069
16	22	137	0.0245	0.139	0.103
19	25	210	0.0375	0.214	0.158
21	24	234	0.0418	0.238	0.175
23	30	251	0.0449	0.255	0.189
27	30	274	0.0489	0.278	0.206
30	22	307	0.0549	0.312	0.231
33	45	317	0.0565	0.322	0.238
41	49	348	0.0621	0.354	0.266
47	34	377	0.0673	0.383	0.289
54	28	385	0.0687	0.391	0.295
62	25	391	0.0698	0.397	0.299
75	56	415	0.0742	0.422	0.322
82	33	427	0.0763	0.434	0.331
103	25	462	0.0825	0.469	0.357
131	58	480	0.0857	0.487	0.373
163	60	490	0.0876	0.498	0.383
233	62	512	0.0914	0.520	0.404
335	23	544	0.0971	0.552	0.427
414	44	565	0.1009	0.574	0.445
597	78	602	0.1075	0.612	0.485

14Rack6 (Initial Elastic Strain = 996 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
28	53	0	0.0000	0.000	0.000
29	22	77	0.0138	0.077	0.056
30	22	120	0.0214	0.120	0.087
33	45	157	0.0281	0.158	0.118
35	50	175	0.0312	0.175	0.133
37	21	197	0.0352	0.198	0.149
41	49	225	0.0402	0.226	0.174
47	34	252	0.0450	0.253	0.194
54	28	260	0.0464	0.261	0.200
62	25	272	0.0485	0.273	0.209
82	33	305	0.0545	0.306	0.235
103	25	331	0.0591	0.332	0.254
131	58	346	0.0618	0.348	0.268
163	60	355	0.0634	0.356	0.276
233	62	379	0.0677	0.381	0.299
335	23	406	0.0725	0.408	0.319
414	44	420	0.0750	0.421	0.330
597	78	462	0.0824	0.463	0.375

18Rack1 (Initial Elastic Strain = 875 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
1	32	0	0.0000	0.000	0.000
2	40	107	0.0298	0.137	0.110
3	28	137	0.0379	0.174	0.138
6	48	186	0.0516	0.237	0.191
8	40	206	0.0573	0.263	0.212
12	51	237	0.0659	0.302	0.247
14	59	236	0.0656	0.301	0.246
17	33	254	0.0704	0.323	0.263
21	62	263	0.0730	0.335	0.274
24	34	269	0.0748	0.343	0.280
27	30	296	0.0821	0.377	0.306
34	48	291	0.0808	0.371	0.300
38	25	320	0.0888	0.407	0.327
45	35	322	0.0895	0.411	0.330
54	60	336	0.0935	0.429	0.347
66	58	344	0.0957	0.439	0.356
98	60	367	0.1018	0.467	0.382
168	62	387	0.1074	0.493	0.406
270	23	424	0.1179	0.541	0.441
349	44	451	0.1252	0.575	0.469
532	78	485	0.1346	0.618	0.515

18Rack2 (Initial Elastic Strain = 722 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
14	59	0	0.0000	0.000	0.000
15	70	79	0.0219	0.109	0.109
16	26	116	0.0323	0.161	0.148
17	33	144	0.0400	0.200	0.177
19	36	173	0.0482	0.240	0.209
21	62	186	0.0516	0.257	0.225
24	34	204	0.0568	0.283	0.245
27	30	231	0.0640	0.319	0.273
29	22	233	0.0647	0.322	0.275
34	48	254	0.0704	0.351	0.300
38	25	271	0.0752	0.375	0.317
45	35	284	0.0788	0.393	0.331
54	60	292	0.0810	0.404	0.341
66	58	305	0.0847	0.423	0.358
98	60	322	0.0894	0.446	0.380
168	62	333	0.0925	0.461	0.394
270	23	365	0.1014	0.506	0.426
349	44	379	0.1053	0.525	0.443
532	78	388	0.1079	0.538	0.456

18Rack3 (Initial Elastic Strain = 626 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
27	30	0	0.0000	0.000	0.000
28	24	59	0.0165	0.095	0.069
29	22	65	0.0180	0.103	0.076
31	32	102	0.0284	0.164	0.122
34	48	106	0.0296	0.170	0.127
38	25	133	0.0369	0.213	0.158
45	35	149	0.0415	0.239	0.179
54	60	163	0.0453	0.261	0.199
66	58	180	0.0499	0.287	0.223
98	60	200	0.0557	0.320	0.254
168	62	210	0.0584	0.336	0.269
270	23	239	0.0665	0.383	0.303
349	44	245	0.0680	0.391	0.310
532	78	260	0.0722	0.415	0.335

18Rack4 (Initial Elastic Strain = 1540 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
7	49	0	0.0000	0.000	0.000
8	40	279	0.0388	0.181	0.146
9	46	393	0.0546	0.255	0.208
10	56	456	0.0634	0.296	0.245
12	51	536	0.0744	0.348	0.290
14	59	566	0.0786	0.367	0.308
17	33	625	0.0868	0.406	0.337
21	62	688	0.0956	0.447	0.376
24	34	714	0.0992	0.464	0.389
27	30	748	0.1039	0.486	0.406
34	48	781	0.1085	0.507	0.424
38	25	804	0.1117	0.522	0.435
45	35	825	0.1146	0.536	0.446
54	60	841	0.1167	0.546	0.455
66	58	856	0.1189	0.556	0.464
98	60	888	0.1233	0.576	0.483
168	62	916	0.1272	0.595	0.500
270	23	962	0.1336	0.625	0.522
349	44	981	0.1363	0.637	0.532
532	78	1017	0.1412	0.660	0.557

18Rack5 (Initial Elastic Strain = 1452 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
14	59	0	0.0000	0.000	0.000
15	70	189	0.0263	0.130	0.130
16	26	254	0.0353	0.175	0.163
17	33	313	0.0435	0.216	0.195
19	36	370	0.0514	0.255	0.226
21	62	405	0.0562	0.279	0.248
24	34	448	0.0623	0.309	0.271
27	30	494	0.0686	0.340	0.295
29	22	508	0.0705	0.350	0.302
34	32	539	0.0749	0.371	0.318
38	25	563	0.0782	0.388	0.331
45	35	586	0.0813	0.403	0.343
54	60	605	0.0840	0.416	0.355
66	58	626	0.0869	0.431	0.368
98	60	663	0.0921	0.457	0.392
168	62	694	0.0964	0.478	0.412
270	23	741	0.1030	0.510	0.436
349	44	768	0.1066	0.529	0.451
532	78	798	0.1108	0.549	0.473

18Rack6 (Initial Elastic Strain = 1339 $\mu\epsilon$)					
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Creep Strain ($\mu\epsilon$)	Average Specific Creep ($\mu\epsilon/\text{ksi}$)	Average Creep Coefficient	Adjusted Average Creep Coefficient
27	30	0	0.0000	0.000	0.0000
28	24	172	0.0239	0.128	0.09
29	22	202	0.0281	0.151	0.11
31	32	287	0.0398	0.214	0.16
34	48	336	0.0467	0.251	0.19
38	25	391	0.0543	0.292	0.22
45	35	439	0.0610	0.328	0.25
54	60	485	0.0674	0.362	0.28
66	58	520	0.0723	0.389	0.30
98	60	575	0.0799	0.429	0.34
168	62	624	0.0867	0.466	0.38
270	23	686	0.0953	0.512	0.41
349	44	722	0.1003	0.539	0.43
532	78	770	0.1069	0.575	0.47

10SC1				10SC2			
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Age of Concrete after Casting (days)	Relative Humidity (%)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)
1	30	0	0	8	30	0.00	0
2	30	43	28	9	30	23.94	15
3	30	77	49	10	30	30.15	19
4	30	85	54	11	30	62.07	40
8	30	90	58	14	30	107.29	68
9	30	85	54	15	30	119.70	76
14	30	78	50	16	30	122.36	78
15	30	89	56	17	30	133.00	85
16	30	88	56	18	30	144.53	92
18	30	87	55	22	30	160.49	102
22	30	92	59	23	30	153.39	98
30	30	105	67	28	30	181.77	116
35	30	129	82	29	30	200.39	128
39	30	117	75	30	30	197.73	126
49	30	137	87	31	30	198.61	126
56	30	153	98	32	30	186.20	119
63	30	159	101	35	30	219.01	139
72	30	167	106	39	30	224.50	143
80	30	181	115	49	30	242.06	154
88	30	169	108	56	30	264.23	168
99	30	204	130	63	30	268.66	171
112	25	219	139	72	30	289.94	185
126	45	228	146	80	30	309.45	197
140	34	233	149	88	30	292.60	186
155	25	231	148	99	30	321.86	205
175	33	217	139	109	22	348.46	221
196	25	245	155	112	25	346.69	220
224	58	231	143	126	45	358.21	228
256	60	231	143	140	34	367.97	235
326	62	234	146	155	25	369.74	236
381	27	261	163	175	33	352.01	224
382	28	259	162	196	25	375.95	239
383	30	278	174	224	58	372.40	236
384	29	284	178	256	60	366.19	230
387	31	274	171	326	62	362.65	227
408	23	270	169	374	71	383.93	249
428	23	272	170	375	73	392.79	258
507	44	260	161	376	74	393.68	259
690	78	211	106	377	44	375.95	246
				379	26	379.49	248
				381	27	383.04	250
				387	31	398.11	260
				408	23	398.11	260
				428	23	408.75	266
				507	44	405.21	264
				690	78	349.35	200

14SC1				14SC2			
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Age of Concrete after Casting (days)	Relative Humidity (%)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)
1	30	0	0	7	30	0	0
2	30	38	24	8	30	46	29
6	30	72	46	9	30	72	46
7	30	80	51	12	26	124	78
12	26	97	61	14	38	153	98
14	38	114	73	15	28	166	106
19	25	129	82	16	22	184	117
23	30	145	93	19	25	193	122
27	30	158	101	21	24	209	132
30	22	138	89	23	30	223	141
33	45	159	104	27	30	247	156
41	49	153	99	28	53	233	145
47	34	179	117	29	53	214	129
54	28	180	117	30	22	224	135
62	25	186	121	33	45	254	157
75	56	166	104	35	50	229	137
82	33	174	109	37	21	244	146
103	25	207	129	41	49	253	153
131	58	200	124	47	34	286	175
163	60	199	123	54	28	294	180
233	62	197	121	62	25	314	192
335	23	253	154	75	56	302	183
414	44	273	169	82	33	312	189
597	78	186	71	103	25	343	208
				131	58	340	206
				163	60	345	210
				233	62	344	209
				335	23	387	235
				414	44	405	248
				597	78	356	192

18SC1				18SC2			
Age of Concrete after Casting (days)	Relative Humidity (%)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Age of Concrete after Casting (days)	Relative Humidity (%)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)
1	32	0	0	7	49	0	0
2	40	14	10	8	40	39	27
3	28	37	24	9	46	51	37
6	48	43	29	10	56	76	57
8	40	56	38	12	51	120	91
12	51	82	59	14	59	133	103
14	59	76	53	15	70	153	123
17	33	106	72	16	26	172	135
21	62	114	80	17	33	180	140
24	34	119	83	19	36	188	145
27	30	142	98	21	62	215	169
34	48	160	112	24	34	221	173
38	25	176	121	27	30	255	195
45	35	182	125	28	24	269	203
54	60	194	136	29	22	274	207
66	58	191	133	31	32	276	208
98	60	210	150	34	48	286	216
168	62	229	167	38	25	301	224
270	23	300	209	45	35	309	230
349	44	325	228	54	60	335	253
532	78	279	176	66	58	332	251
				98	60	356	271
				168	62	378	291
				270	23	432	323
				349	44	455	340
				532	78	425	306

Age of Concrete after Casting (days)	Relative Humidity (%)	10SP1		10SP2		10SP3	
		Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)
1	30	0	0	0	0	0	0
2	30	117	74	108	69	89	56
3	30	88	56	114	72	111	71
4	30	102	65	134	85	114	73
8	30	128	82	145	92	137	87
9	30	131	83	159	101	154	98
14	30	179	114	202	129	206	131
16	30	179	114	205	130	211	135
22	30	188	120	211	134	211	135
30	30	213	136	247	158	240	153
39	30	222	141	256	163	266	169
49	30	248	158	273	174	288	184
56	30	253	161	290	185	300	191
63	30	259	165	282	179	300	191
72	30	273	174	296	188	308	196
80	30	282	179	307	196	326	207
88	30	273	174	299	190	317	202
99	30	302	192	324	206	334	213
112	25	307	195	341	217	343	218
128	50	282	176	313	195	326	205
140	34	342	215	375	236	371	235
157	33	379	239	410	258	420	267
164	48	381	241	418	265	428	273
175	33	350	221	381	241	383	243
196	25	427	268	475	298	463	292
257	60	333	185	381	216	366	207
326	62	299	155	327	167	331	176
507	44	444	261	461	265	497	297
690	78	327	129	367	158	360	142

Age of Concrete after Casting (days)	Relative Humidity (%)	10SP4		10SP5		10SP6	
		Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)	Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)	Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)
8	30	0	0	0	0	0	0
9	30	43	27	43	27	48	31
10	30	91	58	40	25	128	81
11	30	119	76	71	45	165	105
14	30	176	112	131	83	222	141
16	30	205	130	179	114	253	161
22	30	250	159	222	141	276	176
30	30	307	195	265	168	330	210
39	30	324	206	282	179	347	221
49	30	364	232	310	197	367	234
56	30	375	239	316	201	370	235
63	30	367	233	307	196	336	214
72	30	387	246	336	214	361	230
80	30	401	255	350	223	373	237
88	30	392	250	341	217	353	225
99	30	415	264	367	234	375	239
109	22	492	310	452	284	464	291
112	25	435	275	376	238	418	264
128	50	412	257	361	226	395	246
140	34	384	239	341	213	361	223
157	33	500	315	461	291	489	307
164	48	503	317	461	291	472	294
175	33	461	289	421	265	447	277
196	25	537	336	512	321	566	350
257	60	427	239	393	216	472	268
326	62	384	201	350	178	370	176
507	44	563	331	506	292	472	251
690	78	392	138	316	77	384	151

Appendix G

Age of Concrete after Casting (days)	Relative Humidity (%)	14SP1		14SP2		14SP3	
		Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)
1	30	0	0	0	0	0	0
2	30	23	15	26	16	17	11
6	30	71	45	85	54	68	43
7	30	83	53	94	60	77	49
12	26	151	95	179	112	162	101
14	38	168	107	188	118	171	107
19	25	134	86	154	97	136	86
23	30	160	102	162	103	162	103
27	30	168	107	171	108	162	103
30	22	160	102	171	108	154	98
35	50	143	89	162	101	145	91
41	49	177	115	191	123	179	117
47	34	134	87	188	121	171	112
64	33	251	164	273	177	264	173
71	48	263	172	282	184	267	175
82	33	177	116	202	132	188	123
103	25	277	177	333	211	344	218
164	60	177	90	256	144	273	156
233	62	203	113	213	106	205	95
414	44	345	217	401	243	375	219
597	78	228	85	279	104	253	81

Age of Concrete after Casting (days)	Relative Humidity (%)	14SP4		14SP5		14SP6	
		Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)	Average Shrink. Strain (µε)	Average Adjusted Shrink. Strain (µε)
7	30	0	0	0	0	0	0
8	30	68	43	63	40	65	42
9	30	119	76	114	73	120	76
12	26	196	123	194	122	205	128
14	38	222	141	217	137	228	144
16	22	250	158	251	157	253	159
19	25	202	128	208	131	219	139
23	30	213	135	234	148	248	157
27	30	236	150	251	159	265	168
30	22	228	145	236	150	259	164
35	50	247	160	245	157	265	169
41	49	282	187	285	188	290	188
47	34	279	185	282	186	296	192
64	33	375	248	376	247	359	233
71	48	384	255	387	256	387	255
82	33	307	204	311	206	316	208
103	25	387	253	447	289	484	310
164	60	307	183	370	221	401	238
233	62	307	183	311	168	324	169
414	44	521	339	481	292	478	281
597	78	381	181	362	157	376	165

Appendix G

Age of Concrete after Casting (days)	Relative Humidity (%)	18SP1		18SP2		18SP3	
		Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)	Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)	Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)
1	32	0	0	0	0	0	0
2	40	34	24	34	24	40	28
3	28	71	47	63	42	66	44
6	48	128	91	114	81	123	87
8	40	163	115	137	97	151	107
12	51	157	110	134	94	145	103
17	33	140	99	102	74	125	90
21	62	114	76	179	143	197	154
38	25	251	159	307	221	331	235
99	60	183	99	273	191	299	208
168	62	251	161	222	145	242	157
349	44	391	263	392	269	436	298
532	78	357	224	313	179	308	153

Age of Concrete after Casting (days)	Relative Humidity (%)	18SP4		18SP5		18SP6	
		Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)	Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)	Average Shrink. Strain (μϵ)	Average Adjusted Shrink. Strain (μϵ)
7	49	0	0	0	0	0	0
8	40	57	40	57	40	60	42
9	46	74	53	57	40	68	48
10	56	88	64	71	52	91	67
12	51	105	78	100	74	145	109
17	33	117	85	111	82	148	111
21	62	159	124	102	74	173	134
38	25	313	217	262	171	329	229
99	60	279	187	202	119	273	180
168	62	270	180	253	165	258	167
349	44	449	310	398	270	426	289
532	78	347	195	316	177	341	193

Age of Concrete after Casting (days)	Relative Humidity (%)	10SP1	10SP2	10SP3
		Weight Loss (%)	Weight Loss (%)	Weight Loss (%)
1	30	0.00	0.00	0.00
2	30	0.09	0.10	0.10
3	30	0.13	0.15	0.15
4	30	0.15	0.17	0.18
8	30	0.26	0.29	0.30
9	30	0.27	0.30	0.31
14	30	0.30	0.34	0.36
16	30	0.32	0.36	0.38
22	30	0.32	0.37	0.39
30	30	0.49	0.54	0.57
39	30	0.58	0.63	0.66
49	30	0.65	0.71	0.73
56	30	0.71	0.77	0.79
63	30	0.75	0.81	0.83
72	30	0.88	0.94	0.96
80	30	0.93	0.99	1.01
88	30	0.91	0.97	0.99
99	30	0.99	1.05	1.07
112	25	1.08	1.15	1.17
128	50	1.13	1.19	1.21
140	34	1.19	1.25	1.27
157	33	1.26	1.32	1.34
164	48	1.24	1.31	1.32
175	33	1.25	1.32	1.34
196	25	1.30	1.37	1.39
257	60	1.18	1.25	1.26
326	62	1.14	1.21	1.21
507	44	1.59	1.66	1.66
690	78	1.43	1.48	1.50

Age of Concrete after Casting (days)	Relative Humidity (%)	10SP4	10SP5	10SP6
		Weight Loss (%)	Weight Loss (%)	Weight Loss (%)
8	30	0.00	0.00	0.00
9	30	0.15	0.15	0.15
10	30	0.26	0.27	0.26
11	30	0.34	0.35	0.35
14	30	0.49	0.50	0.50
16	30	0.56	0.56	0.56
22	30	0.64	0.65	0.64
30	30	0.89	0.89	0.88
39	30	0.96	1.00	0.99
49	30	1.08	1.08	1.07
56	30	1.15	1.14	1.13
63	30	1.18	1.18	1.17
72	30	1.31	1.31	1.30
80	30	1.36	1.36	1.35
88	30	1.33	1.33	1.32
99	30	1.41	1.40	1.40
109	22	1.49	1.48	1.48
112	25	1.49	1.49	1.49
128	50	1.53	1.52	1.52
140	34	1.58	1.58	1.57
157	33	1.64	1.64	1.63
164	48	1.62	1.62	1.61
175	33	1.63	1.63	1.62
196	25	1.67	1.67	1.66
257	60	1.56	1.57	1.56
326	62	1.55	1.55	1.55
507	44	1.92	1.92	1.93
690	78	1.78	1.77	1.79

Age of Concrete after Casting (days)	Relative Humidity (%)	14SP1	14SP2	14SP3
		Weight Loss (%)	Weight Loss (%)	Weight Loss (%)
1	30	0.00	0.00	0.00
2	30	0.02	0.04	0.03
6	30	0.29	0.34	0.31
7	30	0.33	0.39	0.36
12	26	0.44	0.50	0.47
14	38	0.46	0.53	0.49
19	25	0.53	0.59	0.56
23	30	0.56	0.63	0.59
27	30	0.58	0.66	0.62
30	22	0.61	0.68	0.65
35	50	0.62	0.70	0.66
41	49	0.66	0.74	0.70
47	34	0.70	0.77	0.73
64	33	0.78	0.85	0.81
71	48	0.78	0.85	0.81
82	33	0.80	0.87	0.83
103	25	0.85	0.92	0.88
164	60	0.75	0.83	0.79
233	62	0.76	0.85	0.80
414	44	1.15	1.22	1.17
597	78	1.04	1.11	1.06

Age of Concrete after Casting (days)	Relative Humidity (%)	14SP4	14SP5	14SP6
		Weight Loss (%)	Weight Loss (%)	Weight Loss (%)
7	30	0.00	0.00	0.00
8	30	0.22	0.18	0.20
9	30	0.27	0.24	0.25
12	26	0.44	0.41	0.41
14	38	0.49	0.46	0.47
16	22	0.54	0.51	0.52
19	25	0.59	0.55	0.56
23	30	0.63	0.60	0.61
27	30	0.67	0.64	0.65
30	22	0.70	0.67	0.68
35	50	0.72	0.69	0.70
41	49	0.77	0.74	0.75
47	34	0.80	0.78	0.78
64	33	0.88	0.86	0.86
71	48	0.88	0.86	0.86
82	33	0.90	0.87	0.88
103	25	0.95	0.92	0.92
164	60	0.89	0.87	0.87
233	62	0.93	0.91	0.90
414	44	1.25	1.22	1.22
597	78	1.17	1.09	1.14

Age of Concrete after Casting (days)	Relative Humidity (%)	18SP1	18SP2	18SP3
		Weight Loss (%)	Weight Loss (%)	Weight Loss (%)
1	32	0.00	0.00	0.00
2	40	0.14	0.11	0.13
3	28	0.24	0.19	0.22
6	48	0.34	0.28	0.32
8	40	0.39	0.32	0.37
12	51	0.47	0.39	0.44
17	33	0.53	0.44	0.50
21	62	0.54	0.45	0.51
38	25	0.71	0.62	0.68
99	60	0.71	0.63	0.69
168	62	0.76	0.67	0.74
349	44	1.16	1.10	1.15
532	78	1.11	1.05	1.01

Age of Concrete after Casting (days)	Relative Humidity (%)	18SP4	18SP5	18SP6
		Weight Loss (%)	Weight Loss (%)	Weight Loss (%)
7	49	0.00	0.00	0.00
8	40	0.24	0.22	0.24
9	46	0.34	0.31	0.34
10	56	0.40	0.38	0.42
12	51	0.51	0.48	0.54
17	33	0.64	0.61	0.68
21	62	0.69	0.65	0.72
38	25	0.90	0.86	0.94
99	60	0.98	0.94	1.02
168	62	1.04	1.00	1.08
349	44	1.38	1.34	1.44
532	78	1.37	1.31	1.41

APPENDIX H – REGRESSION ANALYSIS FOR RECTANGULAR STRESS BLOCK PARAMETERS AND ULTIMATE COMPRESSIVE STRAIN OF CONCRETE

Suppose the rectangular stress block parameter, α_1 , can be linearly explained in terms of the concrete compressive strength, f'_c , as follows:

$$\alpha_1 = a f'_c + b \quad \text{Equation H-1}$$

where a is the slope and b is the intercept of the linear relationship. Thus each test result can be given as:

$$(\alpha_1)_i = a(f'_c)_i + b + \varepsilon_i \quad \text{Equation H-2}$$

where ε_i is the error for each data point and N is the number of data points presented in the graph. Least square fitting method requires defining the slope and intercepting, a and b , so that the sum of the squares of the differences between the data points and the regression line is minimum such as:

$$\sum_{i=1}^N \varepsilon_i^2 = \sum_{i=1}^N ((\alpha_1)_i - a(f'_c)_i - b)^2 \rightarrow \text{Minimum} \quad \text{Equation H-3}$$

thus the derivative of the sum of the squares of the differences with respect to a and b should be equal to zero as shown:

$$\left. \frac{\partial}{\partial a} \sum_{i=1}^N \varepsilon_i^2 \right|_{a,b} = -2 \sum_{i=1}^N ((\alpha_1)_i - a(f'_c)_i - b)(f'_c)_i = 0 \quad \text{Equation H-4}$$

and

$$\left. \frac{\partial}{\partial b} \sum_{i=1}^N \varepsilon_i^2 \right|_{a,b} = -2 \sum_{i=1}^N ((\alpha_1)_i - a(f'_c)_i - b) = 0. \quad \text{Equation H-5}$$

These equations can be simplified as:

$$Nb + a \sum_{i=1}^N (f'_c)_i = \sum_{i=1}^N (\alpha_1)_i \quad \text{Equation H-6}$$

$$b \sum_{i=1}^N (f'_c)_i + a \sum_{i=1}^N ((f'_c)_i)^2 = \sum_{i=1}^N (f'_c)_i (\alpha_1)_i \quad \text{Equation H-7}$$

and a and b can be solved as:

$$a = \frac{\sum_{i=1}^N (f'_c)_i (\alpha_1)_i - \frac{1}{N} \left(\sum_{i=1}^N (\alpha_1)_i \right) \left(\sum_{i=1}^N (f'_c)_i \right)}{\sum_{i=1}^N ((f'_c)_i)^2 - \frac{1}{N} \left(\sum_{i=1}^N (f'_c)_i \right)^2} = \frac{\sum_{i=1}^N ((f'_c)_i - \bar{f}'_c)^2}{\sum_{i=1}^N (\alpha_1)_i ((f'_c)_i - \bar{f}'_c)} \quad \text{Equation H-8}$$

$$b = \bar{\alpha}_1 - a \bar{f}'_c \quad \text{Equation H-9}$$

where

$$\bar{\alpha}_1 = \frac{1}{N} \sum_{i=1}^N (\alpha_1)_i \quad \text{Equation H-10}$$

$$\bar{f}'_c = \frac{1}{N} \sum_{i=1}^N (f'_c)_i. \quad \text{Equation H-11}$$

Similar procedure is used to obtain the linear regression relationship for the other rectangular stress block parameter, β_1 . The resulting equations can be given by:

$$\beta_1 = c f'_c + d \quad \text{Equation H-12}$$

where c is the slope and d is the intercept of the linear relationship. These variables can be calculated as:

$$c = \frac{\sum_{i=1}^N ((f'_c)_i - \bar{f}'_c)^2}{\sum_{i=1}^N (\beta_1)_i ((f'_c)_i - \bar{f}'_c)} \quad \text{Equation H-13}$$

$$d = \bar{\beta}_1 - c \bar{f}'_c \quad \text{Equation H-14}$$

where N is the number of data points presented in the graph and

$$\bar{\beta}_1 = \frac{1}{N} \sum_{i=1}^N (\beta_1)_i \quad \text{Equation H-15}$$

$$\bar{f}'_c = \frac{1}{N} \sum_{i=1}^N (f'_c)_i. \quad \text{Equation H-16}$$

The regression line is the median of the data points in each graph which means 50 percent of the data points lie above and below this line. The combination of the test results in the literature and this research was assumed to be normally distributed along the regression line as presented in Figure H-1.

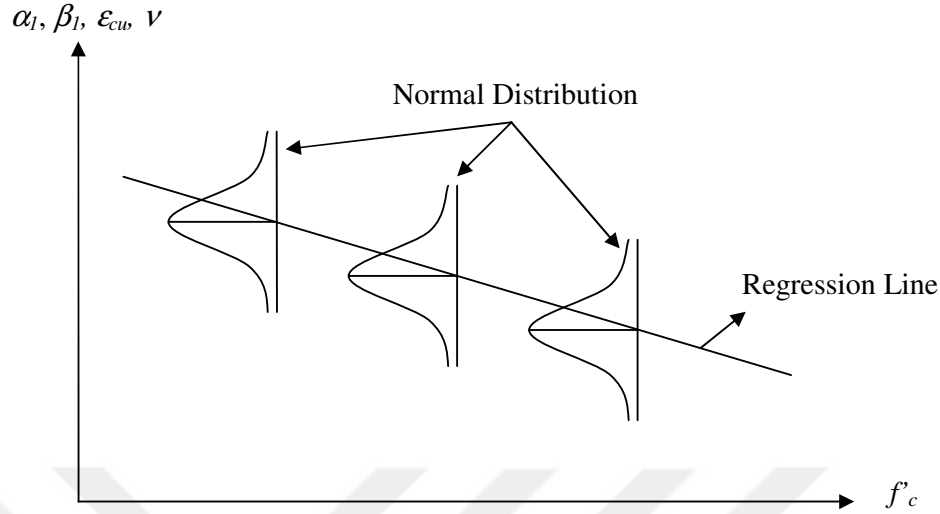


Figure H-1 – Normally Distributed Data along the Regression Line

The standard deviation of α_l for the data points with respect to the regression line is calculated as:

$$\sigma_{\alpha_l} = \sqrt{\frac{1}{N} \sum_{i=1}^N \left((\alpha_l)_i - (\bar{\alpha}_l)_{(f'_c)_i} \right)^2} . \quad \text{Equation H-17}$$

Similarly equations can be derived for β_l as follows:

$$\sigma_{\beta_l} = \sqrt{\frac{1}{N} \sum_{i=1}^N \left((\beta_l)_i - (\bar{\beta}_l)_{(f'_c)_i} \right)^2} . \quad \text{Equation H-18}$$

Using the same method, the ultimate compressive strain of concrete (ϵ_{cu}) can also be modeled with simple linear regression analysis as follows:

$$\epsilon_{cu} = e f'_c + g \quad \text{Equation H-19}$$

where e is the slope and g is the intercept of the linear relationship. These variables can be calculated as:

$$e = \frac{\sum_{i=1}^N ((f'_c)_i - \bar{f}'_c)^2}{\sum_{i=1}^N (\epsilon_{cu})_i ((f'_c)_i - \bar{f}'_c)} \quad \text{Equation H-20}$$

$$g = \bar{\epsilon}_{cu} - e \bar{f}'_c \quad \text{Equation H-21}$$

where N is the number of data points presented in the graph and

$$\bar{\epsilon}_{cu} = \frac{1}{N} \sum_{i=1}^N (\epsilon_{cu})_i \quad \text{Equation H-22}$$

$$\bar{f}'_c = \frac{1}{N} \sum_{i=1}^N (f'_c)_i . \quad \text{Equation H-23}$$

The combination of the test results in the literature and this research was assumed to be normally distributed along the regression line as presented in Figure H-1. The standard deviation (σ_v) of v for the data points with respect to the regression line is calculated as:

$$\sigma_{\varepsilon_{cu}} = \sqrt{\frac{1}{N} \sum_{i=1}^N \left((\varepsilon_{cu})_i - (\bar{\varepsilon}_{cu})_{(f'_c)_i} \right)^2} . \quad \text{Equation H-24}$$

APPENDIX I – SENSITIVITY ANALYSIS FOR RECTANGULAR STRESS BLOCK PARAMETERS AND ULTIMATE COMPRESSIVE STRAIN OF CONCRETE

In all the calculations, Grade 60 longitudinal steel reinforcement is used with 60 ksi yield strength and 30000 ksi elastic modulus.

I1 Flexural Under-Reinforced Concrete Members

The cross-section of the flexural reinforced concrete member is presented in Figure I-1.

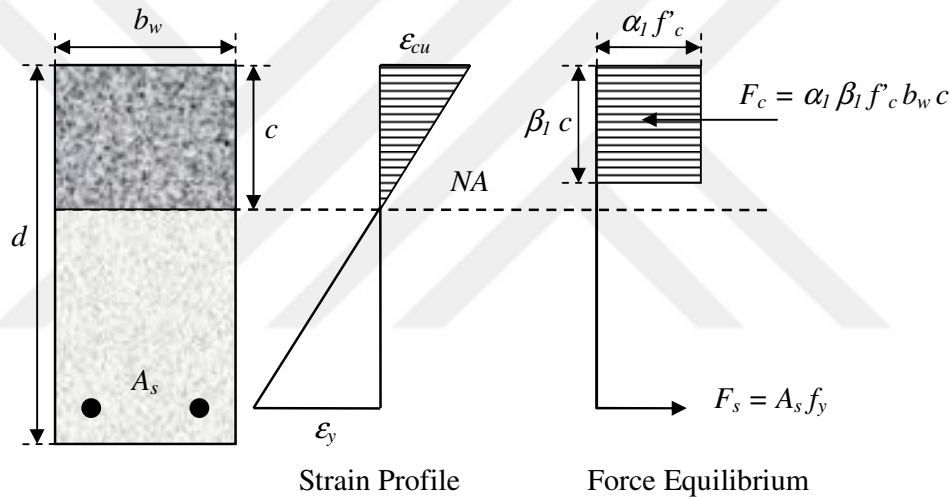


Figure I-1 – Cross-Section of a Flexural Reinforced Concrete Member

The force resisted by concrete (F_c) can be derived as:

$$F_c = \alpha_1 \beta_1 f'_c b_w c \quad \text{Equation I-1}$$

where α_1 and β_1 are the rectangular stress block parameters, f'_c is specified concrete compressive strength, c is the distance from the extreme compression fiber to the neutral axis (depth of the neutral axis) and b_w is the width of the web. Similarly, the force resisted by longitudinal reinforcement (F_s) can be written as:

$$F_s = A_s f_y \quad \text{Equation I-2}$$

where A_s is the cross-sectional area and f_y is the tensile strength of longitudinal reinforcement. The equilibrium of forces requires that the forces resisted by concrete and steel should be equal, such as:

$$\alpha_1 \beta_1 f'_c b_w c = A_s f_y. \quad \text{Equation I-3}$$

Furthermore, the distance from the extreme compression fiber to the neutral axis can be derived as:

$$c = \frac{A_s f_y}{\alpha_1 \beta_1 f'_c b_w}. \quad \text{Equation I-4}$$

The ultimate moment capacity (M_u) of the cross-section can be expressed as:

$$M_u = F_c \left(d - \frac{\beta_1 c}{2} \right) = F_s \left(d - \frac{\beta_1 c}{2} \right) \quad \text{Equation I-5}$$

where d is the distance from the extreme compression fiber to centroid of longitudinal reinforcement.

11.1 Change in Rectangular Stress Block Parameter, α_1

Assume a flexural under-reinforced concrete section with a c/d ratio equals to 0.375 which is the maximum reinforcement limit for flexural members specified by AASHTO LRFD Bridge Design Specifications (2004). This also represents a case with a reinforcement ratio equals to 0.625 times the balanced reinforcement ratio where, c/d ratio is equal to 0.60 for a balanced section. When the rectangular stress block parameter, α_1 , is varied from α_{11} equal to 0.85 (Case 1) to α_{12} equal to 0.75 (Case 2), (11.8 percent decrease), the intensity of the rectangular stress block decreases. Since the force provided by longitudinal reinforcement is constant for under-reinforced sections, equilibrium of forces requires the compensation of this reduction by an increase in the depth of the neutral axis. This increase can be calculated as follows:

$$c_1 = \frac{A_s f_y}{\alpha_{11} \beta_1 f'_c b_w} = \frac{A_s f_y}{0.85 \beta_1 f'_c b_w} \quad \text{for Case 1 } (\alpha_{11} = 0.85) \quad \text{Equation I-6}$$

and

$$c_2 = \frac{A_s f_y}{\alpha_{12} \beta_1 f'_c b_w} = \frac{A_s f_y}{0.75 \beta_1 f'_c b_w} \quad \text{for Case 2 } (\alpha_{12} = 0.75) \quad \text{Equation I-7}$$

where

$$\% \text{ increase} = \frac{c_2 - c_1}{c_1} \% = 13.3\% \quad \text{Equation I-8}$$

yielding

$$c_2 = 1.133 c_1. \quad \text{Equation I-9}$$

This 13.3 percent increase in the depth of the neutral axis results in a reduction in the ultimate moment capacity as follows:

$$M_{u1} = A_s f_y \left(d - \frac{\beta_1 c_1}{2} \right) \text{ for Case 1 } (\alpha_{l1} = 0.85) \quad \text{Equation I-10}$$

$$M_{u2} = A_s f_y \left(d - \frac{\beta_1 c_2}{2} \right) \text{ for Case 2 } (\alpha_{l2} = 0.75) \quad \text{Equation I-11}$$

where

$$d = \frac{c_1}{0.375} \quad \text{Equation I-12}$$

and

$$\frac{M_{u1}}{M_{u2}} = \frac{A_s f_y \left(\frac{c_1}{0.375} - \frac{\beta_1 c_1}{2} \right)}{A_s f_y \left(\frac{c_1}{0.375} - \frac{\beta_1 c_2}{2} \right)} = \frac{\frac{c_1}{0.375} - \frac{\beta_1 c_1}{2}}{\frac{c_1}{0.375} - \frac{\beta_1 c_2}{2}} = \frac{\frac{\epsilon_1}{0.375} - \frac{\beta_1 \epsilon_1}{2}}{\frac{\epsilon_1}{0.375} - \frac{\beta_1 (1.308 \epsilon_1)}{2}} = \frac{\frac{1}{0.375} - \frac{\beta_1}{2}}{\frac{1}{0.375} - \frac{1.133 \beta_1}{2}} \quad \text{Equation I-13}$$

yielding

$$\frac{M_{u1}}{M_{u2}} = 1.019. \quad \text{Equation I-14}$$

Note that the ultimate moment capacities are calculated based on concrete compressive strengths over 8 ksi corresponding to the stress block parameter β_1 equal to 0.65.

It can be concluded that, at the maximum reinforcement limit ($c/d = 0.375$) for flexural members, the reduction of the rectangular stress block parameter, α_l , by 11.8 percent leads to a reduction of the ultimate moment capacity by 1.9 percent. Note that, identical sections with identical amount of longitudinal reinforcement ratios are compared to obtain these results. The amount of longitudinal reinforcement ratio is calculated based on Case 1 ($c_1/d = 0.375$). Furthermore, if the amount of longitudinal reinforcement was calculated based on Case 2 ($c_2/d = 0.375$), the reduction in the ultimate moment capacity would be less ($M_{u1}/M_{u2} = 1.016$, 1.6 percent). These results also implies that concrete compressive strength, f'_c , does not affect the change in the ultimate moment capacity when the rectangular stress block parameter, α_l , is varied unless there is a relationship between f'_c and α_l .

The results indicate that the change in the rectangular stress block parameter, α_l , does not significantly affect the ultimate moment capacity of the flexural under-reinforced concrete members.

I1.2 Change in Rectangular Stress Block Parameter, β_1

Substituting F_s and c in the ultimate moment capacity equation (Equation I5) gives:

$$M_u = A_s f_y \left(d - \frac{A_s f_y}{2 \alpha_1 f'_c b_w} \right). \quad \text{Equation I-15}$$

The resulting equation is independent of rectangular stress block parameter, β_1 , which implies that the change in this parameter has no effect on the ultimate moment capacity of the flexural reinforced concrete member. This change is fully compensated by the change in the distance from the extreme compression fiber to the neutral axis. As β_1 increases, c decreases and vice versa.

I1.3 Change in Ultimate Compressive Strain of Concrete, ϵ_{cu}

The ultimate compressive strain, ϵ_{cu} , for a concrete section is used to predict the yielding of the longitudinal reinforcement. However for an under-reinforced concrete member, the longitudinal reinforcement has already yielded which implies that the ultimate compressive strain of concrete, ϵ_{cu} , has no effect on the ultimate moment capacity of a flexural under-reinforced concrete member.

I2 Flexural Over-Reinforced Concrete Members

The cross-section of the flexural over-reinforced concrete member is presented in Figure I-2.

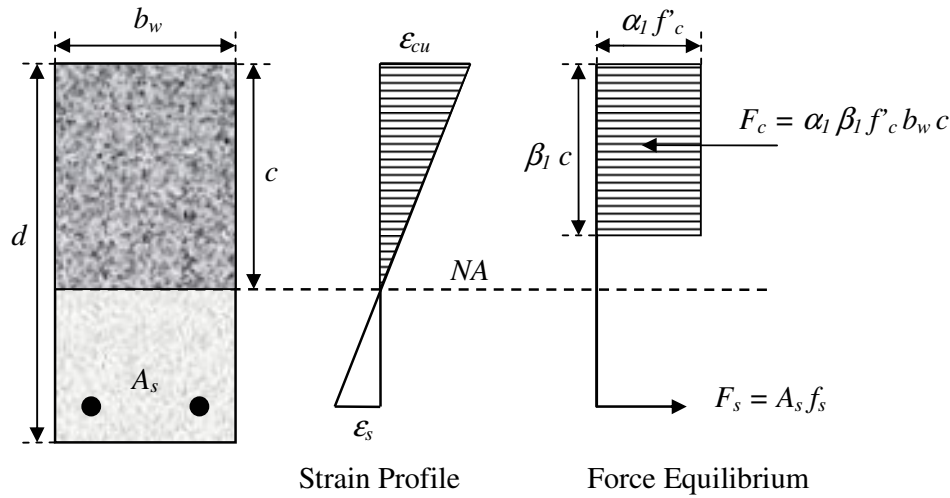


Figure I-2 – Cross-Section of a Flexural Over-Reinforced Concrete Member

The force resisted by concrete (F_c) can be derived as:

$$F_c = \alpha_1 \beta_1 f'_c b_w c \quad \text{Equation I-16}$$

where α_l and β_l are the rectangular stress block parameters, f'_c is specified compressive strength of concrete, c is the distance from the extreme compression fiber to the neutral axis and b_w is the width of the web. Similarly, the force resisted by longitudinal reinforcement (F_s) can be written as:

$$F_s = A_s f_s \quad \text{Equation I-17}$$

where A_s is the cross-sectional area of longitudinal reinforcement and f_s is the stress in longitudinal reinforcement which can be calculated as:

$$f_s = E_s \epsilon_s = E_s \epsilon_{cu} \left(\frac{d-c}{c} \right) \quad \text{Equation I-18}$$

where E_s is the elastic modulus of longitudinal reinforcement, ϵ_s is the strain in longitudinal reinforcement, ϵ_{cu} is the ultimate compressive strain of concrete and d is the distance from the extreme compression fiber to centroid of longitudinal reinforcement. The equilibrium of forces requires that the forces resisted by concrete and steel should be equal, such as:

$$\alpha_l \beta_l f'_c b_w c = A_s E_s \epsilon_{cu} \left(\frac{d-c}{c} \right). \quad \text{Equation I-19}$$

The sensitivity study of the flexural over-reinforced concrete sections was based on c/d ratio equals to 0.75 which is twice the maximum reinforcement limit for flexural members specified by AASHTO LRFD Bridge Design Specifications (2004). This represents an extreme case with a reinforcement ratio equals to 2.5 times the balanced section reinforcement ratio where, c/d is equal to 0.60 for balanced section.

I2.1 Change in Rectangular Stress Block Parameter, α_l

When the rectangular stress block parameter, α_l , is varied from α_{l1} equal to 0.85 (Case 1) to α_{l2} equal to 0.75 (Case 2), (11.8 percent decrease), the intensity of the rectangular stress block decreases. The new equilibrium of forces requires the compensation of the reduction by an increase in the depth of the neutral axis. This increase can be calculated as follows:

$$0.85 \beta_l f'_c b_w c_1 = A_s E_s \epsilon_{cu} \left(\frac{d-c_1}{c_1} \right) \text{ for Case 1 } (\alpha_{l1} = 0.85) \quad \text{Equation I-20}$$

and

$$0.75 \beta_l f'_c b_w c_2 = A_s E_s \epsilon_{cu} \left(\frac{d-c_2}{c_2} \right) \text{ for Case 2 } (\alpha_{l2} = 0.75) \quad \text{Equation I-21}$$

where

$$d = \frac{c_1}{0.75} . \quad \text{Equation I-22}$$

The solution for the depth of the neutral axis can be found from:

$$0.85\beta_1 f'_c b_w (c_1)^2 + A_s E_s \epsilon_{cu} c_1 - A_s E_s \epsilon_{cu} d = 0 \text{ for Case 1 } (\alpha_{I1} = 0.85). \quad \text{Equation I-23}$$

When d is substituted in the equation, the depth of the neutral axis for Case 1 can be expressed as:

$$c_1 = \frac{A_s E_s \epsilon_{cu}}{2.55\beta_1 f'_c b_w} \text{ for Case 1 } (\alpha_{I1} = 0.85) \quad \text{Equation I-24}$$

Similarly the solution for the depth of the neutral axis for Case 2 can be found from:

$$0.75\beta_1 f'_c b_w (c_2)^2 + A_s E_s \epsilon_{cu} c_2 - A_s E_s \epsilon_{cu} \frac{c_1}{0.75} = 0 \text{ for Case 2 } (\alpha_{I2} = 0.75) \quad \text{Equation I-25}$$

When these equations are solved parametrically, the increase in the depth of the neutral axis can be calculated as follows:

$$\% \text{ increase} = \frac{c_2 - c_1}{c_1} \% = 2.5\% \quad \text{Equation I-26}$$

yielding,

$$c_2 = 1.025c_1 . \quad \text{Equation I-27}$$

This 2.5 percent increase in the depth of the neutral axis results in a reduction in the ultimate moment capacity as follows:

$$M_{u1} = A_s E_s \epsilon_{cu} \left(\frac{d - c_1}{c_1} \right) \left(d - \frac{\beta_1 c_1}{2} \right) \text{ for Case 1 } (\alpha_{I1} = 0.85) \quad \text{Equation I-28}$$

$$M_{u2} = A_s E_s \epsilon_{cu} \left(\frac{d - c_2}{c_2} \right) \left(d - \frac{\beta_1 c_2}{2} \right) \text{ for Case 2 } (\alpha_{I2} = 0.65) \quad \text{Equation I-29}$$

where

$$d = \frac{c_1}{0.75} \quad \text{Equation I-30}$$

yielding

$$\frac{M_{u1}}{M_{u2}} = 1.103 . \quad \text{Equation I-31}$$

Note that the ultimate moment capacities are calculated based on the stress block parameter, β_1 equal to 0.65.

It can be concluded that, at 2 times the maximum reinforcement limit ($c/d = 0.75$) for flexural members, the reduction of the rectangular stress block parameter, α_1 , by 11.8 percent leads to a reduction of the ultimate moment capacity by 10.3 percent. Note that, identical sections with identical amount of longitudinal reinforcement ratios are compared to obtain these results. The amount of longitudinal reinforcement ratio is calculated based on Case 1 ($c_1/d = 0.75$). Furthermore, if the amount of longitudinal reinforcement was calculated based on Case 2 ($c_2/d = 0.75$), the reduction in the ultimate moment capacity would be less ($M_{u1}/M_{u2} = 1.102$, 10.2 percent). The results also implies that concrete compressive strength, f'_c , does not affect the change in the ultimate moment capacity when the rectangular stress block parameter, α_1 , is varied unless there is a relationship between f'_c and α_1 .

The results indicate that the change in the rectangular stress block parameter, α_1 , significantly affects the ultimate moment capacity of the flexural over-reinforced concrete members.

I2.2 Change in Rectangular Stress Block Parameter, β_1

When the rectangular stress block parameter, β_1 , is varied from β_{11} equal to 0.85 (Case 1) to β_{12} equal to 0.65 (Case 2), (23.5 percent decrease), the height of the rectangular stress block decreases. The new equilibrium of forces requires the compensation of the reduction by an increase in the depth of the neutral axis. This increase can be calculated as follows:

$$0.85\alpha_1 f'_c b_w c_1 = A_s E_s \epsilon_{cu} \left(\frac{d - c_1}{c_1} \right) \text{ for Case 1 } (\beta_{11} = 0.85) \quad \text{Equation I-32}$$

and

$$0.65\alpha_1 f'_c b_w c_2 = A_s E_s \epsilon_{cu} \left(\frac{d - c_2}{c_2} \right) \text{ for Case 2 } (\beta_{12} = 0.65) \quad \text{Equation I-33}$$

where

$$d = \frac{c_1}{0.75}. \quad \text{Equation I-34}$$

The solution for the depth of the neutral axis can be found from:

$$0.85\alpha_1 f'_c b_w (c_1)^2 + A_s E_s \epsilon_{cu} c_1 - A_s E_s \epsilon_{cu} d = 0 \text{ for Case 1 } (\beta_{11} = 0.85). \quad \text{Equation I-35}$$

When d is substituted in the equation, the depth of the neutral axis for Case 1 can be expressed as:

$$c_1 = \frac{A_s E_s \epsilon_{cu}}{2.55 \alpha_1 f'_c b_w} \text{ for Case 1 } (\beta_{11} = 0.85). \quad \text{Equation I-36}$$

Similarly the solution for the depth of the neutral axis for Case 2 can be found from:

$$0.65 \alpha_1 f'_c b_w (c_2)^2 + A_s E_s \epsilon_{cu} c_2 - A_s E_s \epsilon_{cu} \frac{c_1}{0.75} = 0 \text{ for Case 2 } (\beta_{12} = 0.65) \quad \text{Equation I-37}$$

When these equations are solved parametrically, the increase in the depth of the neutral axis can be calculated as follows:

$$\% \text{ increase} = \frac{c_2 - c_1}{c_1} \% = 5.2\% \quad \text{Equation I-38}$$

yielding,

$$c_2 = 1.052 c_1. \quad \text{Equation I-39}$$

This 5.2 percent increase in the depth of the neutral axis results in a reduction in the ultimate moment capacity as follows:

$$M_{u1} = A_s E_s \epsilon_{cu} \left(\frac{d - c_1}{c_1} \right) \left(d - \frac{0.85 c_1}{2} \right) \text{ for Case 1 } (\beta_{11} = 0.85) \quad \text{Equation I-40}$$

$$M_{u2} = A_s E_s \epsilon_{cu} \left(\frac{d - c_2}{c_2} \right) \left(d - \frac{0.65 c_2}{2} \right) \text{ for Case 2 } (\beta_{12} = 0.65) \quad \text{Equation I-41}$$

where

$$d = \frac{c_1}{0.375} \quad \text{Equation I-42}$$

yielding

$$\frac{M_{u1}}{M_{u2}} = 1.122. \quad \text{Equation I-43}$$

Note that the ultimate moment capacities are calculated based on the stress block parameter α_1 equal to 0.85.

It can be concluded that, at 2 times the maximum reinforcement limit ($c/d = 0.75$) for flexural members, the reduction of the rectangular stress block parameter, β_1 , by 23.5 percent leads to a reduction of the ultimate moment capacity by 12.2 percent. Note that, identical sections with identical amount of longitudinal reinforcement ratios are compared

to obtain these results. The amount of longitudinal reinforcement ratio is calculated based on Case 1 ($c_1/d = 0.75$). Furthermore, if the amount of longitudinal reinforcement was calculated based on Case 2 ($c_2/d = 0.75$), the reduction in the ultimate moment capacity would be slightly higher ($M_{u1}/M_{u2} = 1.124$, 12.4 percent). The results also implies that concrete compressive strength, f'_c , does not affect the change in the ultimate moment capacity when the rectangular stress block parameter, β_1 , is varied unless there is a relationship between f'_c and α_1 .

The results indicate that the change in the rectangular stress block parameter, β_1 , significantly affects the ultimate moment capacity of the flexural over-reinforced concrete members.

12.3 Change in Ultimate Compressive Strain of Concrete, ϵ_{cu}

When the ultimate compressive strain of concrete, ϵ_{cu} , is varied from ϵ_{cu1} equal to 0.003 (Case 1) to ϵ_{cu2} equal to 0.0025 (Case 2), (16.7 percent decrease), the tensile strain of longitudinal reinforcement decreases. Consequently, the new equilibrium of forces requires a reduction in the depth of the neutral axis. This reduction can be calculated as follows:

$$\alpha_1 \beta_1 f'_c b_w c_1 = A_s E_s 0.003 \left(\frac{d - c_1}{c_1} \right) \text{ for Case 1 } (\epsilon_{cu1} = 0.003) \quad \text{Equation I-44}$$

and

$$\alpha_1 \beta_1 f'_c b_w c_2 = A_s E_s 0.0025 \left(\frac{d - c_2}{c_2} \right) \text{ for Case 2 } (\epsilon_{cu2} = 0.0025) \quad \text{Equation I-45}$$

where

$$d = \frac{c_1}{0.75}. \quad \text{Equation I-46}$$

The solution for the depth of the neutral axis can be found from:

$$0.85 \beta_1 f'_c b_w (c_1)^2 + A_s E_s 0.003 c_1 - A_s E_s 0.003 d = 0 \text{ for Case 1 } (\epsilon_{cu1} = 0.003). \quad \text{Equation I-47}$$

When d is substituted in the equation, the depth of the neutral axis for Case 1 can be expressed as:

$$c_1 = \frac{A_s E_s 0.003}{3 \alpha_1 \beta_1 f'_c b_w} \text{ for Case 1 } (\epsilon_{cu1} = 0.003). \quad \text{Equation I-48}$$

Similarly the solution for the depth of the neutral axis for Case 2 can be found from:

$$\alpha_1 \beta_1 f'_c b_w (c_2)^2 + A_s E_s 0.0025 c_2 - A_s E_s 0.0025 \frac{c_1}{0.75} = 0 \text{ for Case 2 } (\epsilon_{cu2} = 0.003).$$

Equation I-49

When these equations are solved parametrically, the increase in the depth of the neutral axis can be calculated as follows:

$$\% \text{ reduction} = \frac{c_1 - c_2}{c_1} \% = 3.7\%$$

Equation I-50

yielding,

$$c_1 = 1.037 c_2.$$

Equation I-51

The 3.7 percent reduction in the depth of the neutral axis results in a reduction in the ultimate moment capacity as follows:

$$M_{u1} = A_s E_s 0.003 \left(\frac{d - c_1}{c_1} \right) \left(d - \frac{\beta_1 c_1}{2} \right) \text{ for Case 1 } (\epsilon_{cu1} = 0.003)$$

Equation I-52

$$M_{u2} = A_s E_s 0.0025 \left(\frac{d - c_2}{c_2} \right) \left(d - \frac{\beta_1 c_2}{2} \right) \text{ for Case 2 } (\epsilon_{cu2} = 0.0025)$$

Equation I-53

where

$$d = \frac{c_1}{0.75}$$

Equation I-54

yielding,

$$\frac{M_{u1}}{M_{u2}} = 1.026.$$

Equation I-55

Note that the ultimate moment capacities are calculated based on the stress block parameters, α_1 equal to 0.85 and β_1 equal to 0.65.

It can be concluded that, at 2 times the maximum reinforcement limit ($c/d = 0.75$) for flexural members, the reduction of the ultimate compressive strain of concrete, ϵ_{cu} , by 16.7 percent leads to a reduction of the ultimate moment capacity by 2.6 percent. Note that, identical sections with identical amount of longitudinal reinforcement ratios are compared to obtain these results. The amount of longitudinal reinforcement ratio is calculated based on Case 1 ($c_1/d = 0.75$). Furthermore, if the amount of longitudinal reinforcement was calculated based on Case 2 ($c_2/d = 0.75$), the reduction in the ultimate moment capacity would be slightly higher ($M_{u1}/M_{u2} = 1.027$, 2.7 percent). The results also implies that concrete compressive strength, f'_c , does not affect the change in the

ultimate moment capacity when the ultimate compressive strain of concrete, ϵ_{cu} , is varied unless there is a relationship between f'_c and ϵ_{cu} .

The results indicate that the change in the ultimate compressive strain of concrete, ϵ_{cu} , does not significantly affect the ultimate moment capacity of the flexural over-reinforced concrete members.



APPENDIX J – REGRESSION ANALYSIS FOR POISSON’S RATIO

The resulting equations can be given by:

$$\nu = hf'_c + j \quad \text{Equation J-1}$$

where h is the slope and j is the intercept of the linear relationship. These variables can be calculated as:

$$h = \frac{\sum_{i=1}^N ((f'_c)_i - \bar{f}'_c)^2}{\sum_{i=1}^N (\nu)_i ((f'_c)_i - \bar{f}'_c)} \quad \text{Equation J-2}$$

$$j = \bar{\nu} - h\bar{f}'_c \quad \text{Equation J-3}$$

where N is the number of data points presented in the graph and

$$\bar{\nu} = \frac{1}{N} \sum_{i=1}^N (\nu)_i \quad \text{Equation J-4}$$

$$\bar{f}'_c = \frac{1}{N} \sum_{i=1}^N (f'_c)_i. \quad \text{Equation J-5}$$

The combination of the test results in the literature and this research was assumed to be normally distributed along the regression line as presented in Figure H1. The standard deviation (σ_ν) of ν for the data points with respect to the regression line is calculated as:

$$\sigma_\nu = \sqrt{\frac{1}{N} \sum_{i=1}^N \left((\nu)_i - (\bar{\nu})_{|(f'_c)_i} \right)^2}. \quad \text{Equation J-6}$$