ABSTRACT

Wood, Bryan Thane. Use of Slurry Infiltrated Fiber Concrete in Hinge Regions of Earthquake Resistant Structures. (Under the direction of Dr. John Hanson, Distinguished Emeritus Professor of Civil Engineering)

This dissertation reports on an experimental and analytical study of the use of precast slurry infiltrated fiber concrete (SIFCON) flexural hinges to improve the seismic resistance of reinforced concrete moment frames. The main thrust of the research was to investigate how different variables effect the nonlinear, cyclic, flexural behavior of reinforced SIFCON hinges, and to determine how to optimize hinge performance. In addition, a conceptual analysis was performed to evaluate the improvement in seismic resistance from using SIFCON hinges in concrete structures.

Seven 10” wide, 16” deep, and 26” long reinforced SIFCON hinges were designed and fabricated, then tested under quasi-static loading. All specimens were fabricated using between 9 and 11%, by volume, Dramix 30/50 fibers, made by the Bekaert Corporation. Grade 60, Grade 75, and ASTM A722 (Dywidag) bars were used, in combination with three different SIFCON compression strengths. Additionally, various end connection details were used in testing three different reinforcing arrangements.

It was shown that precast SIFCON hinges can exhibit better performance than reinforced concrete hinges. The maximum curvature ductility achieved was 26.4 over a 4” inch long
interior region of a specimen. The curvature ductility of this hinge specimen, when taken over the full 26 inch hinge length, was 10.5. SIFCON hinges absorb approximately 30% more energy than fiber-reinforced concrete hinges. SIFCON hinge ductility is limited by the ultimate tensile strain of the reinforcing steel. Grade 60 reinforcing resulted in the best hinge behavior seen in testing. Transverse ties may be required to prevent buckling of compression reinforcing. SIFCON flexural stiffness is approximately half that of comparable strength reinforced concrete beams.

It was found that SIFCON material behavior is highly variable. Fiber orientation and size effects are the main variables that affect SIFCON behavior. Fabrication technique and skill of workmanship greatly affects fiber orientation, while size effects make it difficult to predict insitu SIFCON properties. State-of-the-art models are not accurate enough to facilitate using SIFCON hinges to build more earthquake resistant structures. With present models, the weakest region of a beam may actually be the strongest region.
Use of Slurry Infiltrated Fiber Concrete (SIFCON) in Hinge Regions for Earthquake Resistant Structures

by

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A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the Degree of Doctor of Philosophy

Civil Engineering

Raleigh
2000

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PERSONAL BIOGRAPHY

Mr. Bryan Wood was born within one mile, and raised within 10 miles, of the University of Alabama campus in Tuscaloosa, Alabama. His formative years were during the pinnacle of Coach Paul Bear Bryant’s coaching career. He was raised in a middle class family of seven in the small town of Duncanville Alabama. The small town values learned as a youth remain a major part of his character today.

Two traits have influenced Bryan’s life. He is always curious in nature. It was never enough to know that something worked. He had to know why it worked. The second trait that influenced his life is that he liked building and creating. Whether it be a simple wood working project, or a complicated computer code, simple pleasure was taken from the fact that he was creating something from nothing.

Bryan attended the University of Alabama, where he earned a bachelor’s degree and master’s degree in civil engineering in 1987 and 1989, respectively. After graduation, he accepted a position at BE&K Engineering and Construction in Birmingham, Alabama. It was there that he earned his professional engineer’s license. To once again satisfy the question “what if”, Bryan chose to pursue his Ph.D. degree in structural engineering at North Carolina State University. After attending North Carolina State University for two years, he changed his major and spent the next three years finishing this thesis. Let’s hope that the “what ifs” will never end.
ACKNOWLEDGEMENTS

The author would like to thank Dr. John Hanson for his patience and guidance throughout this project. His attention to detail and ability to isolate important issues and concepts proved invaluable in producing quality, meaningful research. The author would also like to thank Dr. James Nau for his support, not only with the research, but also as a friend. Dr. Neven Krstulovic was a valuable sounding board for brainstorming innovative ideas associated with this research. His never-ending thirst for trying something new was refreshing.

Many fellow graduate students assisted in specimen fabrication throughout this research. I just hope that I was as much assistance to them as they were to me. Those students include Erdem Dogan, Patrick Theideman, David Hawkins, Jeff Morrison, Isa Mavi, Greg Ellen, Star Longo, and Chris Ply. A special thanks goes to Dr. Satrajit Das for his assistance in using Ruaumoko to model the prototype structure.

The author would like to thank Dayton Richmond Fasteners, Bekaert Corporation, and Headed Reinforcement Corporation for providing complimentary material used in this project. He also thanks Mr. Owen Cordell and The North Carolina Department of Transportation for providing material testing support for this project.

The support and kindness received from all the administrative assistants and lab technicians within the Civil Engineering Department must be acknowledged. The kindness and friendship of Mrs. Edna White (retired CE graduate department administrative assistant) and the assistance of Mr. Jerry Atkinson (CFL lab technician) will never be forgotten.

Finally, I would like to thank my parents. My father always said that nobody can do it for you.
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Seismic design forces recommended in modern codes are significantly less than the inertial forces induced if a structure responds in the elastic range throughout a major earthquake [UBC, 1994]. However if the structure deforms beyond its elastic range, the inertial forces will be reduced. As a result, codes allow design forces to be reduced below elastic response levels. The magnitude of the reduction depends upon how far a structure can deform beyond its elastic range. The farther a structure can deform past its elastic range, the lower the design forces for that structure.

The ability of a structure to deform beyond its elastic range is referred to as ductility. Ductility is quantified by either a displacement ductility factor, $\mu (\mu = \Delta_{\text{max}} / \Delta_y)$, or a curvature ductility factor, $\mu_\phi (\mu_\phi = \phi_{\text{max}} / \phi_y)$. ($\Delta_{\text{max}}$ and $\phi_{\text{max}}$ are the maximum displacement and curvature a structure or member can withstand. $\Delta_y$ and $\phi_y$ are the displacement and curvature at the yield point of a structure or member). The higher the ductility factor of a structure, the lower the recommended design forces for that structure.

In cast-in-place concrete construction, beam-column joints usually limit the structural ductility. Joint diagonal cracking leads to joint softening and degradation. In precast construction, connections usually limit the post elastic response by being the weak link in the system [Restrepo et. al, 1995; Stanton et. al, 1997; Priestley, 1997]. Thus, to improve the earthquake resistance of concrete moment frames, joint regions should be strengthened in cast-in-place construction and connections should be strengthened in precast construction. Although strengthening joints and connections is appealing, it has historically proven difficult. Limiting forces on the joints and connections may hold more promise.
Loads on connections and joints can be limited by providing flexural hinges with well-defined flexural strength adjacent to the beam column interface. Flexural hinges can limit the forces transferred to beam-column joints and precast connections. In addition to limiting loads, hinges are also the best method available for achieving ductile, post elastic, seismic deformations [Park, 1995].

Reinforced concrete hinging has been researched since the middle 1960’s [Mattock, 1965]. Flexural hinges are regions within members that undergo plastic rotations under loads equal to or greater than the yield loads. Much of the early researched was aimed at limit state analysis and inelastic moment redistribution. As such, the distribution of flexural yielding was of prime importance to limit state analysis. Only later did researchers recognize the importance of controlled flexural yielding in resisting earthquake forces.

Early research on using hinges to resist earthquake forces attempted to concentrate all of the inelastic action to a single cross-section. However, as research progressed on using hinges to reduce earthquake forces, it became evident that distributed flexural yielding resulted in better hinge behavior and better behavior of the overall structure. Researchers began designing specific regions within beam ends to yield at predetermined load levels. These regions often comprised much different details as the remainder of the beam. As a result, this entire, specifically designed region became know as the hinge region—even though flexural yielding did not develop within the full length of this region. Furthermore, the details used in these regions were so varied that the concepts developed by Mattock [1965] and others were often, not applicable.

This thesis will refer to the full length of the specifically designed region as the hinge region. There will be a length of distributed yielding within the hinges that will be referred to as the yield zone. Specifically designed hinge lengths are usually 1.5 to 2.5 times the section depth. This length was determined to be the minimum length for proper
reinforcing development. Longer hinge lengths have not been found to improve behavior.

No researchers have clearly stated why longer hinge lengths have not improved behavior. Mattock’s research has shown that it is unlikely that flexural yielding can be distributed over a length greater than 1.0 times the section depth. Thus, hinge lengths greater than 1.0 time the specimen depth will not improve the nonlinear behavior of the hinge.

The characteristics of reinforcing development are the greatest factors affecting this length. Stronger bond stresses lead to larger reinforcing stress gradients along the length of the bar. Thus, a shorter distance is required to reduce the reinforcing bar stress below its yield level. This results in smaller lengths of distributed yielding (smaller yield zone lengths). The opposite is true for weaker bond stresses. Weaker bond stresses result in longer yield zone lengths.

By placing properly designed hinges at beam ends, joint and connection loads can be limited by the moment and shear capacity that the hinge can transfer to the joints. This is defined as capacity-based-design and is required by the New Zealand building code [NZS 3101:1995]. The primary consideration for hinge research in this study is to limit the post-elastic forces on beam-column connections.

A secondary consideration in the design of hinges is energy absorption. All of the earthquake energy must be absorbed by the structure. The more energy a system can absorb, the lower the seismic loading and deformations. Thus, if the hinges absorb a high level of energy, loads induced on the remaining parts of the structure will be reduced [Restrepo et. al, 1995].

Reinforced concrete has performed adequately when used in hinges. However, shear slip on a through-depth flexural crack, buckling of compression reinforcement, inadequate compressive strain capacity, and inadequate shear strength have limited the performance
Recent developments in high performance, fiber reinforced concretes hold great potential for hinge zones in precast, or cast-in-place, concrete moment frames. Fiber reinforced concrete (FRC) is mixed and placed with up to 2% fibers. Steel, wood, carbon, and many types of glass are used as fibers. High performance, fiber reinforced concrete (HPFRC) contains between 2% to 30% fibers. Such high fiber volumes result in properties superior to that of fiber reinforced concrete. Slurry infiltrated fiber concrete (SIFCON) is one type of high performance, fiber reinforced concrete. As a result of its strength, ductility, and toughness, SIFCON shows the most promise for use in hinge regions.

SIFCON is similar to FRC in that it has discrete interlocking fibers that lend significant tensile properties to the composite matrix. In SIFCON, the fibers are preplaced inside the forms, and a cement rich slurry is poured or pumped into the forms. SIFCON specimens have been produced with fiber volume fractions ($V_f$) between 5% and 30% [Lankard, 1984; Homrich and Naaman, 1987; Naaman et. al., 1991]. Such high fiber volume fractions produce a tough, ductile material that exhibits minimal strength degradation and a high level of energy absorption.

It is expected that superior hinge behavior can be achieved using a SIFCON matrix. Compressive strengths up to 20 ksi and compressive strains over 10% without severe strength degradation have been reported in SIFCON specimens [Homrich and Naaman, 1987; Naaman et al., 1991; Naaman and Homrich, 1989]. Superior reinforcing bar development length [Hamza and Naaman, 1996], confinement, [Abdou, 1990], and toughness [Lankard, 1984; Naaman and Homrich, 1989; Naaman et al., 1987] indicate the potential of using SIFCON hinges in seismic resistant structures.
1.1 Research Scope and Objectives

The research described in this study is aimed at investigating the behavior of SIFCON hinge regions under inelastic cyclic loading. The loading is intended to simulate the flexural load conditions on girders in concrete moment frames exposed to severe earthquake accelerations. The program includes an experimental investigation of seven 2/3-scale hinges, an analytical evaluation of those hinges, and the application of the results to a full-scale prototype structure.

Seven SIFCON hinges were tested under quasi-static displacement loading in order to evaluate the critical parameters that control the response of SIFCON hinges. These parameters are varied in an effort to optimize hinge performance. Parameters studied include reinforcement geometry, reinforcement strength, reinforcement yield and ultimate strain capacity, and SIFCON matrix strength.

An analytical model was developed and used to predict the yield and ultimate moments of each hinge. Comparisons of the experimental results and the analytical predictions are made. In addition, a time step, non-linear analysis was performed to determine the curvature and ductility demands placed on the prototype structure when exposed to severe seismic loading. The experimental curvature ductility exhibited by the specimens was compared to the curvature ductility demands from the model to assess whether a more earthquake resistant structure can be constructed using SIFCON hinges.

The objective is to develop a precast SIFCON hinge for use in seismic resistant moment frames. There are two primary factors that produce an effective hinge. First, an effective hinge must limit the loads on beam-to-column connections. Second an effective hinge must maintain stable flexural yielding, i.e. stable, inelastic, moment curvature behavior when exposed to nonlinear cyclic loading. Additionally, the yield and ultimate capacity, as well as its plastic cyclic behavior, must be highly predictable. Thus, to achieve the research objective, the yield moment, ultimate moment, and strength and stiffness
degradation properties of the hinge must be highly predictable. The final characteristic of a stable hinge is shear capacity. An effective hinge zone must exhibit stable shear capacity while undergoing the inelastic cyclic loading.

A secondary consideration of an effective hinge is maximizing energy absorption. As hinge energy absorption increases, structural damping increases and inertial forces decrease. This project will evaluate design and detailing parameters to maximize energy absorption of SIFCON hinges.

In summary, the research objectives are to produce a hinge that exhibits:
1. Stable, cyclic, inelastic moment-curvature behavior.
2. Highly predictable yield and ultimate moment capacities.
3. Highly predictable strength and stiffness degradation.
4. Stable shear capacity throughout nonlinear, flexural cycling.
5. A high level of energy absorption.

1.2 Research Significance

This research should lead to the use of SIFCON hinge zones in precast construction in high earthquake zones. The hinges would limit the forces applied to connections and create more earthquake resistant structures. In addition to the obvious safety benefits to both precast and monolithic construction, it should also remove many restrictions presently applied to precast concrete moment frames in high seismic zones. As a result, precast concrete frames could become a real alternative building system in Uniform Building Code (UBC) earthquake zones III and IV. This project is the first research conducted on the nonlinear cyclic behavior of lightly reinforced SIFCON flexural members.
1.3 Thesis Structure

This thesis is organized into 9 chapters. Chapter 2 presents a state-of-the-art literature review on previous work performed on all aspects of SIFCON. Previous research on compressive, tensile, shear, and reinforcing development length are discussed. The limited research on the flexural behavior of SIFCON is also discussed in chapter 2. Chapter 3 describes the experimental research program. This includes the prototype structure analysis and design, testing setup, instrumentation, and testing procedure. A list of the test specimens and the purpose of each specimen is discussed in Chapter 4.

Chapter 5 describes the behavior and results of each test specimen. Chapter 5 also discusses the behavior of the reinforcing steel and SIFCON matrix that make up each specimen. Moment curvature plots, curvature ductilities, and energy absorption results are discussed in this chapter 6. Chapter 6 also lists the SIFCON compression, and reinforcing tension test results.

An analytical moment curvature model of reinforced SIFCON hinges is presented in Chapter 7. This model is used to predict the monotonic behavior of the specimens tested in this project. Comparisons are made between the predicted and observed behavior in Chapter 7. A nonlinear, time-step, dynamic model of the prototype structure is performed in chapter 8. The hinge test results are used to input accurate material properties for the SIFCON hinges. The purpose of this analysis was to determine the effect of SIFCON hinges on the global behavior and performance of reinforced concrete frames exposed to severe seismic forces. Chapter 9 completes this thesis with observations, conclusions, modeling recommendations, and recommendations for future research.

Throughout the thesis, tables and figures are presented at the end of each chapter. Tables and figures are numbered according to their corresponding chapter and appear in the order of their occurrence.
2 LITERATURE REVIEW

2.1 Seismic Resistant Design Practice

Designing structures to maintain a fully elastic response throughout a severe earthquake event is cost prohibitive and impractical. Thus, the seismic design forces recommended by codes [UBC-1997; NZS 4203:1992] are considerably less than the inertial forces that result from the fully elastic response of a structure. The justification of this practice lies in a structure’s ability to withstand load and displacement beyond its yield levels. Consequently, design codes base the design seismic forces upon the achievable structure ductility. The more a structure can displace beyond its yield displacement, the lower the design seismic forces. As a structure displaces beyond it’s yield point, period shift, increased damping, and hysteretic energy dissipation are among several factors that justify the above practice [Restrepo et al, 1995].

The ductility of a structure or element is quantified by its displacement ductility factor, $\mu$. The displacement ductility factor is the maximum usable displacement of a structure or element normalized to its yield displacement ($\mu = \Delta_{\text{max}} / \Delta_{y}$). Ductility of precast concrete structures is considered to be less than cost-in-place construction. This is primarily due to inadequate connection ductility. Precast concrete structural connections typically exhibit brittle failure modes that have often led to catastrophic structural failures. These connections are almost always the weak link in the structural system [Park, 1995; Soubra, 1992]. As a result achieving ductile failure modes in precast concrete moment frames has been difficult.

The design objective for precast concrete moment frames in seismic zones is to emulate the inelastic, cyclic, behavior of monolithic construction. Numerous techniques exist for constructing earthquake resistant structures. All of these methods involve designing specific regions for post elastic deformation when exposed to severe seismic loading. In
a coupled wall system, the coupling beam is designed to yield in flexure. Moment frames can be designed for column yielding (column side sway) or beam yielding (beam side sway). Of these two mechanisms, the beam side sway mechanism makes more moderate demands on the curvature ductility required at the plastic hinge regions of the beams and at the column bases. Additionally, ductility can be more easily provided in beams than in columns [Park, 1995; Park and Pauley, 1975]. Therefore, the beam side sway mechanism is the preferred mode of post elastic deformation. This design approach is called the strong column-weak beam design approach, and is advocated in the Uniform Building Code [UBC-1997] and the New Zealand Code [NZS 3103:1995]. The New Zealand Code contains explicit provisions for a capacity based design to ensure strong column-weak beam behavior.

The nonlinear region must maintain sufficient strength and ductility throughout any deformations that might occur. Elements not involved in the failure must have sufficient strength to remain elastic during the inelastic cycling of the design failure mode. Hinges are typically used as the design mode of failure or ductility. However, they must be able to maintain high levels of cyclic strain curvature while maintaining stable and predictable moment and shear plateaus.

Flexural hinge regions have been used in cast-in-place moment frames since the mid 1960’s in an effort to create earthquake resistant structures. These hinge zones ensure a strong column-weak beam behavior under severe earthquake loading. This type of design is known today as the capacity based design approach [Park, 1995]. In capacity based design, the primary lateral load resisting systems withstand earthquake loads by isolating nonlinear yielding to specific regions which are specially designed to withstand the nonlinear loading. All other elements are then sufficiently over-designed as to remain linear elastic throughout the cyclic yielding of the designed yield regions. Hinges are typically used as the designed yield regions—providing the specified mode of ductility. In this manner, precast connection loads can theoretically be limited to the loads transmitted through the hinge zones.
2.2 Hinge use in Cast-In-Place Construction

Hinges were initially placed at the beam ends, adjacent to the column face, in an effort to prevent beam-column joint damage. This practice was partly successful, but as the hinge formed in the beam, the damage extended into the joint region [ACI 352-1995; Paulay, 1981; Park, 1986]. Subsequently, hinges were placed at some distance from the column face. Although successful at improving beam-column joint behavior, this practice greatly increased the demands on the hinge. Increased rotation demand and increased shear demand led to sliding along a through-depth crack and buckling of compression reinforcing bars in addition to other problems [Paulay, 1981; Al-Haddad and Wight, 1986; Paulay and Priestley, 1992; Abdel-Fattah and Wight, 1985; and Joh et. al, 1991]. Hinge lengths equal to the member depth, or 20 inches are recommended [Paulay and Bull, 1979; Park and Milburn, 1983; and Abdel-Fattah and Wight, 1985] to ensure proper reinforcement development within the hinge.

Shear sliding is a very difficult problem to overcome in reinforced concrete hinges. Without special web reinforcing, researchers have suggested limiting the nominal concrete shear stress to $3\sqrt{f_c}$ [Bertero and Popov, 1977; Scribner and Wight, 1978]. In addition, even when special web reinforcing is used, it is suggested to limit the web shear stress to $6\sqrt{f_c}$ if a ductility ratio of 4 or greater is expected [Bertero and Popov, 1977].

To overcome some of these problems, fiber reinforced concrete (FRC) was investigated as a material to strengthen hinges and improve their behavior. Olariu et al. [1988], Abdou [1989], Abdou [1990], and Soubra [1992] have all demonstrated the advantages of using FRC in flexural hinge zones. Improved confinement and post peak behavior have led to significant increases in ductility, and energy absorption, with less strength and stiffness degradation. Displacement ductilities up to six have been achieved. However, as a result of the significant post peak softening of fiber reinforced concrete, reinforcement detailing remains a critical factor to hinge behavior. Using SIFCON in
hinge zones of moment frames promises an even greater increase in performance than does using FRC.

FRC has been shown to produce superior strength, ductility, energy absorption, and less stiffness and strength degradation than reinforced concrete construction [Jindal and Hassan, 1984; Henager, 1977; Craig et al. 1984; and Sood and Gupta, 1987]. FRC hinges have been shown to increase strength, ductility, and energy absorption [Soubra, 1992; Soubra et al., 1991; Naaman et al., 1987, Vasconez et al., 1997].

2.3 Potential of Using SIFCON in Hinge Regions

The advantages and potential of FRC hinge regions have previously been discussed. It is expected that superior behavior can be achieved using structural elements made with SIFCON. Compressive strengths up to 20 ksi and compressive strains over 10% without severe strength degradation have been reported in SIFCON specimens [Homrich and Naaman, 1987]; Naaman et al., 1991; Naaman and Homrich, 1989]. Superior reinforcing bar development length, confinement, and toughness indicate the potential of using SIFCON hinges in seismic resistant structures [Hamza and Naaman, 1996; Abdou, 1990; Lankard, 1984; Naaman and Homrich, 1989; Naaman et al., 1987].

Figure 2.1 shows a comparison of the load deflection curves for a SIFCON specimen, a fiber reinforced concrete specimen, and a plain concrete specimen. A typical stress strain curve of a SIFCON specimen in compression is shown in Figure 2.2. A stress strain curve of a SIFCON specimen in tension is shown in Figure 2.3. The strength, ductility, and energy absorption capacity of SIFCON is exhibited by comparing the magnitude of the strain axis compared to known values for plain concrete.
2.4 An Introduction to SIFCON

SIFCON is similar to fiber reinforced concrete in that it has a discrete fiber matrix that lends significant tensile properties to the composite matrix. The fiber volume fraction, $V_f$ (volumetric percent of fibers), of fiber reinforced concrete is limited by the ability to effectively mix the fibers into the wet concrete. This limits the fiber volume ($V_f$) to between 1% and 3%, depending upon the type of fiber used and the workability of the mix. SIFCON specimens, on the other hand, have been produced with $V_f$ between 5% and 30% [Homrich and Naaman, 1987; Naaman et al., 1991]. In SIFCON, the fibers are preplaced inside the form prior to placing a cement-based binder. Once the fibers are placed, a fine, cement rich slurry is poured or pumped into the forms. Thus, the slurry must have sufficient fineness to infiltrate the fiber matrix. As a result, slurry mixes used to make SIFCON must be made using sand grains no larger than 150 Microns (no. 100 sieve). Additionally, $V_f$ is only a function of the fiber type and the vibrating energy inputted to fill the form.

There are four main variables to consider when evaluating a SIFCON specimen. These variables are slurry strength, fiber volume ($V_f$), fiber alignment, and fiber type. Each of these variables will be discussed in greater depth as they relate to individual characteristics of SIFCON specimens.

The cement slurry greatly affects the behavior of SIFCON specimens because the slurry is the backbone of the specimen. The elastic modulus, tensile strength, and compression strength of the slurry affect the behavior of the composite SIFCON matrix. Fiber pullout strength is just one variable that depends upon the slurry compressive strength.

As stated before, the fiber volume depends only upon the fiber type and the vibration effort. Smaller or shorter fibers will pack denser than longer fibers. Higher fiber volumes can be achieved with added vibration time.
Fiber alignment greatly affects the behavior of a SIFCON specimen. Fibers can be aligned normal to loading or parallel to loading. The ultimate strength, residual strength, ductility, and energy absorption are all be affected by the fiber alignment. These factors will be discussed later.

The two main fiber types are steel fibers and glass fibers. Steel fibers come in three main shapes and several sizes and strengths. Glass fibers are generally rod-like in shape with various lengths, diameters, and strengths. Steel fibers can be hooked, crimped or deformed with various aspect ratios (l/d). The most popular steel fiber is the Dramix fiber made by Bekaert Corporation. This fiber is “hooked” at each end and comes in three sizes. The sizes are 30 mm, 50 mm, and 60 mm long with diameters of 0.5 mm, 0.8 mm and 1 mm. Typically, the Bekaert fibers have yield stresses between 130 ksi and 170 ksi. For this study, the 30 mm by 0.5 mm hooked Bekaert fiber (denoted ZL 30/50 by Bekaert) was used exclusively. The smallest fiber shown in Figure 2.4 is the ZL 30/50 fiber used in this study.

2.5 SIFCON in Compression

The potential that SIFCON offers as a superior building material for seismic regions is evident in its behavior under a simple compression test. A typical stress strain curve of SIFCON in compression is shown in Figure 2.2. Peak compressive stresses usually occur at a strain greater than 1.5% [Homrich and Naaman, 1987; Naaman et al., 1991]. Additionally, SIFCON can undergo strains of over 10% while maintaining a residual strength over 60% of its peak strength. SIFCON compression specimens have been found to be over 50 times tougher than regular concrete of the same strength [Homrich and Naaman, 1987].

Although theoretical models have been developed to model the behavior of SIFCON in compression [Stevens, 1992], the most significant research in this area has been
Homrich and Naaman tested a series of over 35 three-inch diameter by six-inch tall SIFCON cylinders in compression. The variables investigated include fiber type, fiber orientation, slurry strength, and mold edge influence. Crimped, hooked, and deformed fibers were used in this study. For fibers aligned parallel to the loading direction hooked and deformed fibers showed better performance all along the stress strain curve. For fibers aligned normal to the loading direction, hooked and deformed fibers showed larger ultimate strength but also greater post peak strength degradation than crimped fibers. Additionally, research has shown that hooked fibers generally exhibit slightly higher peak strengths than deformed fibers [Naaman et al., 1991].

In comparing specimens made from 30 mm hooked fibers to those made with 50 mm hooked fibers, it was found that specimens made with 30 mm fibers show an increased strength over those made with 50 mm fibers [Lankard, 1984; Naaman et al., 1991].

Cored specimens showed a 15% to 30% higher ultimate and post peak compressive strength than molded specimens. It is important to note that the specimen width was less than three times the fiber length. Thus, it is reasonable to conclude that significant edge effects might be encountered. Results generally showed that approximately 15% to 20% greater peak stresses can be achieved with fibers aligned perpendicular to the loading direction as opposed to specimens with fibers aligned parallel to the loading direction. However, less post peak strength degradation was observed with fibers aligned parallel to the load direction. This difference can be seen in Figure 2.5.

Naaman et al. [1991] performed curve fits on the test results to produce an analytical model for SIFCON in compression. The compression curve was broken into the three sections shown in Figure 2.6. There is an ascending branch, a descending branch, and plateau region. The ascending branch is that part of the curve up to the maximum stress.
The descending branch is the part of the curve between the maximum stress and the inflection point where the curve transitions from negative curvature to positive curvature. The plateau region is that part of the curve that starts at the inflection point and asymptotically approaches a limiting plateau stress.

To model the stress strain curve in compression, Homrich and Naaman suggested two equations. The ascending portion of the curve can be modeled by:

$$\sigma_c = \sigma_{\text{max}} \left[ 1 - \left(1 - \frac{\varepsilon}{\varepsilon_{\text{max}}} \right)^A \right]$$  \hspace{1cm} (2.1)

where:
- $\sigma_c$ = Compression stress at strain, $\varepsilon$
- $\sigma_{\text{max}}$ = Maximum SIFCON compression stress from cylinder test
- $\varepsilon_{\text{max}}$ = Strain occurring at $\sigma_{\text{max}}$
- $A = E_o \left(\varepsilon_{\text{max}} / \sigma_{\text{max}}\right)$
- $E_o$ = Secant modulus or ACI 318 recommendation for plain concrete

The descending branch can be modeled by:

$$\sigma_c = (\sigma_{\text{max}} - \sigma_{\text{plat}}) e^{-b e^m \left(\varepsilon / \varepsilon_{\text{max}} - 1\right)^m}$$ \hspace{1cm} (2.2)

where:
- $\sigma_{\text{plat}}$ = Constant stress reached at the end of a compression test
- $\sigma_{\text{infl}}$ = Stress at which the compression curve reverses curvature
- $\varepsilon_{\text{infl}}$ = Strain at which the compression curve reverses curvature
- $m = \left(1 + \ln \left[(\sigma_{\text{infl}} - \sigma_{\text{plat}})/(\sigma_{\text{max}} - \sigma_{\text{plat}})\right]\right)^{-1}$
- $b = (m-1)/(m(\varepsilon_{\text{infl}} - \varepsilon_{\text{max}}))$

See Figure 2.6 for clarification of terms.
2.6 SIFCON in Tension

Limited information is available on the tensile properties of SIFCON. The most recent work using steel fibers was performed at the University of Michigan in the late 1980’s. Three types of fibers with two cement based slurries were investigated. Two series of tests were performed. One series with very high fiber volumes [Naaman et al., 1991], and one series with more practical fiber volumes [Naaman and Homrich, 1989]. Both of these programs were performed using 3” by 1.5” dog bone uniaxial tension specimens with 80% to 90% of the fibers aligned parallel to the loading direction.

All of these specimens underwent multiple cracking at low and intermediate levels of stress. Many small cracks developed up to the maximum load level. However, after the peak stress was reached the failure of these specimens proceeded along a single crack opening, with fibers debonding on either side of the crack area. As this primary crack opened and the load dropped, many of the smaller cracks closed. As a result, the elongation beyond the peak load represents the opening of this critical failure crack, so any strains calculated beyond this point can be misleading. Thus, Naaman and Homrich [1989] “translated” this crack opening into a strain by limiting the strain scale to 2% and using a flat plateau to model the behavior at the peak stress. Figure 2.7a shows the load-elongation curve for one of the tension specimens. Figure 2.7b shows how this same specimen is translated into a stress strain curve.

Test results for the very high fiber volume SIFCON specimens were impressive. These specimens exhibited average tensile strengths of up to 6 ksi in specimens made with 5 ksi compressive strength slurry, 30 mm long fibers oriented parallel to tension loading, and a fiber volume fraction of 16.7%. The same results have been achieved using the identical slurry, 50 mm fibers oriented parallel to loading, and a volume fraction of 12.2% [Naaman et al. 1991]. Compression cylinders made with this slurry and 9.5% of randomly placed 30/50 fibers exhibited a maximum compression strength of 13.0 ksi.
Packed with 7.1% of 50/50 fibers the maximum compressive strength is 9.8 ksi. When using 30 mm and 50 mm long hooked fibers, volume fractions over 16% and 12% respectively, require an excessive amount of vibration, and as such are impractical.

When using 30mm long by 0.5 mm diameter (Bekaert ZL 30/50 fibers), fiber volume of 10% to 12% are more practical. Volume fractions of 5% to 6% are more practical for 50 mm (Bekaert ZL 50/50) long hooked fibers [Naaman, et al, 1987; Homrich and Naaman, 1987; Naaman et al., 1991].

Naaman points out in the report that specimens with similar fiber reinforcement indexes ($V_f l/d$) gave similar strength results. For example, a specimen using 10% volume fraction of 30 mm long hooked fibers gave similar results as a specimen using 5% volume fraction of 50 mm long hooked fibers. This suggests that with similar fiber geometry, equivalent strength can be achieved using different fiber lengths if the fiber index is maintained. The fiber index is an important factor to consider when modeling SIFCON.

The more practical testing performed by Naaman and Homrich [1989] resulted in more moderate, yet still impressive, results. These results are more useful to the research performed in this program. The average tensile strength of the 30 mm hooked specimens using a 10 ksi compressive strength slurry was 2.26 ksi at a strain of approximately 0.9%. These specimens continued to hold 2.24 ksi at a strain of 2%. The compressive strength of the matrix was not given. But in comparison to Homrich and Naaman [1987] the compressive strength of this matrix can be estimated at approximately 14 ksi. Strains of up to 1% can be withstood with some recoverable strain. No statement was made concerning the limit on strain with full recovery of deformation.

The toughness index of SIFCON tension specimens was found to average between 600 and 800 times greater than regular concrete. Hooked fibers were found to perform better
than deformed fibers under uniaxial tension loading. However, deformed fiber SIFCON showed an initial modulus three times that of the hooked fiber SIFCON.

No model for predicting the strength of SIFCON in tension was presented. However, Naaman and Homrich suggested that the tensile strength of SIFCON is between “15% to 25% of its compressive strength depending on the orientation of the fiber axis to the axis of loading.” Two assumptions must be made to qualify this statement. One must first assume that the fiber orientation referred to is that of the compression specimens and not that of the tension specimens. Second, one must also assume that the compression specimen and the tension specimen have the same volume fraction and fiber orientation. It is important to note that all of the tension specimens in this study had 80% to 90% of the fibers oriented parallel to the loading direction. Thus, with random fiber distribution, the tension capacity of a SIFCON specimen should be somewhat less than the 15% to 25% suggested by Naaman and Homrich. Additionally, size affects for 1.5” wide tension specimens may be quite significant. Neither Naaman et al. [1991] nor Naaman and Homrich [1989] addressed size effects in their studies.

Figure 2.8 shows a model stress strain curve of SIFCON in tension. Naaman and Homrich performed a statistical curve fit to analytically model the ascending portion of a SIFCON stress strain curve in tension [Naaman and Homrich, 1989]. A model for the descending branch of this curve was proposed by Naaman et al. [1993] to be used for flexural analysis. The following equations were proposed for this model.

\[
\sigma_t = \sigma_{\text{tmax}} \left[1 - \left(1 - \frac{\varepsilon}{\varepsilon_{\text{tmax}}}\right)D\right]
\]

where:

\[\sigma_{\text{tmax}} = \frac{1}{3} \sigma_{\text{plat}}\] (Max tension stress of SIFCON)
\[D = \frac{E_i \varepsilon_{\text{tmax}}}{\sigma_{\text{tmax}}}\]
\[E_i = \text{Initial tension modulus}\]
\[\varepsilon_{\text{tmax}} = \varepsilon_{\text{smax}} + K \frac{V_{\ell}}{d}\] (Strain at max tensile stress)
\[ \epsilon_{\text{max}} = 0.0005 \quad (\text{Ultimate strain of unreinforced slurry}) \]

\[ K = 0.00174 \text{ for hooked fibers and } 0.00097 \text{ for deformed fibers} \]

Descending branch:

\[ \sigma_t = \sigma_{\text{max}} (1 - 2 \, k \, \delta / \ell)^2 \quad (2.4) \]

where:

\[ k = 1 \]

\[ \delta = \epsilon \, h \quad (\text{crack opening}) \]

\[ \ell = \text{fiber length} \]

\[ h = \text{depth of member recommended} \]

Homrich and Naaman go on to give approximations for \( \sigma_{\text{plat}} \) (compression plateau stress) and \( E_{\text{it}} \) (initial tension modulus) based solely on a simple compression tests of a SIFCON specimen. The approximations proposed by Homrich and Naaman are:

\[ \sigma_{\text{plat}} = V_f \ell / d \left( 200 + 8 \sqrt{\sigma_{\text{max}}} \right) \text{ psi} \quad (2.5) \]

\[ \sigma_{\text{plat}} = V_f \ell / d \left( 1.4 + 0.66 \sqrt{\sigma_{\text{max}}} \right) \text{ Mpa} \]

\[ E_{\text{it}} = k_1 \, V_f \, E_f + (1 - V_f) \, E_m \quad (2.6) \]

where:

\[ k_1 = 1/30 \]

\[ E_m = 1000 \text{ ksi (modulus of unreinforced slurry)} \]

\[ E_f = \text{Modulus of elasticity of fibers} \]

Using these approximations together with an assumption on the tensile strain at maximum tension stress, one can estimate the tension response of a SIFCON specimen. Several factors must be considered when using equations 3 through 6. First, these equations were all developed from tension test where 80 % to 90% of the fibers were oriented parallel to the loading direction. The fiber orientation referred to for the plateau stress \( (\sigma_{\text{plat}}) \) used in Equations 3 and 5 is unclear. It is reasonable to assume that the
tension capacity of a SIFCON specimen with random fiber distribution will be somewhat less than that given by Equation 3.

Second, in determining the crack opening in Equation 4, a “smeared crack” width equal to the member height is assumed. This was observed in the work performed by Naaman et al. [1993] on 1/3 point loading flexural tests. After significant cracking initiates in any cementitious material, tensile strain gives way to crack opening. To “translate” crack opening to strain, Naaman distributed the crack opening over a set length of the beam. Naaman refers to this length as the “smeared crack width.” During their testing, crack groups in flexural tests were observed at a spacing of 15 to 20 cm. The specimens they tested were 25 and 30 cm high. Thus a smeared crack width equal to the specimen height was presumed to be reasonable. However, actual crack spacing will vary based upon many factors. Reinforcing details and moment variation over a member length are just two prominent variables that will affect the crack spacing of any flexural member.

### 2.7 SIFCON in Shear

There is a good body of knowledge available on shear in fiber reinforced concrete. However, the equations for the shear strength predictions in fiber reinforced concrete do not give accurate results for shear strength predictions in SIFCON [Wang, and Maji, 1992; and Van Mier et al. 1991]. Shear tests on SIFCON have been performed with direct shear specimens [Balaguru and Kendzulak, 1987], torsion specimens [Wang and Maji, 1994], deep beam specimens [Mondragon, 1987], direct, double shear specimens [Naaman and Baccouche, 1995], and specimens under combined tension and shear [Van Mier et al., 1991]. Each of these tests was performed with various slurry strengths, fiber reinforcement indexes, and fiber types. As a result, only limited correlations can be drawn between each of these tests. Additionally, each test setup introduces another set of variables that must be considered. For example, a pure torsion test induces a linearly varying torsional shear stress across a cross-section. Thus, the shear strengths determined from torsional tests are average shear strengths, and not maximum shear strengths [Wang
and Maji, 1994]. It is generally agreed, however, that fiber orientation is the most important factor affecting SIFCON shear strength and behavior [Wang and Maji, 1994; and Van Mier et al., 1991].

Torsional specimens exhibited the most conservative results, while direct, double shear specimens exhibited the highest results. Wang and Maji [1994] found that torsional shear strengths were approximately 13% of the maximum compressive strength. They also suggested, from the results of deep beam tests and direct shear tests, that a conservative shear strength capacity of 20% and 25% of the maximum compression stress can be used.

Van Mier et al. [1991] investigated the effect of crack width, fiber type, and fiber orientation on the shear strength of SIFCON. Although many specimens did not fail, several conclusions were made. Hooked fibers performed substantially better than straight fibers. Specimens with fibers oriented normal to the shearing plane (fibers bridging the shear plane) generally could not be failed. And finally, crack widths of up to 200 micrometers appeared to have minimal affect on the ultimate shear strength of the specimens that could be broken. It is important to note 200 micrometer cracks are not visible to the naked eye.

The highest shear strengths achieved in testing come from direct, double shear specimens [Naaman and Baccouche, 1995]. SIFCON shear capacities achieved were 3.6 ksi for ZL 30/50 fibers with a compressive strength of 14.8 ksi, and 4 ksi for ZL 50/50 fibers and compressive a strength of 11 ksi. Thus, no correlation can be made using only compressive strength. Naaman and Baccouche used the “fiber reinforcement index”, $V_l/d$ ($V_f$ = fiber volume fraction, $l$ = fiber length, $d$ = equivalent fiber diameter), as a parameter to predict SIFCON shear strength. The fiber reinforcement index of each of these specimens is similar, and the slurry used in each of these specimens is identical. It is important to note that, all else being equal, SIFCON specimens using longer fibers usually exhibit lower compressive strengths than those using shorter fibers while maintaining the same fiber reinforcing index [Naaman et al., 1991; Homrich and
Naaman, 1987]. Thus, Naaman and Baccouche state that specimens having similar fiber-reinforcing indexes will achieve similar results. They go on to say that increasing the slurry strength had little effect on the shear capacity. Thus, it is possible that once minimal compressive strength is achieved, the fiber reinforcement index might become the controlling factor in the shear strength and behavior of SIFCON.

Naaman and Baccouche found that SIFCON specimens exhibited energy absorption values up to 250 times greater and shear capacities of up to 10 times greater than unreinforced concrete. Dowel reinforcement across the shear plane increased the shear strength by only 12%. The major advantage of dowel reinforcement was to improve the post peak behavior of the specimens. Dowel reinforcing reduces the strength degradation once adjacent shearing planes reach slips over 0.2 inches and increases the slip at which the peak shear stress occurs. Dowel reinforced SIFCON specimens exhibited energy absorption values 1000 times greater than unreinforced concrete.

To predict the shear strength of SIFCON, Naaman and Baccouche recommended the model proposed by Narayanan and Darwish [1988] for use on fiber reinforced concrete. The ultimate shear strength of SIFCON can be predicted as follows.

$$\tau_{ult} = \tau_{SIFCON} + 0.5 \rho f_y$$  \hspace{1cm} (2.7)

Where:

$$\tau_{SIFCON} = \tau_{fiber} + \tau_{slurry}$$

$$\tau_{slurry} = 400 \text{ psi} \hspace{0.5cm} \text{(for 6 ksi slurry)}$$

$$\tau_{fiber} = 3600 \text{ psi} \hspace{0.5cm} \text{(for mixes used in the study)}$$

$\rho$ and $f_y$ are the reinforcing ratio and yield strength of the dowel reinforcing across the shear plane.
2.8 Development Length of Bars Embedded in a SIFCON Matrix

Hamza and Naaman [1996] investigated the bond characteristics of reinforcing bars embedded in SIFCON made with 5% of Bekaert 50/50 fibers having a compressive strength of 8.9 ksi. For their tests, they determined:

- Average bond stress was over 2500 psi. Values of up to 4000 psi were obtained during testing. The range of bond stresses was between 2 and 4 times that of bars embedded in plain concrete.
- 50% to 160% higher bond stiffness than plain concrete.
- Pullout work was over 20 times greater than that for plain concrete.
- Reinforcing bars embedded in SIFCON can slip up to 10 times more than when embedded in plain concrete and still maintain the peak load.
- No cracking was observed up to 70% of the peak load
- Cover as small as ½ inch resulted in better performance than plain concrete.

Development lengths were determined to be between $4d_b$ and $8d_b$ (where $d_b =$ reinforcing bar diameter). These embedment lengths are approximately 40% to 50% than those required by the ACI 318-95 code. Reinforcing bars embedded in SIFCON exhibit higher bond strength, energy absorption, and load levels at larger slips than bars embedded in plain concrete, confined concrete, or fiber reinforced concrete.

2.9 SIFCON Shrinkage Strains

Research has shown that unreinforced slurry used in SIFCON can exhibit shrinkage strains up to 15% in 28 days, and grow to 25% in 100 days. However, Lankard [1984] has shown that SIFCON with 5 to 18 percent volume fraction exhibits relatively low shrinkage strains of between 0.02 and 0.05 percent. Lankard further states that “the magnitude of drying shrinkage strain for SIFCON is within the range exhibited by conventional Portland cement concrete (PCC).”
2.10 SIFCON in Bending

Only one program has been carried out investigating the flexural response of reinforced beams cast with SIFCON. This was performed in the early 1990’s at the University of Michigan [Naaman et al., 1992; Naaman et al., 1993]. Nine beams were tested under monotonic 1/3 point loading with a shear span to depth ratio of four. The testing program concentrated on specimens with reinforcing ratios exceeding that allowed by ACI-318 ($\rho > 0.75\rho_b$). Only two beams were tested with reinforcing ratios below the maximum reinforcing allowed by ACI-318. No research has been performed on the behavior of mildly reinforced SIFCON beams. The SIFCON compressive strength using 4.3% volume fraction of 50/50 Bekaert fibers was approximately 7700 psi. The SIFCON compressive strength using 8.8% volume fraction of Bekaert 30/50 fibers was approximately 11,100 psi.

SIFCON beams were shown to be 2.5 to 3.6 times more ductile, and absorb 3.3 to 5.7 times more energy than plain concrete beams [Naaman et al. 1992]. Reinforcement ratios of up to $3.5\rho_{\text{max}}$ ($\rho_{\text{max}} = 0.75\rho_b$) were tested, and it was determined that anything above $3.5\rho_{\text{max}}$ would lead to detrimental behavior. SIFCON beams made with 8.8% of the Bekaert 30/50 (30mm long) fibers achieved about 14% higher strength and substantially better post peak ductility than the beams made with 4.3% of the Bekaert 50/50 (50mm long) fibers. There was no difference in the behavior or crack patterns of SIFCON beams made without stirrups and SIFCON beams made with stirrups. This suggests that the use of SIFCON will eliminate the need for stirrup shear reinforcing. Nominal shear stresses found during testing (according to the ACI building code approach) were approximately 500 psi. Multiple cracking was observed in all beams tested.

First cracking occurred at random in the tensile zone, but not necessarily initiating from the bottom fiber. With continued loading, new cracks developed in close proximity to the old cracks. Often clusters of cracks developed and were spaced 15 cm to 25 cm apart.
Crack widths did not increase as loading continued. Instead, the SIFCON beams exhibited an increase in the number of cracks as loading continued. However, this might only be true for over reinforced SIFCON beams.

First cracking was used to estimate the modulus of rupture of the beam cast with 30/50 fibers. The modulus of rupture was judged to be approximately 4300 psi. The compressive strength of this mix was 11,100 psi. In another study [Hamza and Naaman, 1996] modulus of rupture values of 3510 psi, 4875 psi, and 5950 psi were reported for SIFCON specimens using 50/50 fibers and matrix compressive strengths of 5890 psi, 8850 psi, and 10,150 psi respectively. The average value for the modulus of rupture for SIFCON made with 50/50 fibers is 58% of its compressive strength. Fifty percent is a slightly conservative estimate. SIFCON made with 30/50 fibers shows much higher compressive strengths that SIFCON made with 50/50 fibers. The modulus of rupture of SIFCON made with 30/50 fibers is approximately 39% of the compressive strength, with 30% being a conservative estimate. An apparent contrast exists, however. Abdou [1990] states that 30/50 SIFCON specimens ($V_f = 6.4\%$) with a compressive strength of 4400 psi exhibit higher modulus of rupture values than 50/50 SIFCON specimens ($V_f = 4.5\%$) with compressive strengths of 5500 psi. From review of all the available research and experience manufacturing SIFCON specimens, producing SIFCON compression specimens with fiber volume fractions less than 9% with 30/50 fibers is a difficult task.

Naaman et al. [1993] modeled the flexural behavior of over-reinforced SIFCON sections with the following assumptions: (1) plane sections remain plane, (2) force equilibrium, and (3) compatibility of strains between the SIFCON matrix and the reinforcement. The compressive and tensile behavior previously mentioned was used to determine the material stress level at a given strain profile. Sargin’s [1971] model for the stress strain relationship of the reinforcing bars was used. This model is shown in Figure 2.9 and includes the yield plateau and strain hardening portion on the reinforcing stress strain curve. For SIFCON in tension, a smeared crack width equal to the section depth (25 cm) was used to compute the post peak tension stress in the SIFCON.
The model was shown to reasonably predict the moment curvature and load deflection behavior of the over reinforced SIFCON beams up to the maximum moment. However, the model did not accurately predict the post peak softening incurred in all of the specimens tested. This was explained by suggesting that the post peak softening was a result of highly localized compression failure that occurred in the extreme compression fibers of the specimens. Since the model was incapable of predicting a localized failure, the post peak flexural softening was not predicted.

### 2.11 SIFCON Used for Flexural Concrete Hinges

Much research has been performed on reinforced concrete and reinforced FRC hinges, but only two studies have been performed in which SIFCON was used in hinge regions. Abdou [1990], and Naaman, Wight, and Abdou [1987] tested 12 beams, 6 beam-column connections, and 3 cantilevered flexural specimens with SIFCON hinge regions. Vasconez, Naaman, and Wight [1997] tested two cantilevered flexural specimens with SIFCON hinge regions. As a result of the chosen reinforcing arrangement and the chosen connection details, isolating the hinging or failure within the SIFCON region was difficult. Indeed, many tests ended with failure outside the SIFCON region. However, shear span to depth ratios as small as 3 were used without requiring any beam shear reinforcing. None of the SIFCON regions contained any stirrups.

The results obtained by Abdou must be viewed with the following considerations. The first consideration is that all of the specimens were loaded with the reversed cyclic, displacement controlled loading shown in Figure 2.10. Note that each displacement cycle was only performed once. Thus, the softening effects of repeated cycling were not modeled. The second consideration is that all of the ductility values ($\mu$) reported for these tests were calculated from the load versus displacement plot for the whole beam and not the moment versus curvature plot for only the SIFCON region. Thus, the results
are not isolated to show just the performance of the SIFCON region. Instead, the results include the performance of the entire system.

The twelve 8”x10” beams investigated by Abdou [1990] were tested under cyclic, 1/3 point loading. A 10 inch long section in the middle of the 5 foot long beams was cast with the SIFCON matrix. As a result, the SIFCON hinge was loaded under constant moment with zero shear. The SIFCON matrix consisted of 5% of the Dramix 50/50 fibers and had a compressive strength of 4700 psi. These fibers are 50 mm long hooked fibers with a diameter of 0.5 mm. Only four of the twelve beams failed within the SIFCON section. The best results for SIFCON hinging were obtained from specimen B8. Figure 2.11 shows the reinforcing details used for this specimen. The SIFCON region was highly reinforced except for a weakened 2 inch long section in the center (see arrow). As a result, the failure was isolated within this 2 inch long section. No cracks appeared within the SIFCON region except for the primary failure crack. The displacement ductility ($\mu$) of this specimen, as determined from the load versus displacement plot of the entire beam, was 5. This result must be tempered by comparing it to specimen B9. In this specimen, the reinforcing across the joint between the SIFCON section and the concrete beam yielded. As such, the hinge formed at this joint and not within the SIFCON section. The displacement ductility for specimen B9 was 6—greater than that for specimen B8. The energy absorption required to fail the SIFCON beams were said to be 5 time greater than that required for reinforced concrete beams.

The 6 beam column subassemblies were tested under the same reversed cyclic loading as the beams. Each of these specimens consisted of reinforced concrete with a 10-inch long SIFCON hinge region beginning at a distance of h/2 from the column face. The shear span to depth (a/d) ratio of the hinges was 5.5. Five specimens were cast using a 4.5% of 50/50 Dramix hooked steel fibers with a compressive strength of 5500 psi. One specimen was cast using a SIFCON matrix of 6.5% of 30/50 Dramix hooked steel fibers with a compressive strength of 5500 psi. Only two of these specimens yielded within the
SIFCON region. Of the specimens that yielded within the SIFCON region, specimen X4 exhibited the best behavior.

The reinforcing details for specimen X4 are shown in Figure 2.12. Again, the hinge region was heavily reinforced except for a 1-inch length in the center. Predictably, failure was the result of the formation of one primary failure crack at the weakened plane of the hinge. This beam column subassembly only achieved a displacement ductility of 3.95. However, the load at this ductility had fallen to 60% of the yield load of the specimen. Yielding of the reinforcing bars joining the SIFCON hinge to the column occurred during the first cycle. It is likely that this joint yielding contributed significantly to this ductility value.

Specimen X5 was identical to specimen X4 except it contained the 30/50 fibers. This specimen did not yield within the SIFCON region. The hinge developed at the column face and led to concrete spalling in the beam-column joint region. This specimen withstood a displacement ductility of 3.5 while maintaining 70% of its yield load. The 30/50 fibers were said to increase the tensile strength and reinforcing bond strength. It was further concluded that these two phenomena led to increasing the moment capacity of the SIFCON region beyond what the adjacent concrete could withstand. It is noteworthy that, although the reinforcement within the SIFCON region yielded, very minimal damage was observed within the SIFCON section.

Three cantilevered flexural specimens were tested using SIFCON 11” wide, 14.5” deep, and 15” long hinges. These specimens were designed such that the connection interface between the SIFCON and the adjacent concrete would yield when the SIFCON hinge yielded. The SIFCON matrix used contained 4.5% of the 50/50 Dramix hooked steel fibers and had a compressive strength of 4400 psi. These specimens were all tested with shear span to depth ratios of 5.5. The reinforcing details for specimen T2 and T3 are show in Figure 2.13. All three T specimens eventually failed inside the SIFCON region, but only after reinforcing yielding had occurred at the joint connecting the SIFCON to the
adjacent concrete. Failure of all the specimens occurred as one primary failure crack opened within the SIFCON section. Specimen ‘T3’ was identical to ‘T2’ except the hinge was 21 inches long instead of 15 inches long. It was determined that lengthening the hinge had minimal effect in improving the system performance. A displacement ductility value of 4.5 was achieved with specimen ‘T3’ while maintaining 84% of its yield load.

Abdou concluded his work with the following statements:

1. Achieving distributed cracking within the SIFCON was impossible. Therefore, it is necessary to design the interface connections to yield simultaneously with the SIFCON hinge.
2. It was recommended that yielding at the column face be allowed at displacement ductility values over 3.
3. An 18% overdesign factor should be used for designing the moment capacity at the column face.
4. The moment capacity of the SIFCON hinge should be less than 70% of the moment capacity at the column face.
5. The moment capacity of the SIFCON hinge should be less than 95% of the moment capacity of the interface between the SIFCON region and the column side beam.
6. The moment capacity of the SIFCON hinge should be approximately 12% higher than the moment capacity of the interface between the SIFCON region and the beam side on the interior of the span.
7. With shear span to depth ratios of 5.5, no shear beam reinforcing was required in the SIFCON regions.

A few comments, from this author, on these recommendations are in order.
Recommendation number one is both cursory and impractical. With the chosen reinforcing details, it is almost certain that distributed cracking is not possible. But with modifications to the SIFCON reinforcing, distributed cracking may be possible. Additionally, designing the connections to yield simultaneously with the SIFCON hinge
may only work with the given moment curves tested. Recommendations 3 through 6 are the result of a regression analysis of all of the specimens. They simply represent the variation in the moment curve along the length of a beam span. These moment ratios can be derived more accurately by considering all the possible moment gradients in designing girders for the structure.

Vasconez et al. [1997] only tested two SIFCON cantilevered flexural specimens for hinging. The specimens were loaded with a quasi-static, reversed cyclic load with the test setup shown in Figure 2.14. Each displacement cycle was repeated three times, and a 75% yield cycle was executed prior to moving to the next greatest displacement cycle. The specimens were 11” by 14.5” with a 20” long hinge region. The shear span to depth ratio used for both specimens was 3. The matrix used consisted of 4.5% of the Dramix 50/50 fibers with a compressive strength of 3500 psi.

Although reinforcing yielding was observed within the SIFCON region, failure occurred at the interface connection to the testing base prior to developing any significant cracking in the SIFCON regions. As a result, limited data was obtained for either hinge. However, two important observations were noted.

First, reinforcing bar strain hardening was neglected when calculating the flexural strength of the SIFCON regions. As a result, the ultimate moment capacity of the SIFCON section was higher than the yield capacity and the hinges did not fail. Figure 2.15 shows a comparison of the ultimate moment capacity of the SIFCON, fiber reinforced concrete, and the concrete sections. The SIFCON sections exhibit significant strength increase as a result of strain hardening. But the reinforced concrete sections did not show a significant strength increase after the yield moment was reached. This phenomenon must be considered when designing SIFCON hinges.

Second, of the two SIFCON specimens, one contained shear reinforcing and the other did not. There was no noticeable difference in their performance. The maximum shear stress
resisted by the specimens was 300 psi. It was concluded that shear reinforcing is not required in SIFCON specimens.
Figure 2.1: Comparison of load deflection curves for SIFCON, FRC, and plain concrete beams [Naaman et. al 1987]

Figure 2.2: Typical stress-strain curve of a SIFCON compression specimen.
Figure 2.3: Typical stress-strain curve of a SIFCON tension specimen [Naaman and Homrich, 1989].

Figure 2.4: Various Bekaert Dramix fibers used to make SIFCON.
Figure 2.5: Effect of fiber orientation on SIFCON stress-strain behavior
[Naaman and Homrich, 1989]

Figure 2.6: SIFCON compression stress-strain curve model
[Homrich and Naaman, 1987].
Figure 2.7: Translation of crack opening to strain for SIFCON in tension [Naaman and Homrich, 1989].

Figure 2.8: Tension model of SIFCON stress-strain curve [Naaman et. al, 1993].
Figure 2.9: Stress-strain model for reinforcing [Sargin, 1971].

Figure 2.10: Quasi-static displacement controlled loading sequence used by Abdou [Abdou, 1990].
Figure 2.11: Reinforcing used in specimen B8. 1/3 point loading specimens by Abdou [Abdou, 1990].
Figure 2.12: Reinforcing used in beam-column specimen X4 [Abdou, 1990].
Figure 2.13: Reinforcing used in specimens T2 and T3 [Abdou, 1990].
Figure 2.14: Test setup used by Vasconez.
[Vasconez et. al, 1997].
Figure 2.15: Load rotation comparison between fiber reinforced, SIFCON, and concrete specimens [Vasconez et. al, 1997].
3 EXPERIMENTAL RESEARCH PROGRAM

Seven SIFCON hinges were designed, constructed, and tested in the experimental program. All hinges contained 9% to 11% volume fraction of Dramix ZL 30/50 fibers, and tested to investigate their use in seismic resistant applications. The purpose of the testing program was to assess how slurry strength, reinforcing geometry, and reinforcing strength affect hinge performance. Each hinge was tested under conditions simulating forces and displacements experienced in a full-scale structure when exposed to severe seismic loading. Thus, an analysis and design of a prototype structure was necessary to determine the nature and magnitude of forces exerted on the hinges, and appropriate member sizes.

3.1 Prototype Structure

The prototype structure is shown in Figures 3.1 and 3.2. This structure was chosen because it closely mirrored the structures considered by the U.S.-Japan cooperative on earthquake resistant structures [1997]. It is a twelve story, five bay, doubly-symmetric concrete moment frame with 24 foot bay spacing and rigid column bases. All six vertical moment frames are part of the lateral load resisting system. Beam sizes are 16”x24” on floors 2 through 6, 14”x22” on floors 7 through 10, and 12’x20’ on floors 11, 12, and the roof. Columns sizes are 24” square below the 5th floor, 22” square between the 5th and 10th floors, and 20” square from the 10th floor to the roof. A six inch slab thickness was assumed for all floors, and composite action of the girders and slabs was neglected in the prototype design.

The length of the SIFCON hinge region was conservatively set at 3’-9” (1.875d). This length should ensure adequate reinforcing development length, adequate room for proper connection detailing, and adequate length to achieve distributed hinge yielding [Paulay and Bull, 1979; Park and Milburn, 1983; Abdel-Fattah and Wight, 1987]. This hinge
length was determined from previous research on required yield zone length for reinforced concrete hinges and reinforced FRC concrete hinges.

An equivalent lateral force analysis was performed to determine appropriate member sizes and load magnitudes. Lateral loads were determined according to the special moment resisting frame (SMRF) provisions of the UBC [1997] in an earthquake zone 4 using a ductility factor (R) of 8.5, a soil profile of $S_c$, and a type ‘B’ seismic source type. A concrete strength of 6000 psi and Grade 60 reinforcing steel were assumed for design purposes. Beam and column stiffness were determined by using $0.7I_g$ for columns and $0.5I_g$ for beams. Fifty percent of $I_g$, rather than 35 percent as recommended by Section 10.11.1 of ACI 318-99, was used because it is the value recommended by Das [2000] and because SIFCON, as a result of its tension capacity, will maintain more of its stiffness than reinforced concrete when loaded to yield. The seismic forces were increased 10% to consider the effect of 5% structural eccentricity required by UBC. An additional 30% increase in the seismic forces was required by UBC to account for orthogonal effects. Loads used for the design are shown in Table 3.1. The required strength from Section 9.2 of ACI 318-99 is:

$$U = 0.75\{1.4DL + 1.7LL + 1.7(1.1E)\}$$

Where:

- $U$ = Ultimate design loads
- $DL$ = Dead loads
- $LL$ = Live loads
- $E$ = Earthquake (seismic) loads

The analysis resulted in a roof horizontal displacement of 9.5 inches. This exceeded the UBC drift limitation of approximately 7 inches. Increased member sizes are necessary to remain within the UBC drift limitations. Previous research [Das, 2000] indicates that minimal nonlinear action occurs when UBC drift limitations are met. Thus, in an effort
to increase hinge yielding in the nonlinear analysis, a prototype structure was chosen that exceeded UBC drift limitations.

The first floor beam identified in Figure 3.2 and shown in Figure 3.3 was critical. A detailed view of this beam is shown in Figure 3.3. In Figure 3.3, hinges are 3'-9" long beam regions adjacent to the columns. The moment on the beam is also shown in Figure 3.3, and clearly indicates that the left side hinge is the critically loaded hinge. This hinge was chosen as the prototype hinge for the experimental test program. The moment gradient for this hinge is approximately linear—varying between approximately 604 ft-k at the column face to 313 ft-k at the interior end of the hinge. The hinge moment to shear ratio (M/V) ranges between 6.7 ft and 4.5 ft. A cantilevered test setup was chosen because it would closely model the moment gradient exhibited by the prototype hinge.

3.2 Testing Setup

A geometric scale factor of 2/3 was chosen for model hinge testing. This resulted in a 10 inch wide by 16 inch deep by 30 inch long hinge region. The dashed line in Figure 3.4 shows the critical moment gradient of the prototype reduced by a factor of 8/27 (cube of the geometric scale factor). Note that the model hinge will have 2 inch thick steel plates at each end for testing purposes. Therefore, the length of the model hinge is 26 inches. The solid line is the linear moment gradient resulting from a cantilevered test with a moment to shear (M/V) ratio of 6 ft at the base of the hinge. This is the M/V ratio used in testing. As seen in Figure 3.4, the applied test moment is a good approximation to that of a 2/3 scaled prototype hinge. The scale factor for shear forces is 4/9 (squared of the geometric scale factor). As a result of the different scale factors for moment and shear forces, the moment gradient and shear gradient of the prototype could not be simultaneously maintained in the scaled test specimens. Maintaining the moment gradient was deemed more important and resulted in a more shear critical loading. Thus, a test setup was chosen to accurately model the moment gradient. This resulted in a M/V ratio of 4.75 ft in the center of the test specimens—varying from 6 to 3.5 through the
length of the specimen. The M/V ratio from the prototype varied from 6.7 to 4.5 feet. It is important to note that although the moment to shear force ratio of the test specimens differed from the prototype, the moment to shear stress ratio remained constant. In other words, the stresses on the scaled test specimens were the same as for the full-scale prototype.

The test setup is shown in **Figure 3.5**. The 2” thick bottom plate of each specimen is bolted to a W24 moment base. The moment base is bolted to a five inch thick plate, which is then connected to the reaction floor with 4 - 3” diameter high strength rods. The specimens are loaded by a 55 kip hydraulic actuator connected six feet from the specimen base. The other end of the actuator is connected to a strong wall using another five inch thick plate and 4 – 1 ½ inch diameter high strength rods.

The basic connection from the specimens to the testing apparatus is shown in **Figure 3.6**. All bolts connecting the specimens to the testing apparatus were tightened according to the turn-of-the-nut method outlined by the *Specification for Structural Joints* ... [1985]. Three rows of ¾” diameter x 6” long embedded connectors are used to connect the SIFCON to the specimen end plates. Two types of embedded connectors were used. Standard Nelson studs were used on the weak axis centerline of the specimen, and A325 bolts were used 3” to each side of the weak axis centerline of the specimen. To avoid confusion, both the embedded Nelson studs and the embedded A325 bolts are referred to as embedded connectors. The embedded Nelson studs were welded to the end plates while the embedded A325 bolts were continued through the end plates and used to connect the specimens to the testing apparatus.

Four-inch long connectors were used only on the first specimen (V44). To ensure failure in the interior of the specimens, it was decided to increase the connector length to 6 inches for all other specimens. Connector spacing, shown in Detail ‘A’, is 3 ¼” on center. The connector spacing shown in Section ‘A-A’ is 3” on center. Thus the resulting clear cover, in these two directions is 1 1/8” and 1 5/8” respectively. As a result
of group action of the embedded connectors and the toughness of SIFCON, this low cover was not considered to be critical.

As stated earlier, the center rows are 60 ksi yield strength Nelson studs welded to the end plates. The outside rows are 11” long A325 bolts with nuts on both sides of the mounting plates. To prevent the SIFCON from sliding on the plate, the interior nuts were welded to the plates. The threaded connectors were used to connect the hinge specimen to the testing apparatus via a slip critical, end plate moment connection at the upper and lower end plates. The testing apparatus was designed to remain linear elastic throughout cyclic yielding of the hinge. The specimen to hinge connection was designed for economy and ease of hinge replacement for this research program.

3.3 Instrumentation

Two different instrumentation plans were used. The first two specimens (V44 and H190) were tested using instrumentation plan ‘A’, shown in Figure 3.7. All other specimens were tested using instrumentation plan ‘B’, shown in Figure 3.8. The difference in the plans is that zone ‘E’ was added to plan ‘B’ after the first two specimens were tested. There was no instrumentation in plan ‘A’ that measured a zone of constant cross sectional properties. By adding zone ‘E’, zone ‘C’ was shortened such that it instrumented a region of a constant cross section. Approximately 40 channels were used to measure strains and deformations for each specimen. The instrumentation is intended to isolate the nonlinear behavior and energy absorption of different zones of the SIFCON hinge. With the exception stated above, curvature was measured in five separate zones of each specimen (zones ‘A’ through ‘E’). Curvature was measured from LVDT’s placed on the hinge extreme compression and tension surfaces. From the displacement measurements and load data, moment curvature relationships were plotted for each zone. Additional LVDT’s are used to measure separation between the bottom plate and the specimen.
Potentiometers are placed on the interior (weak axis) face of the specimens to measure the shear deformations near the anticipated failure zone. Consequently, shear deformations can be isolated from flexural deformations.

Potentiometers are placed on the full depth of the specimen in zone ‘C’. These potentiometers are intended to verify the assumption that plane sections remain plane. If there is a nonlinear strain profile in zone ‘B’, these potentiometers will establish the magnitude of the nonlinearity.

Strain gages were used to measure reinforcement strain in all specimens. Six locations were instrumented, as shown in Figure 3.8. These locations are on continuing bars at sections where other bars are bent or hooked, or where connectors are terminated. Strain gages were placed at the end of the bottom connectors (zone ‘B’), the end of the hooked #5’s (zones ‘B’ or ‘C’), and at the end of the hooked reinforcement in the center of the specimens (zones ‘C’ or ‘D’).

### 3.4 Hinge Testing Procedure

The testing methods discussed in this section were used for most of the specimens tested in this project. Specimen V44 used a variation of this procedure which is discussed in Chapter 5, section 5.2.

Test specimens were subjected to the nonlinear displacement-based cyclic loading shown in Figure 3.9. This loading sequence is modified from that recommended by the Precast Seismic Structural Systems (PRESSS) program [Priestley, 1997; Report on the Fourth U.S.-PRESSS Coordinating Meeting, 1994]. The modifications included three cycles at 25% (0.25\(P_{yn}\)) and 50% (0.5\(P_{yn}\)) of the nominal yield force, and three increments at 2.5 times the nominal yield displacement (2.5\(\Delta_{yn}\)). The displacements below the yield load were added to allow further investigation into the linear behavior of SIFCON specimens.
The $2.5\triangle y_n$ increment was added so that actual specimen yielding could be more accurately identified if it occurred between $2.0\triangle y_n$ and $3.0\triangle y_n$.

The first nine cycles are load-controlled displacements at 25%, 50% and 75% of the calculated nominal yield load. The yield load is defined as the load at which nominal yield strain is reached in the reinforcing at the anticipated critical failure plane. Once the displacement at 75% of the yield load is established, the displacement at yield load is estimated from reinforcing strains by equation 3.1. The only exception to this method is discussed in Chapter 5, section 5.2.

$$\triangle y_n = \frac{\triangle 0.75}{(\varepsilon_{0.75}/\varepsilon_{yn})}$$

where:
- $\triangle y_n$ = nominal yield displacement of the specimen
- $\triangle 0.75$ = specimen displacement at 75% of nominal yield load
- $\varepsilon_{0.75}$ = average first cycle, tensile reinforcing strain for the positive and negative displacements of the critical section for $0.75P_{yn}$
- $\varepsilon_{yn}$ = nominal reinforcing yield strain
  $(f_y/E = 60\text{ksi}/29000\text{ksi} \text{ for Grade 60 steel})$
- $P_{yn}$ = nominal yield load. Load at which reinforcing reaches nominal yield strain at the critical section

The initial loading rate was set at ¼ inch per minute. Upon reaching cycle 22 ($2.5\triangle y_n$) the displacement rate was increased to ½ inch per minute. At cycle 34 ($5.0\triangle y_n$) the displacement rate was increased to 1 inch per minute for the remainder of the testing. Data was read from all instrumentation once every 6 seconds. At the completion of the first cycle of every displacement increment, testing was paused so that cracks could be marked and measured, and photographs taken.

### 3.5 Compression Specimen Testing Procedure

Four by eight SIFCON compression cylinders were poured and tested to evaluate the SIFCON hinge material properties. Three by six unreinforced (plain slurry) compression
cylinders were tested for compression strength only. All SIFCON specimens were tested to obtain stress-strain behavior. Plain slurry specimens were tested at a load rate of 50 psi/minute until failure. SIFCON specimens were tested at a strain rate of approximately 0.0001 inch/inch/sec. Prior to testing, the top and bottom of all SIFCON specimens were ground to ensure true and parallel loading surfaces. Specimen displacement was measured by three potentiometers spaced at 120-degree intervals around the specimens. These potentiometers measured the relative vertical displacement between the upper and lower platens. Strain was calculated by dividing the average platen displacement by the original specimen height.

3.6 Tension Testing Procedure

All the reinforcing used was tension tested by an independent laboratory. There are eight different types of bars used. Two tension tests were performed on every size and type of bar. The yield and tension strength for each bar were determined, but no strain data was taken. Load versus elongation plots were recorded for each bar.

Reinforcing yield strain and strain at elongation strength is an input parameter for hinge analysis. Yield strains are calculated from the yield load by assuming a tensile modulus, \( E \), of 29000 ksi. This gives a strain reference point on the load-elongation plot. A strain scale factor can be applied to the elongation axis of the load-elongation curve by assuming linear elastic behavior up to yield. The ultimate strain is then scaled off of the load-elongation plot. This process is illustrated in Figure 6.1.
Table 3.1: Loads used for prototype structure design.

<table>
<thead>
<tr>
<th>Element</th>
<th>Load (psf)</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Frame DL</td>
<td>50</td>
<td>Beams and Columns</td>
</tr>
<tr>
<td>Slab DL</td>
<td>55</td>
<td>Slab weight</td>
</tr>
<tr>
<td>Sustained floor DL</td>
<td>30</td>
<td>Partitions / Mech. / Elect.</td>
</tr>
<tr>
<td>Sustained roof DL</td>
<td>30</td>
<td>Mech. / Elect.</td>
</tr>
<tr>
<td>Cladding DL</td>
<td>20</td>
<td>Siding on perimeter only</td>
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<tr>
<td>Floor LL</td>
<td>50</td>
<td>Live load</td>
</tr>
<tr>
<td>Roof LL</td>
<td>25</td>
<td>Live load</td>
</tr>
</tbody>
</table>
Figure 3.1: Prototype structure. Plan view.
Figure 3.2: Prototype structure. Interior frame elevation.
Figure 3.3: Critically loaded prototype beam and hinges.

Figure 3.4: Prototype moment gradient versus test moment gradient.
Figure 3.5: Test setup.

Figure 3.6: Specimen to testing apparatus connection details
Figure 3.7: Instrumentation plan ‘A’. Used for specimens V44 and H190 only.
Figure 3.8: Instrumentation plan ‘B’. Used for all specimens except V44 and H190.
Figure 3.9: Displacement-based testing sequence.
4 TEST SPECIMENS

This chapter will list and describe each specimen tested in this program. In addition, this chapter will describe the intended purpose of each specimen. This chapter will not discuss the end region connection details except where they differ from those discussed in Chapter 3 Section 2. The focus will be on the interior hinge reinforcing details. The method used to design the specimens will be briefly discussed in section 4.2. An in-depth look at the design of each specimen, including actual material strengths and hinge moment capacities, is given in Chapter 6. Section 4.3 will discuss how each specimen was fabricated. Section 4.4 will discuss the compression specimens used and how each was fabricated. Specimen fabrication can greatly affect fiber alignment. Thus, it is very important how each specimen was fabricated.

4.1 Slurry Mixes

Over 30 trial mixes were investigated for use in the testing program. It was difficult to develop mixes with the proper combination of viscosity and strength. The mix needed to exhibit low viscosity, negligible segregation, and strengths between 4 and 10 ksi. As fiber volume increases, the annular space available for slurry injection decreases. The fiber matrices used in this program were very difficult to inject using a high viscosity mix. Casting of one specimen was halted due to high mix viscosity and mix segregation.

Five of the 30 trial mixes were chosen for the testing program. The one-cubic foot quantities for mixes 1, 2, F3, F4, and F5 are given in Table 4.1. Table 4.1 also give the 28 day strength (using a 7 day wet cure) of each mix. The sand used for all the mixes was F-250 ground silica sand produced by The U.S. Silica Company. It is a ground sand with an AFS (American Foundrymen’s Society) grain size of 250. The consistency is similar to that of baking flour. The sand is completely dry and there is no known absorption value associated with this sand.
4.2 List of Specimens

A list of specimens is shown in Table 4.2. The purpose of each specimen is summarized in Table 4.3. Details of all seven specimens are shown in Figures 4.1 through 4.10. The specimen identification is as follows.

<table>
<thead>
<tr>
<th>Reinforcement Geometry</th>
<th>Nominal Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>H190D</td>
<td></td>
</tr>
<tr>
<td>(‘H’ = Longitudinal or ‘V’ = Inclined)</td>
<td>(‘D’ = Dywidag or ‘HS’ = High Strength reinforcing (comparable to Grade 75), otherwise Grade 60)</td>
</tr>
</tbody>
</table>

\[ A_s \times 100 \text{ (in}^2) \text{ (Area of tensile reinforcement)} \]

The columns in Table 4.2 include, fiber volume fraction, SIFCON matrix strength, area of tensile reinforcing, type of reinforcing used, slurry mix used, and specimen age at testing. Two types of reinforcing geometry were tested. The reinforcing was either inclined (Specimen V44), or horizontal (parallel to specimen faces). The matrix strength is the maximum compressive strength of the SIFCON matrix at testing. The fiber volume fraction is the percent volume of the specimen occupied by steel fibers. The area of tensile reinforcing is the total area of steel on each side of the geometric neutral axis of the specimen. Three different grades of reinforcing were used, Grade 60, High Strength (comparable to Grade 75), and Dywidag bars. Grade 60 and High Strength reinforcing have minimum specified yield stresses of 60 ksi and 75 ksi respectively. No clear yield stress is defined for Dywidag reinforcing. However, Dywidag bars have a nominal tensile strength of 150 ksi. No Grade 75 reinforcing bars could be located. In lieu of Grade 75 bars, heat treated Grade 60 bars, called Thermex and supplied by Ameristeel, were used because the mechanical properties of the High Strength Thermex bars were comparable to Grade 75 reinforcing steel. There were eight different bars used in the hinge specimens. These are listed below and in Table 4.4.

1) No. 3’s from Ameristeel, Grade 60, used in V44, and H146D
2) No. 4’s from Ameristeel, Grade 60, used only in H190
3) No. 5’s from Ameristeel, Grade 60, used in all specimens
4) No. 6’s from Ameristeel, Grade 60, used in H190
5) No. 6’s from Dayton-Richmond, Grade 60, used in H194
6) No. 7’s from Dayton-Richmond, Grade 60, used in H226
7) No. 6 Thermex bars from Ameristeel, comparable to Grade 75, used in H194HS
8) No. 5 Dywidag bars, ASTM A722, used in H118D, and H146D

All specimens were constructed using Dramix ZL 30/50 fibers produced by the BEKAERT Corporation. These fibers are 30 mm long (1.18 inches) with an equivalent diameter of 0.5 mm (0.0197 inches). The ends are bent (hooked) 90 degrees. The ultimate tensile strength of the fibers is 130 ksi. Figure 4.11 shows the Dramix 30/50 fiber.

4.3 Design of Test Specimens

A key to designing test specimens is understanding that the SIFCON tensile strain of approximately 0.5% at maximum tensile stress is more than twice the nominal tensile yield strain of Grade 60 reinforcing. Nominal yield strain of Grade 60 reinforcing is 0.2%. Consequently, when Grade 60 reinforcing begins yielding, only modest cracking is observed in a SIFCON flexural specimen. Extensive cracking and specimen degradation only develops after Grade 60 reinforcing exceeds its yield strain by over 100%.

Unlike previous research on SIFCON flexural members that used heavy and over-reinforced sections, this study considers only lightly reinforced sections. A yield zone was provided close to the center of each hinge in an effort to force first yielding deep within the hinge, away from the end regions. Decreasing the flexural reinforcement in the center of the specimens provides this weakened zone. Furthermore, an attempt was made to develop yielding in at least two locations within the interior of the hinges. This will help spread the yielding throughout the hinge and result in higher ductility.
The nominal moment capacity of each specimen was predicted using a computer program written by the author to predict the monotonic moment-curvature response of reinforced SIFCON beams. The full program is given in the Appendix. In the program, the tensile and compression stress strain equations discussed in Chapter 2 are used to calculate the stresses in the SIFCON and reinforcing bars with an assumed strain distribution. Force equilibrium is used to ensure the assumed strain profile was accurate. If the compression and tension forces do not balance, the strain distribution is adjusted accordingly. Once the tension and compression forces are balanced, the moment and curvature are computed. The strain distribution is then incremented to a higher level and the process is repeated. Failure is presumed to occur when the reinforcing reaches a tensile strain of 1.6% or the SIFCON reaches a compression strain of 8%. The program has been validated by comparing its output against flexural test results of previous research by Naaman et al. [1993].

Nominal moment capacity analysis (moment capacities based on nominal yield and nominal ultimate tensile strength of the reinforcing) will not be discussed in this paper. Discussing nominal and actual moment capacity analysis may lead to reader confusion. Instead, only actual moment capacity analysis will be discussed in Chapter 6. Thus, this section will not discuss moment capacities except in qualitative terms.

All specimens were symmetrical. Thus, when discussing the details of each specimen, only the tensile reinforcing will be discussed. The reader should understand that the reinforcing being discussed is symmetrical for both the tension and compression faces.

4.3.1 V44
Specimen V44, shown in Figure 4.1, was the first specimen tested. It was constructed using four inclined No. 3, Grade 60 reinforcing bars on each extreme fiber face. The SIFCON matrix had a compressive strength of 8.7 ksi and a fiber volume fraction of 9.2%. The reinforcing was inclined so that the moment capacity curve would more
closely match the gradient of the applied test moment. Fifteen connectors were embedded 4 inches into each end of the specimen. Ten of the connectors are A325 bolts also used to attach the test specimens to the apparatus. The five connectors in the center of the specimen are A307 Nelson studs. The No. 3 reinforcing bars were hooked within the embedded stud regions, providing adequate embedment for full bar development.

Specimen V44 was cast prior to determining the specimen moment capacity and, as a result, minimal steel was used to ensure failure within the hinge and not within the connection region. Subsequent analysis showed that 0.44 square inches of reinforcing steel was insufficient, forcing the critical failure plane to the left most end (bottom) embedded connectors.

4.3.2 H190
Specimen H190 was the first specimen designed with the critical failure plane within the central region (zones ‘C’ and ‘D’) of the hinge. The details for specimen H190 are shown in Figure 4.2. The reinforcing used in this specimen is shown in Figure 4.3. The SIFCON matrix had a compressive strength of 7.4 ksi and a fiber volume fraction of 9.2%. Grade 60 reinforcing was used exclusively in this specimen. Standard 90 degree hooks were used at the ends of all reinforcing termination. Fifteen ¾ inch diameter embedded connectors were used at the left and right end plates.

Two No. 6’s ran the full length of the specimen and were hooked within the embedded connectors at each end. According to previous research [Hamza and Naaman, 1996], this provided adequate embedment for full reinforcing development. To achieve failure within the interior of the hinge, two No. 4’s were hooked 15 inches from the left (bottom) embed plate. This termination of reinforcing forced the critical failure plane into zone ‘C’. There was a concern that the left most embedded stud region might undergo rigid body rotation, thereby creating stress concentrations at the end of the headed connectors. Thus, two No. 5’s were hooked 8 inches from the left end plate (4 inches for the end of
the embed connectors) as an additional measure to prevent failure at the end of the embedded connectors.

This specimen was designed such that first yielding was to occur in the center of the section (end of the No. 4’s). As loading continued, a second yield location was designed to occur at the end of the No. 5 reinforcing bars. Failure should occur at the end of the No. 4’s in the center of the section.

4.3.3 H194
Specimen H194 was designed after specimen H190 failed unexpectedly in the embedded stud region. To relieve the reinforcing congestion in the embedded stud region, the connection region was redesigned using bar splices connected directly to the end plates. Grade 60 reinforcing and 7.8 ksi SIFCON with a volume fraction of 9.3% was used in specimen H194. By using reinforcing hardware anchors, the top outer rows (three connectors each) of connectors were removed on both end plates. All hardware used were capable of developing 150% of the nominal yield strength of the reinforcing.

Figure 4.4 shows the details of specimen H194. Pictures of the top and side view are shown in Figure 4.5. Two No. 6’s run the full length of the specimen. In lieu of the hooks used in H190, the No. 6’s have upset threads at each end and are threaded into Dayton-Richmond coupler splices. These couplers are 1 ¼” in diameter by 2 ¼” long. They are connected to the end plates, and testing apparatus, by through bolting 7/8” diameter ASTM B7 high strength threaded rod. The two No. 4’s in the center of specimen H190 are replaced with one No. 6 bar in specimen H194. The No. 6 bar has upset threads that are screwed into a Dayton-Richmond welded half splice. The splice is welded directly to the end plate. The two No. 5’s in the interior of the specimen remain the same as in specimen H190. To prevent slip between the male threaded bar and the female threaded couplers, a rigid epoxy was placed on all threads prior to installation.
As in specimen H190, this specimen was designed such that yielding would occur throughout the section prior to failure. First yielding was to occur in the center of the section (end of the No. 6’s). As loading continued, a second yield location was to develop at the end of the No. 5 reinforcing bars. This would achieve two yield locations prior to ultimate failure developing at the end of the No. 6’s in the center of the section.

4.3.4 H226

The purpose of this specimen was to isolate the properties of SIFCON, such as strength and toughness, in an effort to determine which factors are critical to the performance of SIFCON hinges. By decreasing the moment strength contributed by SIFCON, increasing the moment strength contributed by the reinforcing, and comparing the results to other specimens, the strength and toughness properties of SIFCON can be more accurately isolated to determine which is more important.

Specimen H226 was cast using mix F3. This produced a SIFCON matrix strength of 3.1 ksi at a volume fraction of 9.4%. Grade 60 steel was used exclusively in specimen H226. The moment capacity lost by using a lower strength SIFCON matrix was balanced by using more reinforcing steel. Even though the SIFCON strength will be greatly reduced, the confinement, and toughness properties should remain relatively high.

As a result of the success of specimen H194, similar connection details were used for specimen H226. The lone variation is that no epoxy was used in specimen H226 to ensure no slipping between the male threads of the reinforcing and the female threads of the anchorage hardware. Figure 4.6 shows the details of specimen H226. This specimen is identical to specimen H194 except the two No. 6’s running the full length are replaced with two No. 7’s. Consequently, larger Dayton-Richmond couplers are required. The couplers are 1 ½” in diameter by 2 ½” long and connected to the end plate and testing apparatus by 1” diameter B7 high strength threaded rods. No epoxy was used between the couplers and the reinforcing bars.
As in specimen H194, this specimen was designed such that yielding would occur throughout the section prior to failure. First yielding was to occur in the center of the section (end of the No. 6’s). As loading continued, a second yield location was designed to occur at the end of the No. 5 reinforcing bars. This should achieve full specimen yielding prior to an ultimate failure flexural crack developing at the end of the No. 6’s in the center of the section.

4.3.5 H118D

This specimen was intended to evaluate the performance of high strength reinforcing used with SIFCON flexural regions. The main reinforcing in specimen H118D is Dywidag bars. Dywidag bars are made using ASTM A 722 steel. ASTM A 722 specifies an ultimate strength in excess of 150 ksi and a yield strength exceeding 80% of the ultimate strength. Consequently, the yield and ultimate strains will also be higher than that of Grade 60 or Grade 75 reinforcing steel.

Just as in specimen H226, the goal is to isolate the properties of SIFCON, such as strength and toughness, in an effort to determine which factors are critical to the performance of SIFCON hinges. An additional benefit of this specimen is that more extensive cracking will develop prior to yielding in the reinforcing. In theory, a broader distribution of larger cracks may result—possibly leading to very high energy absorption.

Mix F4 was used for specimen H118D and resulted in a maximum SIFCON compressive stress of 6.5 ksi at a fiber volume fraction of 10.8%. Figure 4.7 shows the details of the specimen. The main reinforcing consist of two No. 5 Dywidag bars (Area = 0.28 in²) that run the full length of the specimen. On the left of the specimen the bars are screwed into hex nuts, which are welded to the end plates. On the right end plate, the bars are screwed into Dywidag couplers, which are then welded to the end plate. To construct the specimen, the hardware is welded to the plates, and the Dywidag bars are screwed into the full depth of the couplers. The plates are then stationed the appropriate distance apart and the Dywidag bar is simultaneously backed out of the couplers and threaded into the
hex nut of the left end plate. This leaves adequate anchorage for the Dywidag bar at both end plates. No epoxy was used between the Dywidag bars and the hardware. The only other reinforcing are two No. 5’s hooked 10 inches from the left end plate (4 inches for the end of the embed connectors) as an additional measure to prevent failure at the end of the connectors. The end plates are connected to the testing apparatus using the standard ¾” diameter A325 threaded connectors on the interior, and 1” diameter A325 bolts in place of the B7 high strength threaded rods on previous specimens.

The critical failure plane in this specimen was located at the end of the interior Grade 60 No. 5 reinforcing (10 inches from the left end plate). First yielding and ultimate failure was expected at this location. This specimen was designed to achieve extensive cracking prior to yielding in the reinforcing. It was anticipated that larger energy absorption and ductility would result.

4.3.6 H194HS
The reinforcing layout of specimen H194HS is identical to specimen H194. However, there are three differences between the two specimens. Those differences are matrix strength, reinforcing grade, and anchorage hardware. A 6.8 ksi SIFCON matrix strength with a fiber volume of 9.4% is used for specimen H194HS. Grade 75 reinforcing is the main reinforcing in specimen H194HS. BarSplice grip-twist couplers are used for the anchorage hardware.

Standard Grade 75 reinforcing could not be located. However, heat treated “thermex” bars (produced by Ameristeel Corporation) exhibit mechanical properties meeting the requirements of Grade 75 reinforcing. Since these are heat treated bars, upset threads could not be machined into the bars because it would change the mechanical properties. Thus, suitable anchorage hardware was needed. BarSplice Product Incorporated produces a cold crimped rebar splice system that meets grade II seismic requirements (develop 150% of nominal yield stress). In this system, one end of a steel coupler is swaged onto the reinforcing bar, while threads are machined into the opposite end to
accept B7 high strength threaded rods. BarSplice grip-twist couplers perform the same roll at Dayton-Richmond coupler splices but are slightly larger. The BarSplice grip-twist couplers used were 2 5/8” diameter by 3 ¾” long.

**Figure 4.8** shows the details of specimen H194HS. A grip-twist coupler is swaged onto each end of the two No. 6’s that run the full length of the specimen. The couples are then anchored to the end plate and test apparatus using 7/8” diameter B7 high strength threaded bars. The one No. 6 that is hooked in the middle of the specimen is anchored to the end plate by using a combination of the BarSplice and Dayton-Richmond system. A Dayton-Richmond welded half splice is welded to the end plate. A 7/8 diameter B7 rod is threaded into the Dayton-Richmond anchor. The other end of the B7 rod is threaded into the BarSplice grip-twist coupler that is swaged onto the No. 6 rebar. No epoxy was used on the anchorage hardware for this specimen. Once again the interior No. 5’s are placed to prevent connection region failure.

As in specimen H194, this specimen was designed such that yielding would occur throughout the section prior to failure. First yielding was to occur in the center of the section (end of the No. 6’s). As loading continued, a second yield location was designed to occur at the end of the No. 5 reinforcing bars. This would achieve full specimen yielding prior to a critical flexural crack developing at the end of the No. 6’s in the center of the section.

**4.3.7 H146D**

Specimen H146D was constructed as a result of the unexpected failure mode observed in specimen H118D, using Dywidag reinforcing, but not Dywidag hardware. BarSplice Products Incorporated specially designed a grip-twist coupler to join No. 5 Dywidag bars to 7/8” diameter high strength B7 threaded bars. The couplers are 1 3/8” in diameter by 5 ¼” long. The purpose of using this hardware is to remove the reinforcing slip observed in specimen H118D, as discussed in Chapter 5 Section 2.5. Mix F7 with a fiber volume fraction of 10.0% is used, resulting in a SIFCON matrix strength of 6.4 ksi.
Figure 4.9 shows the details of specimen H146D. A picture of the reinforcing is shown in Figure 4.10. The reinforcing is the same as that for specimen H118D except for the addition of one No. 5 Dywidag bar, and two Grade 60 No. 3 ties. The main reinforcing consists of three No. 5 Dywidag bars. Two of these bars run the full length of the specimen and are 1 ½ inch inside the extreme fiber. The other No. 5 Dywidag is terminated in the center of the specimen and is 3 inches inside the extreme fiber. BarSplice grip-twist couplers are swaged onto each end of all the No. 5 Dywidag bars. The couplers are bolted, using 7/8” diameter B7 threaded rod, through the end plates to connect the hinge to the testing apparatus. The extra No. 5 Dywidag (3 inches down from the extreme fiber) is connected to the left (or bottom) end plate by threading the B7 rod into a Dayton-Richmond welded half splice, which is welded to the end plate. A 3”x3”x1/2” plate is anchored to the other end of the No. 5 Dywidag using 7/8” B7 rod. Two Grade 60, No. 3 ties were added, as shown in Figures 4.9 and 4.10, to prevent buckling of the compression reinforcing that was observed in a previous specimen. Grade 60 No. 5 bars are once again used at the end of the connectors to ensure failure outside of the connection region.

The goal is to isolate the properties of SIFCON, such as strength and toughness, in an effort to determine which factors are critical to the performance of SIFCON hinges. An additional benefit of this specimen is that more extensive cracking will develop prior to yielding in the reinforcing. In theory, a broader distribution of larger cracks may result—possibly leading to very high energy absorption. In this specimen, first yielding should occur at the ends of the No. 5 Dywidag bars. As loading continues, yielding should also initiate at the end of the Grade 60 No. 5 bars. Ultimately, failure should occur in the middle of the hinge near the end of the terminated No. 5 Dywidag bars.

4.4 Hinge Specimen Fabrication
Fiber orientation is predominantly affected by the technique used in placing fibers in the specimen. Fiber volume fraction is also affected by the fiber placement technique. Thus, a consistent fiber placement technique was used on all specimens. Once the reinforcing was positioned in a specimen, three sides of the form were placed. The top of the specimen (10 inches wide) remained open for fiber placement. The form was then placed on a vibrating table for fiber placement. The form was vibrated between 20 and 25 hertz while the fibers were hand sprinkled in the specimen using hand motions parallel to the specimen. This method promotes primary fiber orientation alignment parallel to the tension and compression axis. Extra care was taken to ensure that the connection regions were completely packed with fibers. Figure 4.12 shows fiber placement in a specimen. This procedure was continued until the form was full of fibers. Figure 4.13 shows a specimen full of fibers prior to placing the top of the form.

The specimens were placed on the vibrating table with the larger end plate down (90 degrees from that shown in Figure 4.12) for slurry injection. A vibrating frequency of 25 to 30 hertz was during injection. Using a grout pump, pressures up to 80 psi were used to inject each specimen through five 1” diameter injection ports in the forms. Forms were stripped the day after casting, and the specimens were wet cured 7 days. After 7 days of wet curing, all specimens were dry cured until testing.

There were a few problems during specimen injection. The most note worthy occurred on specimen V44. The slurry used for specimen V44 performed poorly. This resulted in a ½ inch to 1 inch void (fibers without slurry) along one face of the specimen. The void was patched by pouring a slurry, identical to the slurry used in the specimen, into this void two days after the specimen was cast.

4.5 Compression Specimens

SIFCON and plane slurry compression specimens were cast simultaneously with the SIFCON hinges. The appropriate fiber volume fraction was used in the SIFCON
compression specimens. However, proper fiber orientation was more difficult to obtain. As stated earlier, fiber orientation is a primary factor in the behavior of SIFCON specimens. Thus, fiber placement in compression specimens should be such that fiber orientation remains consistent to that of the hinge specimens. Fiber placement technique dictates fiber orientation.

A series of SIFCON compression tests were performed to determine an appropriate fiber placement method to accurately reflect the compression behavior of the SIFCON hinges. These included horizontally filled cylinders, diagonally filled cylinders, vertically filled prisms, and horizontally filled prisms. After testing these specimens, it seemed that diagonally filled compression cylinders would give adequate results if the fibers were placed with the following procedure.

After weighing out the proper amount of fibers, and setting the vibrating table at 25 hertz, the cylinders were filled in approximately 4 layers. A small sheet of plastic (24” square) was placed on the table and about 25% of the fibers were poured onto the plastic. Opposite ends of the plastic were lifted, held together, and shaken. This helped the fibers align in one direction. While still holding the opposite ends of the plastic, the cylinder was held at a 45° to 60° angle, and the fibers were poured into the cylinder. This process was repeated until the fiber volume fraction was achieved.

Compression specimens for the first five hinges (all hinges except H194HS and H146D) were cast in this manner. However, it became obvious that this method was not returning consistent stress-strain data. At this point, horizontally filled compression specimens and diagonally filled compression cylinders were cast. Two sizes of prisms were used for specimens H194HS and H146D. These were 4”x4”x8” long and 3”x3”x8” long.

Prisms were filled horizontally by hand sprinkling the fibers into the forms in a similar manner to that of the SIFCON hinges. The 3x3 and 4x4 prisms gave similar results, and it was concluded that this method should give an adequate indication of SIFCON
compression behavior. However, prismatic compression specimens were only cast and tested for two hinges. Thus to more accurately predict the compression behavior of the other five specimens, a correlation was drawn between the compression curve for horizontally cast prisms and diagonally cast cylinders. This correlation, instead of the actual cylindrical compression test data, is used to predict the compression behavior of all hinges when prismatic compression members were not cast (all hinges except H194HS and H146D).
Table 4.1: List of slurry mixes.

<table>
<thead>
<tr>
<th>Mix Item</th>
<th>One Cubic Foot Mix Quantities (lbs)</th>
<th></th>
<th></th>
<th>F3</th>
<th>F4</th>
<th>F5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Slurry Mix</td>
<td>1</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type III Cement</td>
<td>39.6</td>
<td>42.5</td>
<td>43.3</td>
<td>33.0</td>
<td>43.2</td>
<td></td>
</tr>
<tr>
<td>Fly Ash†</td>
<td>--</td>
<td>--</td>
<td>13.0</td>
<td>19.8</td>
<td>13.0</td>
<td></td>
</tr>
<tr>
<td>Sand (AFS-250)</td>
<td>59.6</td>
<td>55.3</td>
<td>39.4</td>
<td>37</td>
<td>39.3</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>26.9</td>
<td>27.7</td>
<td>28.1</td>
<td>31.7</td>
<td>28.7</td>
<td></td>
</tr>
<tr>
<td>Water Reducer</td>
<td>1.4*</td>
<td>1.0*</td>
<td>0.3**</td>
<td>0.1**</td>
<td>0.3**</td>
<td></td>
</tr>
<tr>
<td>28 Day Strength (ksi)</td>
<td>8.4</td>
<td>7.8</td>
<td>8.8</td>
<td>3.6</td>
<td>7.7</td>
<td></td>
</tr>
</tbody>
</table>

† Pro Ash, manufactured by Roanoke Cement Company. † Melment, manufactured by WR Grace & Company. ** AdvaFlow, manufactured by WR Grace & Company.

Table 4.2: List of hinge specimens with mechanical properties.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>SIFCON f_c at testing (psi)</th>
<th>V_f* (%)</th>
<th>A_t (in²)</th>
<th>Steel ratio (ρ)</th>
<th>Main Reinf. Type</th>
<th>Slurry mix</th>
<th>Slurry f_c at testing (ksi)</th>
<th>Age at testing (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V44</td>
<td>8.7</td>
<td>9.2</td>
<td>0.44</td>
<td>0.003</td>
<td>‘3’</td>
<td>1</td>
<td>8.4</td>
<td>28</td>
</tr>
<tr>
<td>H190</td>
<td>7.4</td>
<td>9.2</td>
<td>1.90</td>
<td>0.013</td>
<td>‘6A’, ‘4’</td>
<td>2</td>
<td>8.5</td>
<td>32</td>
</tr>
<tr>
<td>H194</td>
<td>7.8</td>
<td>9.3</td>
<td>1.94</td>
<td>0.013</td>
<td>‘6B’</td>
<td>F3</td>
<td>9.2</td>
<td>35</td>
</tr>
<tr>
<td>H226</td>
<td>3.1</td>
<td>9.4</td>
<td>2.26</td>
<td>0.016</td>
<td>‘7’</td>
<td>F4</td>
<td>3.4</td>
<td>21</td>
</tr>
<tr>
<td>H118D</td>
<td>6.5</td>
<td>10.8</td>
<td>1.18</td>
<td>0.008</td>
<td>‘5D’</td>
<td>F5</td>
<td>7.2</td>
<td>21</td>
</tr>
<tr>
<td>H194HS</td>
<td>6.8</td>
<td>9.4</td>
<td>1.94</td>
<td>0.013</td>
<td>‘6B’</td>
<td>F5</td>
<td>7.6</td>
<td>24</td>
</tr>
<tr>
<td>H146D</td>
<td>6.4</td>
<td>10.0</td>
<td>1.46</td>
<td>0.010</td>
<td>‘5D’</td>
<td>F5</td>
<td>7.1</td>
<td>19</td>
</tr>
</tbody>
</table>

* Dramix ZL 30/50 fiber
Table 4.3: Purpose of each specimen.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Specimen Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>V44</td>
<td>Control specimen to show the basic behavior of unreinforced SIFCON hinge.</td>
</tr>
<tr>
<td>H190</td>
<td>First attempt at real hinge. Achieve failure in central hinge region (zones 'B', 'C', or 'D').</td>
</tr>
<tr>
<td>H194</td>
<td>Redesign of H190. Redesign of connection region to relieve reinforcing congestion at bottom end plate.</td>
</tr>
<tr>
<td>H226</td>
<td>Same moment capacity as H194 except using more steel with lower strength SIFCON matrix. Compare results to H194.</td>
</tr>
<tr>
<td>H118D</td>
<td>Investigate behavior of SIFCON using very high strength reinforcing (Dywidag bars). Expect extensive cracking at steel yield.</td>
</tr>
<tr>
<td>H194HS</td>
<td>Investigate behavior of SIFCON using Grade 75 reinforcing. Compare results to H194. More cracking expected at steel yield.</td>
</tr>
<tr>
<td>H146D</td>
<td>Redesign of H118D (Dywidag reinforced specimen) using swaged hardware anchors to prevent reinforcing slip.</td>
</tr>
</tbody>
</table>

Table 4.4: Definition of reinforcing steel types used in specimens.

<table>
<thead>
<tr>
<th>Reinforcing Type</th>
<th>Reinforcing Grade</th>
<th>Reinforcing size</th>
<th>Reinforcing supplier</th>
<th>Specimens used in</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘3’</td>
<td>60</td>
<td>No. 3</td>
<td>Ameristeel</td>
<td>V44</td>
</tr>
<tr>
<td>‘4’</td>
<td>60</td>
<td>No. 4</td>
<td>Ameristeel</td>
<td>H190</td>
</tr>
<tr>
<td>‘5’</td>
<td>60</td>
<td>No. 5</td>
<td>Ameristeel</td>
<td>All</td>
</tr>
<tr>
<td>‘6A’</td>
<td>60</td>
<td>No. 6</td>
<td>Ameristeel</td>
<td>H190</td>
</tr>
<tr>
<td>‘6B’</td>
<td>60</td>
<td>No. 6</td>
<td>Dayton-Richmond</td>
<td>H194 &amp; H226</td>
</tr>
<tr>
<td>‘7’</td>
<td>60</td>
<td>No. 7</td>
<td>Dayton-Richmond</td>
<td>H6-9-226</td>
</tr>
<tr>
<td>‘6HS’</td>
<td>75</td>
<td>No. 6</td>
<td>Ameristeel</td>
<td>H194HS</td>
</tr>
<tr>
<td>‘5D’</td>
<td>ASTM A 722</td>
<td>5/8” (0.28 sq. in.)</td>
<td>Dywidag Inc.</td>
<td>H118D &amp; H146D</td>
</tr>
</tbody>
</table>
Figure 4.1: Specimen V44. Top view and side view
Figure 4.2: Specimen H190. Top view and side view.
Figure 4.3: Preparation of specimen H190.

a) Top view.

b) Side view.
Figure 4.4: Specimen H194. Top and side views.
Figure 4.5: Preparation of specimen H194.
Figure 4.6: Specimen H226. Top and side view.
Figure 4.7: Specimen H118D. Top and side view.

- 2 No. 5's
- 2 No. 7's (T&B)
- No. 6 (T&B)
- Tack Welded (TYP)
- No. 7 Double sided dowel in with couplers at ends
- 1" HS B7 Threaded Rod used for connection
- 9 - 3/4" studs each plate. Three rows deep.
- 2'-2"
- 2" 10"
- 2" 5"
- 2" 1'-4"
- 2 No. 7 Couplers Typ. 4 places T&B
- Note: Studs not shown for clarity
Figure 4.8: Specimen H194HS. Top and side view.
Figure 4.9: Specimen H146D. Top and side views.
Figure 4.10: Side View of specimen H146D.

Figure 4.11: Dramix ZL 30/50 fiber used exclusively in this program.
Figure 4.12: Fiber placement in specimen H146D.

Figure 4.13: Specimen H146D filled with fibers.
5 HINGE SPECIMEN BEHAVIOR

This chapter discusses hinge specimen behavior during testing. There are two phenomena to understand and one term to define before discussing specimen behavior. With the fiber volume fraction considered in this study, SIFCON fibers tend to bridge cracks. Numerous hairline cracks develop within about one inch of the initial crack. These crack widths are difficult to measure. Spalling often occurs before a measurable crack width develops—usually leading to inaccurate measurements. As a result, crack width measurements in SIFCON specimens can be extremely difficult to make, and even misleading.

Plain slurry on the surface of SIFCON specimens greatly affects cracking patterns. When specimens were cast, they were injected from one face to another using injection pressures up to 80 psi. These pressures compressed the individual fibers on the injection face of the specimens, and result in a ¼ inch to 1 inch layer of unreinforced, plain slurry on the face on which the specimen was injected. Thus, instead of numerous small cracks, these surfaces exhibit fewer larger cracks. In addition, surfaces with plain slurry tend to crack at earlier stages of loading.

The term “depth of specimen” will be used to discuss specimen behavior. Depth is defined as the distance between the extreme fiber faces. Thus, the full depth of the specimens is seen by viewing the 16 inch deep side faces of the specimens. (east and west faces).

All the hinges exhibited similar characteristic behavior. The first indication of specimen loading was the separation between the specimen and the bottom end plate. As loading progressed up to the yield load, a good distribution of cracking developed on the weak side faces, flexural cracking initiated on the extreme fiber faces, and the crack opening at the base of the specimen increased. After exceeding the yield load, flexural cracking became well distributed with individual cracks migrating into the depth of the specimen.
In some cases the flexural cracks became more inclined as they grew into the depth of the specimen, but in most cases they did not. A primary flexural crack usually developed after reinforcing strains exceeded 6000 microstrain, which led to failure. Further loading resulted in the opening of the primary failure crack in the critically loaded zone. After significant opening of the primary failure crack, reinforcing bar rupture usually halted testing.

5.1 Specimen V44

Specimen V44 was the first specimen tested. As noted in chapter 3, the displacement sequence did not include intermediate cycles at 0.75 yield displacement. SIFCON compression cylinders tested prior to quasi-static testing indicated a SIFCON matrix compressive strength of 5 ksi. However, a subsequent compression test performed a day after the quasi-static testing indicated a compression strength of 8.7 ksi. It is believed that the actual compression strength is 9 ksi. Moment capacity calculations performed for the purpose establishing the testing procedure were based upon the 5 ksi initial compression strength. As a result, the quasi-static testing sequence was adjusted as specimen V44 was tested.

First cracking occurred during cycle 10. Strain gages did not function properly during testing. Cycle 10 was the first displacement cycle at 0.365 inches. One inch long cracks appeared in zone ‘B’ on the extreme fiber faces. No further cracking was seen until cycle 20. Cycles 20, 21, and 22 were load controlled cycles at 24.7 kips. The displacement increased from 1 inch at cycle 20 to over 1.5 inches at cycle 22. Consequently, the yield displacement for further testing was set at 1.0 inches. Cycle 20 resulted in a primary failure crack opening in zone ‘A’ at the end of the embedded connectors.

Cycles 23, 24, and 25 were load-controlled cycles performed to exceed 24.7 kips (load from cycle 20). However, when a displacement of 1.7 inches was reached, the actuator was switched to displacement controlled loading. A load of 23 kips was reached for
cycle 23, but decreased to 19.7 kips at cycle 25. The specimen was clearly yielding. These three cycles resulted in the opening of the primary failure crack in zone ‘A’ at the end of the embedded connectors.

Cycle 26 resulted in the primary flexural crack penetrating 90% into the depth of the specimen. Cycle 26 was a displacement cycle of 2.0 inches. The load reached for cycle 26 was 20.7 kips, but decreased to 15.8 kips upon cycle 28. Cycle 29, 30, and 31 were displacement cycles of 2.5 inches. Once again the load fell from 16 kips during cycle 29 to 9.2 kips for cycle 31. Similarly, cycles 32, 33, and 34 were displacement cycles at 3.0 inches. The load decreased from 8.4 kips for cycle 32 to 7.1 kips for cycle 34. Opening of the primary failure crack continued with no new cracks forming in the specimen. Failure occurred during cycle 35 when a reinforcing bar ruptured in tension. Figure 5.1 shows the specimen after testing. Figure 5.2 shows a close-up of the primary failure crack. After completing the test the specimen was broken in two pieces. The lower piece of the specimen is shown in Figure 5.3. The heads of the connectors are seen in this figure.

Although specimen V44 contained inclined tensile reinforcing as shown in Figure 4.1, testing showed that the reinforcing was not adequate to effectively bridge the primary failure crack. Once the primary flexural crack opened beyond the fiber interlock capacity of the SIFCON, the specimen softened and no further cracking was seen in the specimen. This specimen performed like an unreinforced SIFCON flexural member in that there was not sufficient reinforcing to effectively bridge the primary failure crack and distribute damage.

5.2 Specimen H190

The outer ¾ inches of slurry on the west face had no fibers. As a result, the west face exhibited cracking before the east face, and the cracking pattern of the west face is much different than that of the east face (compare Figures 5.4a and 5.4b).
Minor cracking was observed on the extreme fiber faces (north and south faces) during cycles 1, 2 and 3 (0.25Pyn). During cycle 4, separation between the base plate and the specimen was observed. Cycle 4 was the first cycle to 0.5Pyn. Vertical cracks also appeared on the west face over the outside rows of embedded connectors (1.5 inches inside the extreme fiber faces) at both top and bottom of the specimen. The east face of the specimen showed no evidence of cracking.

The first clearly defined flexural cracks appeared on the extreme fiber faces during cycle 7. Cycle 7 was the first cycle at 0.75Pyn. During this cycle, flexural cracks appeared on the extreme fiber faces in zones ‘A’, ‘C’, and ‘D’ (end of the embedded connectors at the top and bottom of the specimens, and near the end of the No. 4 reinforcing bars at the mid-height of the specimen). Once again, there was no cracking on the east face of the specimen. Reinforcing bar strain gage readings at the critical section (in zone ‘C’) averaged 1534 microstrain (0.1534 % strain) during cycle 7, and the average displacement for cycle 7 was +/- 0.54 inches. Unusual strain gage readings were observed in zone ‘A’ during cycle 7. The strain gages in zone ‘A’ were located on the No. 6 reinforcing bars at the end of the connectors. During the positive stroke, a tensile strain of 1385 microstrain was recorded. However, during the negative stroke, a tensile strain of only 610 microstrain was recorded. This indicated that slip was occurring in the south face reinforcing during reversed loading. Further testing revealed that the slip was the result of a splitting tension failure plane that developed over the outer rows of connectors.

The displacement for cycle 10 was calculated as described in Chapter 3, from Equation 3.1, as 0.77 inches. On the 10th cycle, the specimen was displaced 0.65 inches (0.75Δyn), then displaced in 0.05 inch increments until an end displacement of 0.8 inches was reached. After each increment the load and strain in the reinforcing in the middle of the specimen was noted. This incremental displacement method was used to ensure that the specimen was not loaded significantly beyond its yield load. At the positive stroke
displacement of 0.8 inches the strain in the reinforcing at the critical section was 2,080 microstrain. Thus, the nominal yield displacement of this specimen was set at 0.8 inches ($\Delta_{yn} = 0.8''$). The strain in the tensile reinforcing in zone ‘A’ (end of the connectors) was 2080 microstrain during the positive stroke but only 745 microstrain during the negative stroke. Again, the tensile reinforcement at the end of the connectors was not being developed under reversed loading. The average load for cycle 10 was 22.9 kips.

First cracking on the east face was observed during cycle 10. These cracks appeared at two locations—the end of the connectors, and toward the end of the No. 4’s at mid-height on the specimen (zones ‘A’, ‘C’, and ‘D’). Additionally, and possibly more importantly, was a vertical crack at the base of the specimen. This vertical crack developed over the outside row of connectors on the east side (see bottom right of specimen in Figure 5.4a). This crack appears to have initiated at an injection port, where the fiber volume fraction was low. Cracking on the west face of the specimen continued to develop and lengthen, while and the vertical cracks over the outside row of connectors (1.5 inches from the extreme fiber faces) continued to open.

Cycles 14, 15, and 16 were displacements at $1.5\Delta_{yn}$ (1.2 inches). The incremental displacement method described above was used to reach the first positive displacement cycle of 1.2 inches. The average reinforcing strain at the anticipated failure plane in zone ‘C’ was 3,410 microstrain. Once again there was a large discrepancy in reinforcing strain in zone ‘A’ during the positive and negative strokes. The tensile reinforcing in zone ‘A’, for positive loading, indicated a strain of 3016 microstrain (45% higher than nominal yield strain). However, the tensile reinforcing strain was only 960 microstrain for negative loading. Obviously the tensile reinforcing was not being developed for negative loading. The average load for cycle 14 was 28.1 kips. During cycle 14, cracking on the east face started becoming more significant with the number of new cracks distributed equally in zones ‘B’ and ‘C’. On the north and south faces of the specimen, many new, well distributed cracks were produced in zones ‘B’, ‘C’, and ‘D’. However, few new cracks developed near the ends of the connectors.
Cycle 17 was a 75% ($0.75\Delta_{yn}$) displacement cycle. Softening of the specimen had initiated because the load required for cycle 17 was only 4.2 kips. The initial 75% displacement cycle required a 19.5 kip load.

Cycles 18, 19, and 20 were displacements cycles at $2.0\Delta_y$ (1.6 inches). The incremental displacement method was used to reach the first positive displacement cycle of 1.6 inches. The average reinforcing strain at the anticipated failure plane (in zone ‘C’) was 8350 microstrain. The average load for cycle 18 was 30.0 kips. Once again there was a large discrepancy between the strains obtained in zone ‘A’ for the positive and negative displacement cycles. This was a clear indication of reinforcing slip in zone ‘A’ on the south face of the specimen. Few new cracks developed during cycle 18. During cycle 18 the east face vertical crack over the outside row of embedded connectors in zone ‘A’ grew to cover almost the full length of the stud. **Figure 5.4** shows the cracking on the east and west faces of the specimen after cycle 18. Separation from the base plate had grown to 1/16th inch during cycle 18. Upon the completion of the 18th cycle, the specimen was displaced to the zero position and left over night. Testing was resumed the next day at cycle 19.

At the completion of cycle 20, the flexural cracking on the east and west faces had grown across the full depth of the weak axis faces over the end of the embedded connectors (zones ‘A’ and ‘D’). However, little crack distribution developed near the critical section in zone ‘C’ (ends of the No. 4 reinforcing bars). Contrarily, the north and south faces exhibited little crack growth near the ends of the connectors, but developed a well-distributed flexural crack pattern in zone ‘C’ at the critical section (end of the No. 4 bars). The separation of the specimen from the base plate had grown to approximately 1/8th inch during cycle 20.

Cycle 21 ($0.75\Delta_{yn}$) only required an average load of 2.3 kips. Once again, this was a clear indication of specimen softening.
Cycles 22, 23, and 24 were displacements cycles of $2.5\Delta y_n$ (+/- 2.0 inches). The incremental displacement method was used to reach this displacement on the first positive displacement cycle. Once this displacement was reached, the displacement rate was increased to $\frac{1}{2}$ inch per minute for the duration of the testing. The average reinforcing strain at the anticipated critical section (zone ‘C’ near mid-height of the specimen) was 10,600 microstrain. The average load for cycle 22 was 30.6 kips. In zone ‘A’, the tensile strain gage for positive displacement ceased working, but the tensile strain gage for the negative displacement returned a reading of 3175 microstrain. This indicated that the south face reinforcing in zone ‘A’ had finally begun yielding. For cycle 24, the average crack width between the base plate and the specimen at full positive and negative displacements was approximately $\frac{3}{16}$ inch.

The only new cracks that developed during cycle 22 occurred mainly at the top of the specimen near the heads of the embedded connectors. The only noteworthy new cracks were vertical cracks over the embedded connectors at the base of the north face.

The $0.75\Delta y_n$ displacement cycle 25 only required an average load of 1.6 kips. Once again, this was a clear indication of specimen softening.

Cycles 26, 27, and 28 were performed at $3.0\Delta y_n$ (2.4 inches). The incremental displacement method was used to reach this displacement on the first positive displacement cycle. The critical section reinforcing strain during positive displacement was 12,170 microstrain. No strain was recorded for the negative displacement because the strain gage on the south face in zone ‘C’ had ceased working. The average maximum load reached for cycle 26 was 30.7 kips. The maximum crack width between the base plate and the specimen during cycle 28 was approximately $\frac{1}{4}$ inch.

During cycle 26, significant cracking developed on the west face at the base of the specimen over the connectors. The most significant of these cracks was the opening of
the vertical crack over the southern most connectors. New cracks, creating a well-distributed cracking pattern, developed on the east face on the interior depths of the specimen. Additional crack opening was noted on the south face in zone ‘C’. At the conclusion of cycle 28, spalling had begun near the base of the specimen.

Cycles 30, 31, and 32 were performed at 4.0\(\Delta y_n\) (3.2 inches). Displacement increments of 0.05 inches were used up to 2.8 inches. Then 0.1 inch displacement increments were used up to the required 3.2 inches. The reinforcing strain in zone ‘C’ for the positive displacement cycle was 13,280 microstrain. No strain was recorded on the negative displacement cycle because the strain gage was damaged. The average maximum load reached for cycle 30 was 32.0 kips. The maximum crack width between the base plate and the specimen during cycle 32 was approximately 3/8 inch.

Cracking during cycles 30, 31, and 32 was concentrated at the base of the specimen over the connectors. Spalling and major cracking was observed on the extreme fiber faces (north and south) during these cycles. Cracking on the east and west faces was marked by two separate observations. First, new cracks developed on the east and west faces at mid-depth of the specimen near the ends of the embedded connectors. And secondly, the vertical cracks over the outside connectors (near both extreme fiber faces) began to widen.

Cycles 34, 35, and 36 were performed at 5.0\(\Delta y_n\), (4.0 inches). At this displacement, all but one strain gage had ceased functioning. The average maximum load reached for cycle 34 was 28.5 kips. The maximum average load during cycle 36 was 23.1 kips. The maximum crack width between the base plate and the specimen during cycle 36 was approximately ½ inch. After the 34th cycle, all the LVDT’s at the base of the specimen were either removed or reset.

First spalling in the middle of the specimen occurred on the east face under negative displacement loading for cycle 34. Crumbling started at the exterior rows of connectors
in zone ‘A’ during cycle 36, and the vertical cracks over these connectors continued to widen.

Cycles 38, 39, and 40 were performed at 6.0Δy (4.8 inches). The maximum average load during cycle 38 was 27.2 kips, but fell to 21.4 kips for cycle 40. The maximum average separation between the base plate and the specimen had reached 0.7 inches during cycle 40. Spalling, and vertical and horizontal crack growth continued at the base of the specimen during these three displacement cycles.

Cycle 42 was performed at 7.0Δy, (5.6 inches). The maximum load for the positive displacement stroke was 24.7 kips. A load of only 13.5 kips was reached for the negative displacement stroke. During the negative displacement stroke, two connectors ruptured at loads of 5.0 and 5.1 kips. Later inspection revealed that two interior connectors had failed, and not connectors near the extreme fiber faces. Throughout the test, none of the exterior rows of connectors (near the extreme fiber faces) failed. The average crack width between the end plate and the specimen at maximum displacement was 0.83 inches. Upon the completion of the 42nd cycle, the specimen was displaced to the zero position and left over night.

Testing was resumed the next day at cycle 43. At this point in the test, it was decided to perform only one cycle at each displacement increment. No data acquisition was used past cycle 42. Cycle 43 was performed at a displacement of 8.0Δy (6.4 inches) with a displacement rate of 2 inches per minute. Two connectors ruptured during the positive displacement to 6.4 inches. Further inspection revealed that these two connectors were interior connectors. Further spalling and splitting along the exterior rows of connectors developed during this cycle. It became clear that a splitting plane was developing between the exterior rows of connectors on the tension flanges. This splitting plane allowed enough rigid body rotation such that the tensile load was transferred to the interior 3 rows of embedded connectors.
Cycle 44 was performed at a displacement of \(10.0\Delta y\) (8.0 inches) with a displacement rate of 4 inches per minute. Two connectors ruptured during the positive displacement, and two connectors ruptured during the negative displacement for this cycle.

Cycle 45 was performed at a displacement of \(12.0\Delta y\) (9.6 inches) with a displacement rate of 4 inches per minute. The last interior stud ruptured during the positive stroke of this cycle. At this point, all of the interior connectors were ruptured and a 300 pound load was required to displace the specimen to 9.6 inches. Figure 5.5 shows specimen H190 after testing was concluded.

Specimen H190 failed as a result of developing a split tension failure zone over the outermost row of connectors near the extreme fiber faces. It was clear that the outside rows of connectors had pulled out of the specimen by splitting the SIFCON. The failure first occurred on the southern most row of connectors. This explains the discrepancies in tensile strain reading for zone ‘A’ for positive and negative loading. As the specimen underwent negative loading, “slipping” developed on this splitting plane and the reinforcing could not develop its full tensile capacity. As slipping continued, the tensile load was redistributed to the central rows of connectors. Reinforcing congestion around the outer rows of connectors led to the split tension failure. There were two No. 6’s, two No. 4’s, and three ¾ inch embedded connectors across a 10 inch wide plane. Little space remained for SIFCON fibers. The result was a weakened plane that had the tensile capacity of the plain slurry.

5.3 Specimen H194

This specimen had a ¼ inch layer of plain slurry, without fibers, on its north face. Prior to testing, there was a longitudinal crack on the north face in zone ‘A’ over one of the connectors. This crack was most likely a shrinkage crack because there was negligible crack growth at this location throughout testing.
First cracking occurred on the north face on the positive stroke of cycle one. This cycle was at 25% of nominal yield load (6.55 kips). No cracking occurred on the south face until cycle 4, at which minimal cracking was observed. Cycle 4 was a load cycle at 50% of nominal yield load (0.5P_{yn} = 13.1 kips). Specimen separation from the base plate was noted, but was not measurable, during cycle 4.

Upon completion of cycle 6 (3rd cycle at 50% moment) the north face exhibited significantly more cracking than the south face. This was clearly a result of the ¼ inch layer of plain slurry on the north face. However, surface cracks on this face only penetrated about an inch into the depth of the specimen. Surface cracks on the south face showed less penetration into the depth of the specimen. Cycle 7 was the first load cycle at 0.75P_{yn} (19.65 kips). The average tensile strain at the critical section (in zone ‘C’) during cycle 7 was 1408 microstrain (0.1408%). The average displacement for cycle 7 was 0.415 inches.

Cycle 10 was a displacement controlled cycle at 0.553 inches (1.0\Delta_{yn}). This was calculated by dividing the displacement at 0.75P_{yn} by 0.75—and is defined as the nominal yield displacement. The average tensile strain at the critical section during cycle 10 was 1998 microstrain. However, average strain gage readings in zones ‘A’ and ‘B’ were 2354 and 2212 microstrain respectively. Thus, nominal yielding of reinforcing bars had developed in zones ‘A’ and ‘B’, but not at the critical section at zone ‘C’. Cracking on the south face was limited to about 10 small cracks at various locations. However, the north face exhibited ten times more cracking than the south face. Again this was the result of the ¼ inch layer of plain slurry on the north face. Very few cracks were seen on either of the weak axis faces (east and west faces).

Significant cracking was first observed on the south face during cycle 14. Few new cracks developed on the other faces of the specimen. Cycle 14 was the first displacement cycle at 1.5\Delta_{yn} (0.83 inches). The average tensile strain at the critical section was 2737
microstrain. The separation of the specimen at the base plate was less than 1/32th inch during cycle 14.

By the 18th cycle, the primary failure crack was being established in zone ‘C’ on both extreme fiber faces (north and south faces). However, these flexural cracks only penetrated about 4 inches into the depth of the specimen (See Figures 5.6a and 5.7a). Cycle 18 was the first cycle at twice the nominal yield displacement ($2\Delta_{yn} = 1.1$ inches). The average tensile strain at the critical section during cycle 18 was 9081 microstrain. Tensile strains in zones ‘A’ and ‘B’ exceeded 10000 microstrain. Crack opening between the base of the specimen and the end plate was approximately 1mm.

Cycle 22 was the first displacement cycle at $2.5\Delta_{yn}$ (1.38 inches). Figure 5.60a shows the cracking on the east face after cycle 22, and Figure 5.7a shows the cracking on the west face after cycle 22. The width of the major crack on the north face was approximately 1/32 inch. As a result of fiber bridging, this crack width on the south face was less defined. Cycle 22 resulted in the first clear migration of cracks into the depth of the specimen. Opening of the primary flexural crack on the north and south faces was also noted, and is shown in Figure 5.8a and 5.8b respectively. The average tensile reinforcing strain at the critical section in zone ‘C’ was 11620 microstrain. Strain gages at all other locations had ceased functioning.

Cycle 26 was a displacement cycle of $3\Delta_{yn}$ (1.66 inches). During this cycle, few cracks migrated into the depth of the specimen. However, good crack distribution started developing on the extreme fiber faces (north and south faces). Figure 5.10a shows the crack distribution on the south face after cycle 26. The major flexural cracks on the north and south faces grew to 0.1 and 1/32 inch respectively. All strain gages had ceased working during cycle 26. The separation of the specimen from the base plate grew to 1/16 inch during cycle 26. This crack closed upon reversed loading and subsequent testing did not increase base plate-to-specimen separation beyond 1/16 inch.
Cycle 30 was noted by the initiation of spalling on the north face of the specimen. At this point the main flexural crack on the north face had grown to a width of 1/8”. Cycle 30 was a displacement cycle of $4\Delta y_n$ (2.2 inches). Increased crack distribution on the south face was noted, while the main flexural crack opened slightly (See Figure 5.10b). Again, crack migrating into the depth of the specimen was minimal (See Figures 5.6b, and 5.7b).

Cracking of all four faces of the specimen after cycle 34 are shown in Figures 5.6c, 5.7b, 5.9, and 5.10c. This cycle was executed at $5\Delta y_n$ (2.77 inches), upon which it was evident that failure was occurring at mid-height of the specimen in zone ‘C’. Flexural cracks were penetrating into the depth of the member, but the load capacity had not decreased.

Cycle 38 was executed at $6\Delta y_n$ (3.32 inches). Few new cracks formed while existing cracks continued to open at the end of the connectors in zone ‘A’ and at the critical failure plane in zone ‘C’ (end of No. 6). Crack growth on the west face was unusual. The major crack seen on the left side of the specimen in Figure 5.7b starts at the extreme fiber and migrates downward and inward to mid-depth of the specimen. During cycle 38, this crack stopped growing down into the No. 5 reinforcing in zone ‘B’ and abruptly moved upward away from the No. 5’s. Apparently the No. 5’s bridged the cracking plane and forced the primary failure crack back up towards zone ‘C’.

The last repetition at $6\Delta y_n$ was cycle 40. On the negative displacement stroke, one of the No. 6 reinforcement bars ruptured in the south face of the specimen. Figures 5.10d and 5.6d show the south and east faces of the specimen after failure.

The reinforcing in this specimen was designed to yield throughout the specimen prior to failure. As a consequence, it was expected that good crack distribution would result within the specimen. Strain gage readings confirmed yielding of the main reinforcing, and good crack distribution was observed during testing. Thus, the specimen behaved as
expected. A good distribution of flexural cracks developed in several zones on the specimen prior to failure on a critical plane in zone ‘C’. Shear deformation and shear cracking throughout testing was negligible. Additionally, none of the flexural cracks developed into flexural shear cracks with crack openings more than 1/16 inch.

5.4 Specimen H226

During specimen injection, a pressure of 60 psi was exerted on the north and east sides of the forms. Consequently, a ¼” layer of unreinforced slurry resulted on the north and east faces. As testing progressed, cracks that formed on these faces were not bridged by fibers, and crack width measurements on these faces could be attained.

First cracking developed only on the north and east faces during cycle 1. Cycle 1 was the first load cycle at 0.25Pyn (6.2 kips). Crack distribution continued on these two faces until cycle 7. Cycle 7 was the first load cycle at 0.75Pyn (18.5 kips). No cracking was observed on the south and west faces until cycle 7. The average displacement for cycle 7 was 0.425 inches and the average reinforcing tensile strain at the critical section in zone ‘C’ (end of 90° bent No. 6’s) was 1493 microstrain (0.1293% strain).

Cycle 10 was the first cycle at nominal yield strain in the reinforcing (1.0∆yn). The nominal yield displacement was 0.6 inches and was calculated according to the procedure described in Chapter 3. The average load on cycle 10 was 22.4 kips. The average tensile strain at the critical failure plain (zone ‘C’) was 2179 microstrain. The average tensile strain at the failure planes in zones ‘A’ (end of connectors) and ‘B’ (end of No. 5’s) was 2208 and 2012 microstrain respectively. Because nominal yield strain of Grade 60 reinforcing is 2070 microstrain, almost all the specimen achieved nominal yielding during cycle 10. Cracking across the extreme fiber faces (north and south faces) occurred at cycle 10. In addition, diagonal cracking to about half of the specimen depth developed on the east face during cycle 10. Crack widths between the bottom end plate and the specimen measured approximately 1/32 inch. It is significant to note that diagonal
cracking on the east face during the positive displacements was much more pronounced than the diagonal cracking during the negative displacements.

Cycle 14 was the first displacement cycle at $1.5\Delta_{yn}$ (0.9 inches). The force required for this displacement was 30 kips. The average tensile strain at the critical failure plane in zone ‘C’ was 2897 microstrain. The average tensile strain in zones ‘A’, and ‘B’ was 2828 and 2794 respectively. Again, the specimen had reached nominal yielding throughout the section. Many new, small cracks developed on the south, east, and west faces during cycle 14. Diagonal cracking lengthened into the mid depth of the specimen on the west face during cycle 14, and was considerably worse for reversed loading. The cracks developed to mid depth during positive load, but extended past mid depth upon negative loading. This is illustrated upon close inspection of Figure 5.12a.

During cycle 14 there was a distinct correlation between strain gage readings and the cracking pattern. As stated above, diagonal cracking on the west face was noticeably worse for negative loading. Additionally, strain gage readings in zone ‘A’ (end of connectors) were markedly different between the positive and negative strokes of cycle 14. The reinforcing strain during the positive stroke was 3354 microstrain but fell to only 2301 microstrain during reversed loading. The main reinforcing was not fully effective during reversed loading. Either the specimen was starting to reveal some asymmetry, or the south face main reinforcing was starting to slip during reversed loading. It is likely that some slipping in the threads of the reinforcing anchorage hardware. Unlike specimen H194, no epoxy was used on the reinforcing anchorage threads for specimen H226. The effect of reinforcing slip was a decrease of the effective member depth, resulting in an increase in shear stress on the specimen. This increased shear stress caused more severe diagonal cracking under reversed loading.

During cycle 18 ($2.0\Delta_{yn}$ and $P = 32.5$ kips), the diagonal cracking on the east and west faces migrated to 2/3 of the full depth of the specimen and became the distinctive feature in the cracking patterns. Numerous new cracks developed on the south and west faces
during cycle 18. However, few new cracks developed on the north face during cycle 18. This was further indication of specimen asymmetry. At this stage of testing, the majority of the cracks had developed within zones ‘B’, ‘C’, and ‘D’. Flexural cracking on the south face appeared to be concentrated at the critical failure plane in zone ‘C’. Once again, the strain gage readings between positive and negative load varied between 4000 and 40000 microstrain. However, unlike cycle 14, the cracking patterns on the weak axis faces (see Figures 5.11a, and 5.12a) indicated similar specimen distress for positive and negative loading. The least tensile strain measure during cycle 18 was 4017 microstrain, occurring in zone ‘A’ under negative loading. This is approximately double the nominal yield strain of the reinforcing. Separation of the specimen from the base plate remained approximately 1/32 inch. Specimen cracking during cycle 18 is shown in Figures 5.11a, 5.12a, 5.13a, and 5.14a for the east, west, north, and south faces respectively.

Cycle 21 was a 0.75$\Delta y_n$ (0.425 inches) displacement stroke to determine specimen softening. The average load required for cycle 21 was 12 kips. This results in a 25% softening from the initial displacement of 0.75$\Delta y_n$.

Cycle 22 ($2.5\Delta y_n=1.5$ inches at $P = 32.5$ kips) resulted in opening of the diagonal cracks on the east and west faces. However, the cracking patterns for the two faces differed slightly as a result of the ¼ layer of plain slurry covering the east face. Two separate diagonal cracks appeared in zones ‘C’ and ‘D’ on the west face for both the positive and negative displacement increments. One the east face, however, there was only one primary diagonal crack that developed in each direction (compare Figures 5.11b and 5.12b). Many new flexural cracks developed on the south face during cycle 22. Most of these cracks seemed to develop at the critical section in zone ‘C’. Flexural cracks on the north face widened with few new cracks developing during cycle 22. All strain gages had ceased functioning during cycle 22. The separation between the specimen and the base plate was approximately 3/32 inch during cycle 22.
Cycle 26 was the first displacement cycle at 3.0\(\Delta y_n\) (1.8 inches and 34.8 kips). First spalling, on the south and west faces was observed during this cycle. Figures 5.11b, 5.12b, 5.13b, and 5.14b show the cracking on each face after cycle 26. Many new cracks developed on the north face with the primary flexural crack widening to 1/32 inch. Few new cracks developed on the east face, and the primary diagonal crack opened to 1/16 inch during the positive displacement stroke. Crack distribution continued on the west face with numerous diagonal cracks developing from the two primary diagonal cracks.

Cycle 30 was distinguished by limited new crack formation, but with increased crack opening and growth. Figure 5.15 shows a 3/32 inch flexural crack in the north face and a 3/32 inch diagonal crack in the west face. Spalling initiated on the north and east faces during cycle 30. The average load for cycle 30 was 36.7 kips at a displacement of 2.4 inches.

The cracking after cycle 34 is shown in Figures 5.11c, 5.12c, 5.13c, and 5.14c. This cycle was performed at 5.0\(\Delta y_n\) at an average maximum load of 35.8 kips. No new cracks developed in the specimen during this cycle and beyond. Severe spalling began on the extreme fiber faces (north and south) as spalling at the diagonal cracks on the weak axis faces continued to worsen. A flexural crack width of ¼” was measured on the south face of the specimen, while a diagonal crack width of 3/16” was measured on the east face.

Spalling and opening of cracks continued during cycle 38. Cycle 38 achieved an average load of 31.6 kips at a displacement of 6.0\(\Delta y_n\) (3.6”). This was the first decrease in specimen load. Flexural and diagonal cracks continued to open throughout cycles 38 through 40. During cycle 39, bulging of the curvature clamps was observed. It appeared that the clamps were acting as external shear reinforcement. Strength degradation of the specimen was evident because the average load during cycle 40 (last cycle at 6.0\(\Delta y_n\)) was only 28 kips. Thus, the specimen exhibited a strength degradation of over 11% within
two cycles at the same displacement increment. It was clear that the diagonal cracks were indicative of a shear failure in the specimen.

The curvature clamps were loosened prior to continuing to cycle 40 so as not to damage the clamps. The maximum average load reached for cycle 42 was 21.8 kips at a displacement of \( 7.0 \Delta_{yn} \) (4.2”). The curvature clamps were so misaligned that one of the LVDT rods slipped off of its contact point during cycle 20. Consequently, no data was plotted for this specimen beyond this point. No new cracks developed during cycles 42, 43, and 44. Opening of existing cracks and continued spalling marked these three cycles. All instrumentation was removed prior to cycle 44. Figures 5.15d, 5.12d, 5.13d, 5.14d, and 5.16 show the specimen after cycle 44. Upon reaching a displacement of –4.2 inches, the specimen was displaced to –5.5 inches at which time a tensile reinforcing bar ruptured. Testing was halted at this point.

Ultimate failure of specimen H226 can be characterized as a shear failure. As indicated above, the reinforcing tensile strains during negative loading were significantly less than tensile strains under positive loading. This seemed to indicate some hardware anchorage slip in the main reinforcing on the south face of the specimen. However, if the south face reinforcing wasn’t as effective as the north face reinforcing, the specimen should have failed as a result of negative shear and the failure crack would have sloped the opposite direction. The ultimate failure of the specimen was a result of POSITIVE shear, and not negative shear. Thus, it is likely that shear slipping during positive loading created the asymmetric reinforcing tensile strains exhibited by the specimen. Before the south face reinforcing could fully develop to resist negative loads, the slip that occurred during positive load must be recovered. The result is less tensile strain and lower loads during negative loads—which is exactly how the specimen behaved.

5.5 Specimen H118D
Specimen H118D was constructed using Dywidag reinforcing. The nominal yield stress of Dywidag reinforcing bars is approximately 141 ksi as determined from tensile tests. This corresponds to a yield strain of 4860 microstrain (0.4860 % strain). The south face of the specimen contained a ¼ inch layer of plain slurry. This fact must be considered when interpreting the south face cracking.

No cracking was observed until cycle 4. Cycle 4 was the first cycle to one-half the nominal yield load (0.5P_{yn} = 16.2 kips). Cycles 4 through 9 were noted by minimal flexural crack formation on the extreme fiber faces (north and south), while significant cracking developed on the interior of the east and west faces. Cycle 7 was a cycle at 0.75P_{yn} (24.5 kips). The average displacement for this cycle was 0.72 inches. The average tensile reinforcing strain at the critical failure plane in zone ‘B’ (end of the No. 5’s) during cycle 7 was 2219 microstrain. Tensile reinforcing strain in zone ‘A’ (end of connectors) was 3190 microstrain. The nominal yield displacement determined as described in chapter 3 was 1.52 inches. However, upon charting the previous test data from this specimen, the actual yield displacement appeared to be around 1.2 inches. Thus, the nominal yield displacement (\( \Delta_{yn} \)) was set at 1.2 inches and was not determined according to the procedure outlined in chapter 3.

The first notable cracks occurred during cycle 10. Cycle 10 was a displacement cycle to 1.0\( \Delta_{yn} \) (1.2 inches at 32.4 kips). Cycle 10 resulted in a good distribution of cracks on the south face, and almost no cracking on the north face. There was no evidence of yielding on these extreme fiber faces, but the average tensile reinforcing strain at the critical failure plane in zone ‘B’ was 3362 microstrain (0.3362% strain). The average reinforcing strain in zone ‘A’ was 3930 microstrain. A good crack pattern developed on the weak axis faces (east and west faces) with more cracks developing near mid depth of the specimen. The separation between the base plate and the specimen had grown to 3/32 inch.
Cycle 14 was noted by a ¼ inch gap between the base plate and the specimen. This cycle was a displacement cycle at $1.5\Delta_{yn}$ (1.8 inches at 36.7 kips). The strain gages located in zone ‘A’ had ceased working. The average tensile strain in the reinforcing in zone ‘B’ was 4200 microstrain—still less than nominal yield strain. The most significant result of cycle 14 was the development of longitudinal cracks over the first row of connectors at the base of the specimen on the east and west faces (4.5 inches inside the extreme fiber faces). These longitudinal cracks can be seen in Figures 5.17a and 5.18a and were parallel to the embedded connectors. Few new cracks developed on the east and west faces, but crack widening was observed on these faces during cycle 14. Figures 5.17, 5.18, 5.19, and 5.20 show the cracking patterns after cycle 14.

Cycle 18 was noted by the initiation of popping sounds heard while unloading the specimen after cycle. Further testing revealed that these sounds were the result of deformation of the female threads of the Dywidag hardware. As the bars were loaded, the male threads of the bars would bear against the female threads of the hardware. As loading continued, the hardware threads began to deform as the male bar deformations started biting into the female threads of the hardware. Upon unloading, the popping noises occurred when the male and female threads released from each other.

Cycle 18 was a displacement cycle at $2.0\Delta_{yn}$ (2.4 inches at 39.1 kips). A strain gage reading of over 5000 microstrain was taken prior to the all gages returning erroneous data. Thus, nominal yielding was occurring in the specimen. New cracks developed during cycle 18 at the base of the specimen. These cracks lead to damage and spalling on the extreme fiber faces within three inches of the base plate. A flexural crack width of 3/8 inch was measured on the north face in zone ‘B’. No other cracking was notable. The specimen separation at the base had grown to approximately 5/16 inch during cycle 18.

Cycle 22 was a displacement cycle at $2.5\Delta_{yn}$ (3.0 inches at 39.1 kips). Cycle 22 was noted by the opening of the longitudinal cracks over the first row of connectors on the
east and west faces (see Figures 5.17 and 5.18). These cracks started spalling during
cycle 26. This was visible evidence of the initiation of the direct shear failure plane over
the first row of embedded connectors. Few new cracks developed on the specimen as
these cracks began to open. The only notable flexural crack on the south face opened to
approximately 1/64 inch. Base separation grew to approximately ½ after cycle 26.

One the first positive displacement of cycle 30, reinforcing bar rupture occurred at a
displacement of 3.8 inches. This exceeded the prior maximum displacement by 0.2
inches. The Dywidag bar rupture occurred just outside of the welded Dywidag hardware
on the base plate of the specimen. Subsequent loading resulted in the formation of a
primary failure plane over the second row of connectors on the east and west faces at the
base of the specimen. Figure 5.21 shows the specimen after testing and clearly
illustrated this vertical failure plane.

Although the reinforcing reached a yielding strain, the failure planes over the connectors
in zone ‘A’ lead to premature failure of the specimen. As the Dywidag bars slipped
inside the anchor hardware, the bars were not effectively developed. As a result, strain
capability was no longer an accurate assumption. More significantly, however, was how
the tensile force was transmitted the end plate. As the bars slipped inside the hardware
anchor, some of the tensile force to be carried by the Dywidag hardware was then
transferred to the adjacent SIFCON matrix through bond stresses. This load was then
transmitted to the first row of connectors and a shear plane developed over these
connectors to resist this force. Eventually, the force required to resist the bar tension
exceeded the shear strength of this shearing plane. The effect of the partially developed
Dywidag bars was to decrease the effective section depth—leading to higher compression
stresses and spalling in the compression zones. This explains the spalling observed
during cycle 18.

5.6 Specimen H194HS
Specimen H194HS was constructed from heat-treated Grade 60 reinforcing steel. The mechanical properties of this steel were close to that for Grade 75 reinforcing. Grade 75 reinforcing steel has a slightly higher yield strain (2586 microstrain) than Grade 60 reinforcing steel (2069 microstrain). Thus, it was anticipated that cracking at yield load would be more extensive than specimens constructed with standard Grade 60 reinforcing. It is also important to note that the east face of specimen H194HS contained a ¼ to ½ inch layer of slurry without fibers.

Figures 5.22 through 5.25 show the various crack patterns at different stages of loading. Cycle 7 was the first load cycle at 75% of the nominal yield load (23.6 kips at 0.52 inches). After cycle 7, more cracking was evident on the east face as a result of the layer of plain slurry. The north face also showed a distinct lack of cracking as compared to the south face. The average tensile strain in the reinforcing at the critical sections in zone ‘C’ was 1780 microstrain (0.1780% strain). Tensile reinforcing strain in zones ‘A’ and ‘B’ were 2180 and 1740 microstrain respectively.

Cycle 10 was the first displacement cycle at nominal yield displacement ($\Delta_y$). The displacement for cycle 10 was determined to be 0.74 inches according to the procedure described in Chapter 3. The average load required for this displacement was 31.45 kips. The average reinforcing strain in zone ‘C’ was 2545 while the average tensile reinforcing strain for zones ‘A’ and ‘B’ was 3162 and 2414 respectively. Thus, the nominal yield load was reached for zones ‘A’ and ‘C’. Distributed cracking first became evident during cycle 10 with a crack spacing of approximately ½ inch. However, this was only true for the south face of the specimen (similar to cycle 14 as shown in Figure 5.25b). Diagonal cracking initiated on the weak axis faces during cycle 10. At this point the separation of the specimen from the base plate was approximately 1mm.

Cycle 14 is the first cycle at a displacement of 1.11 inches (1.5$\Delta_y$ at 38.3 kips). At this point, the strain gages at the end of the connectors ceased working, but the gages in zones ‘C’ and ‘B’ returned strains of 3182 and 3128 microstrain respectively. Thus, all the
reinforcing in the specimen was undergoing nominal yield loading. During this cycle, crack distribution continued on the south face, and crack distribution initiated on the north face. More diagonal cracks developed on the weak axis faces (east and west faces), and the specimen separation from the base had grown to approximately 2 mm. It is important to note that the load required for negative displacement (tension on the south face) for cycle 14 was approximately 1.5 kips greater than that required for positive displacement.

A well defined flexural crack pattern developed during cycle 18. Cycle 18 was the first displacement cycle to 1.48 inches (2.0\(\Delta_{yn}\) at 42.1 kips). All strain gages had ceased working. The cracking pattern consisted of a distribution of cracks, spaced approximately ½ inches apart, on the extreme fiber faces and the west face. Crack spacing on the east face was dominated by the layer of plain slurry, resulting in a 3 to 5 inch crack spacing. The south face exhibited a primary flexural crack in Zone ‘D’ that was measured at 0.01 inches wide. A 1/8\(^{th}\)-inch crack opened between the specimen and the base plate during the maximum positive and negative strokes. Vertical cracks were observed over the outer most row of connectors at the base of the specimen.

Few new cracks developed during cycle 22 (2.5\(\Delta_{yn}\) at 44.85 kips). This displacement cycle was noted by the opening of existing flexural cracks on the extreme fiber faces and the opening of flexural and diagonal cracks on the east and west faces. The 1/8\(^{th}\) inch separation at the base was maintained during the positive displacement, but opened to ¼ inch during the negative displacement. The unbalanced crack patterns and the difference in base plate separation between the extreme fiber faces indicate that some initial strain was induced in the reinforcing during specimen fabrication prior to placing the SIFCON. Figures 5.22c through 5.25c show the specimen after cycle 22.

During cycle 26 (3.0\(\Delta_{yn}\) at 44.45 kips) the main flexural crack in zone ‘D’ on the south face opened to 0.02 inches. Specimen separation at the base on the north side grew to 3/8 inch, but remained at ¼ inch on the south. Few new cracks developed during cycle 26.
This cycle was noted by opening of existing flexural crack on the extreme fiber faces and opening of existing flexural and diagonal cracks on the weak axis faces.

Spalling initiated during cycle 30 \((4.0\Delta y_n \text{ at } 47.85 \text{ kips})\) on the south face at the primary flexural crack in zone ‘D’. The crack width was measured at 0.06 inches. The specimen was clearly starting to fail at this location. The separation of the specimen at the base was approximately 3/8 inch for both the positive and negative strokes. As a result, the lower LVDT measuring this crack had reached its limiting stroke and was removed.

Cycle 34 \((5.0\Delta y_n \text{ at } 46.85 \text{ kips})\) was noted by further deterioration of the specimen. Spalling initiated on the north face with a primary flexural crack opening of 0.03 inches. Spalling worsened on the south face as the main flexural crack opened to 3/16 inches, and a flexural failure seemed imminent (see Figure 5.26). Spalling also initiated on the weak axis faces as the flexural cracks opened and became more inclined toward the center of the specimen.

Figures 5.23d, 5.24d, and 5.25d show the specimen after cycle 38 \((6.0\Delta y_n \text{ at } 43.65 \text{ kips})\). Cycle 38 resulted in a ¼ inch separation of the specimen at the base plate. The main flexural crack on the north face opened to 1/8\(^{\text{th}}\) inch. However, flexural crack on the south face was over ¼ inch. This can be seen in Figures 5.24d and 5.25d.

Cycle 42 \((7.0\Delta y_n \text{ at } 76.4 \text{ kips})\) resulted in the buckling of the reinforcing on the south face of the specimen. At this point all of the instrumentation was removed and the test was halted. Figure 5.27 shows the specimen after cycle 42.

Specimen H194HS failed due to buckling of compression reinforcing. As a result of the higher yield strain of the reinforcing, cracking at the onset of reinforcing yield was more extensive than in other specimens. In addition, as flexural cracks grew into the depth of the specimen, they grew more diagonal than those exhibited by previous specimens. This
diagonal cracking, together with more extensive cracking at yield, led to a loss of confinement—and thus, buckling of compression reinforcement. Never before had compression buckling of reinforcing been observed in a SIFCON flexural specimen. Thus, as higher strength reinforcing is used, extensive cracking may decrease in specimen’s stability at yield load. Buckling of the compression reinforcing prevented this specimen from developing its full ductility and energy absorption. Appropriately placed stirrups could have prevented buckling of the compression reinforcing.

5.7 Specimen H146D

Specimen H146D was constructed using ASTM A722 Dywidag reinforcing steel. As a result, extensive specimen cracking was anticipated prior to specimen yielding. It is important to note that the west face of the specimen consisted of a ¼ inch layer of slurry without fibers. Thus it was expected that a wider crack spacing would occur on the west face.

Figures 5.28 through 5.31 show the various crack patterns at different stages of loading for the specimen. Cycle 4 was the first load cycle at 50% of the nominal yield load. A good flexural crack pattern developed on the extreme fiber faces during cycle 4. This occurred in zone ‘C’ and ‘D’ only. Flexural cracks also developed in zones ‘C’ and ‘D’ on weak axis faces (east and west faces) during cycle 4. However, the cracking on the weak axis faces was not as well defined as on the extreme fiber faces.

Cycle 7 was the first load cycle at 75% \( P_y \) (28.5 kips) occurring at a displacement of 0.725 inches. The average tensile reinforcing strain at the critical section in zone ‘D’ was 2925 microstrain (0.2925%). The cracking after cycle 7 is shown in Figures 5.28a through 5.31a. Cycle 7 resulted in a continued distribution of flexural cracks on the extreme fiber faces with the south face exhibiting twice as many cracks as the north face. Again, these cracks were almost exclusively in zones ‘C’ and ‘D’. Cracking on the east face was also limited to zones ‘C’ and ‘D’, while cracking on the west face occurred in

Assuming an elastic modulus of 29000 ksi and a yield stress of 130 ksi, the nominal yield strain of the Dywidag reinforcing was determined to be 4500 microstrain. Using the method described in Chapter 3, the nominal yield displacement was determined to be 1.12 inches ($\Delta_{yn}$). The actuator was switched to displacement based loading, and cycle 10 was the first displacement cycle at the nominal yield displacement of 1.12 inches. Upon reaching the +/- 1.12 displacement, one strain gage at the critical section ceased working and the other returned a strain of 3640 microstrain. After cycle 10, a primary flexural failure crack formed in zone ‘D’ on both extreme fiber faces. These were not new cracks, but existing cracks that opened enough to be defined as the primary failure cracks. Flexural crack spacing on the extreme fiber faces was approximately ½ inch. Minor spalling was also observed at these locations. While the flexural cracking remained inside zone ‘B’ and ‘D’ on the north face, cracking on the south face distributed into all zones. At the same time, a very good distribution of cracks, approximately 1.5 inches long, formed on the weak axis faces. These formed in zones ‘B’, ‘C’, and ‘D’, and became inclined as they grew into the center of the specimen. Separation of the specimen from the base plate widened to 1/16 inch during cycle 10.

Cycle 14 is the first cycle at a displacement of 1.68 inches (1.5$\Delta_{yn}$). At this point, all strain gages had ceased working. During this cycle, minimal crack distribution occurred on the south face. The north face exhibited crack opening, accompanied by additional spalling, at the primary flexural crack with no additional crack distribution. This difference is evident by comparing Figures 5.30b and 5.31b. Diagonal cracking continued to develop on the weak axis faces (east and west faces), with crack spacing of approximately ¾ inch. The specimen separation from the base had widened to approximately 1/8 inch.
Cycle 18 ($2.0\Delta y_n = 2.24''$) was noted by continued crack opening, and extensive spalling. **Figure 5.32** shows the cracking and spalling during cycle 18. The primary failure crack had grown into the east and west faces, and spalling initiated over these cracks. Crack opening continued on the extreme fiber faces, and as seen on **Figure 5.32a**, failure seemed imminent on the primary flexural crack on the north face. The separation at the base of the specimen widened to 5/32 inch during cycle 18.

No new cracks developed during cycle 22 ($2.5\Delta y_n = 2.8''$). This displacement cycle was noted by the opening of existing flexural cracks all around the specimen. Primary flexural crack opening proceeded, as all the specimen ductility was isolated to the primary flexural crack. **Figures 5.28c, 5.29c, 5.30c, and 5.31c** show the primary failure crack on all sides of the specimen. The crack width on the extreme fiber faces was estimated to exceed ¼ inch, and the load required for cycle 22 decreased 11 kips from that required for cycle 18. These two facts indicate that the SIFCON fiber interlock was lost and the SIFCON was no longer effective in resisting tensile loads. Reinforcing bar rupture occurred at a 2.4 inch displacement during cycle 24. Cycle 24 was the last cycle at 2.8” ($2.5 \Delta y_n$). The specimen, at completion of testing, is shown in **Figure 5.33**.

Of all the specimens that failed within zones ‘C’ and ‘D’, specimen H146D achieved the least distribution of damage on the extreme fiber faces. The explanation for this lies in the embedded plates used to anchor the Dywidag bars. The end plates used to anchor the discontinuous Dywidag bars created an abrupt discontinuity in the center of the specimen. These plates are clearly shown **Figures 5.34 and 5.35**. **Figures 5.34 and 5.35** show the specimen after it has been broken apart at its primary failure crack in zone ‘D’. As a result of the compression field created by the localized anchorage stresses, very little distributed cracking was achieved on the extreme fiber faces up to specimen failure. If different anchorage details were used (in lieu of embedded end plates), this specimen would have exhibited more distribute yielding and damage.
Figure 5.1: Specimen V44 at conclusion of testing.

Figure 5.2: Specimen V44 failure plane in zone ‘A’.
Figure 5.3: Bottom after separation of specimen V44 at failure plane.

Figure 5.4: Specimen H190 cracking at cycle 18.
Figure 5.5: Specimen H190 after testing.
Figure 5.6: Specimen H194 east face cracking at various stages of loading.
Figure 5.7: Specimen H194 west face cracking at various stages of loading.

a) West face cycle 22
b) West face cycle 34

Figure 5.8: Specimen H194 primary flexural crack.

a) North face primary flexural crack.
a) South face primary flexural crack.
Figure 5.9: Specimen H194 north face cracking after cycle 34.
Figure 5.10: Specimen H194 south face cracking at various stages of loading.
Figure 5.11: Specimen H226 east face cracking at various stages of loading.
Figure 5.12: Specimen H226 west face cracking at various stages of loading.
Figure 5.13: Specimen H226 north face cracking at various stages of loading.
Figure 5.14: Specimen H226 south face cracking at various stages of loading.
Figure 5.15: Specimen H226. Close up of cracks during cycle 30.

a) Flexural Crack on north face

b) Diagonal crack on east face
Figure 5.16: Specimen H226. South and west faces at end of test (cycle 44).
Figure 5.17: Specimen H118D west face cracking at various stages of loading.

Figure 5.18: Specimen H118D east face cracking at various stages of loading.
Figure 5.19: Specimen H118D north face cracking at various stages of loading.

Figure 5.20: Specimen H118D south face cracking at various stages of testing.
Figure 5.21: Specimen H118D after testing.

a) East face after testing

b) West face after testing
Figure 5.22: Specimen H194HS east face cracking at various stages of testing.
Figure 5.23: Specimen H194HS west face cracking at various stages of testing.
Figure 5.24: Specimen H194HS north face cracking at various stages of testing.
Figure 5.25: Specimen H194HS south face cracking at various stages of testing.

a) South face at cycle 7  
b) South face at cycle 14

c) South face at cycle 22  
d) South face at cycle 38
Figure 5.26: Primary flexural crack. SPECIMEN H194HS, south face, zone ‘D’, cycle 34.

Figure 5.27: Specimen H194HS at end of testing.

a) East and north faces  b) West & south faces
Figure 5.28: Specimen H146D east face cracking at various stages of testing.

a) East face at cycle 7

b) East face at cycle 14

c) East face at cycle 22
Figure 5.29: Specimen H4-10-146D west face cracking at various stages of testing.

a) West face at cycle 7

b) West face at cycle 14

c) West face at cycle 22
Figure 5.30: Specimen H146D north face cracking at various stages of testing.

a) North face at cycle 7

b) North face at cycle 14
c) North face at cycle 22
Figure 5.31: Specimen H146D south face cracking at various stages of testing.

a) South face at cycle 7

b) South face at cycle 14

c) South face at cycle 22
Figure 5.32: Specimen H146D cracking at cycle 18.

a) North face primary failure crack
b) South face primary failure crack
c) East face primary failure crack
d) East face primary failure crack
Figure 5.33: Specimen H146D after testing.

Figure 5.34: Broken halves of specimen H146D after testing.
Figure 5.35: Close up of the two halves of Specimen H146D after failure.

(a) Top half

(b) Bottom half

Figure 5.35: Close up of the two halves of Specimen H146D after failure.
6 EXPERIMENTAL RESULTS

This chapter will present the results of the experimental program. As outlined in Chapter 4, tension tests on reinforcing steel and compression tests on SIFCON cylinders were performed to provide basic material properties for the SIFCON hinges. The results from these tension and compression tests are presented in this chapter. However, most of this chapter focuses on the results of the 2/3 scale hinge testing. In summary, the following results are reported in this chapter.

1) Reinforcing steel tension test results.
   a) No. 3’s from Ameristeel, Grade 60
   b) No. 4’s from Ameristeel, Grade 60
   c) No. 5’s from Ameristeel, Grade 60
   d) No. 6’s from Ameristeel, Grade 60
   e) No. 6’s from Dayton-Richmond, Grade 60
   f) No. 7’s from Dayton-Richmond, Grade 60
   g) No. 6 High Strength Thermex bars from Ameristeel
      (Heat treated Grade 60)
   h) 5/8” diameter Dywidag bars, ASTM A722

2) SIFCON compression specimen test results.
   a) Standard 4”x 8” cylinders.
   b) 4”x 4”x 8” prisms
   c) 3”x 3”x 8” prisms

3) Results for the 2/3 scaled quasi-static hinge testing.

6.1 Reinforcing Steel Test Results

Two tension tests were conducted on each of the eight types of reinforcing by an independent laboratory. Yield, ultimate, and strain hardening stresses and strains are reported in Chapter 6. All of the bars exhibited very similar behavior. There was a linear elastic initial stress - displacement portion, followed by a flat yield plateau, followed by
strain hardening. Strain measurements were terminated when the instrumentation was removed prior to bar rupture.

Yield stress ($f_y$), stress at ultimate strength ($f_u$), and percent elongation were experimentally determined. Yield strain ($\varepsilon_y$) and strain at onset of strain hardening ($\varepsilon_{sh}$) are determined according to the method outlined in Chapter 3, section 3.6. Figure 6.1 shows a typical stress versus displacement curve, and how the strain values were obtained. The results were averaged for the two specimens and reported in Table 6.1. It is important to note that the strain values reported are adjusted for the apparent initial slip in the wedge grips of the testing machine.

### 6.2 SIFCON Compression Specimen Test Results

Plane slurry and SIFCON compression cylinders were cast simultaneously with each hinge. As discussed in Chapter 4, fiber orientation in compression specimens must mirror that of the hinges to properly indicate SIFCON material behavior. There were two types of compression specimens used in this research program. These are vertically filled and cast cylinders, and horizontally filled and cast prisms. All compression specimens used for this program were horizontally filled cylinders or vertically filled prisms unless noted otherwise on the graphs. Placing fibers in vertical cylinder molds tended to orient the fibers more perpendicular to the loading direction. Placing fibers horizontally in the prisms tended to orient the fibers parallel to the loading direction. As a result of the different fiber orientation, the cylinders and prisms behave quite differently. Figures 6.3, 6.8, and 6.9 show typical stress versus strain curve for cylinders and prisms using the same fiber volume fraction and slurry strength. The differences clearly illustrate the effect of fiber orientation.

Typical SIFCON compression specimens used in this program are shown in Figure 6.2 after testing. This figure shows a standard 4”x8” cylinder, a 4”x4”x8” prism, and a 3”x3”x8” prism. Cylindrical specimens do not exhibit a clear diagonal failure plane.
Instead they bulge as strains increase beyond 1%. On the contrary, prismatic specimens exhibit diagonal failure planes when loaded beyond 1% strain. Bulging is observed after sliding develops on the failure planes.

As discussed above, fiber orientation in compression specimens affects the compression behavior. Horizontally cast prismatic compression members are considered to more accurately represent hinge compression zone behavior. However, prismatic SIFCON compression specimens were only cast for three specimens. Thus, to predict the compression behavior of the other four specimens, a correlation was drawn between the compression curve for horizontally cast 4” x4” x 8” prisms and vertically cast cylinders. This correlation, instead of the actual cylindrical compression test data, was used to predict the compression behavior of all hinges when prismatic compression members were not cast (all hinges except V44, H194HS and H146D).

Compression stress versus strain curves are shown for all the specimens in Figure 6.3 through Figure 6.9. Two cylinders were tested for each hinge, denoted ‘A’ and ‘B’, except when equipment difficulty was encountered and only one curve is shown. The waviness seen in some of the curves is also the result of difficulties encountered with the equipment. Many cylinders exhibited less than 20% strength reductions at strains of 15%. However, all the prismatic specimens showed significant softening, which agrees with previously reported results of SIFCON when loaded parallel to fiber orientation [Homrich and Naaman, 1987].

The results for specimen H226 and H118D are interesting in that these compression specimens exhibited no strength softening (see Figures 6.6 and 6.7). The maximum stresses continued to increase as strains exceeded 10%. It is interesting to compare compression data of hinge H118D to hinge H146D. The compression specimens for H146D (shown in Figure 6.9) exhibit the softening reported in previous research while the results for hinge H118D show no softening at all. These were the same mixes with only 1% difference in fiber volume fraction and 2 days difference in age at testing. There
is no known explanation for this behavior. The difference between vertically filled prisms and horizontally filled prisms is shown in Figure 6.3.

Table 6.2 shows summary of cylindrical SIFCON compression specimens. Table 6.3 shows a summary of prismatic SIFCON compression specimens. The data includes plane slurry strength at testing ($f_c$), maximum stress and strain ($\sigma_{max}$, $\sigma_{pmax}$, $\varepsilon_{max}$), inflection point stress and strain ($\sigma_{infl}$, $\varepsilon_{infl}$), plateau stress ($\sigma_{plat}$), and initial modulus (E). There was only one correlation between cylindrical specimens and horizontally cast prismatic specimens used. That correlation is that prism test averaged 20% less maximum compressive strength than cylinder tests (see Figures 6.2, 6.8 and 6.9). All other data was taken from the average of the prism tests. The results of the correlation and average are shown below.

1) $\sigma_{pmax} = 0.8 \sigma_{max}$ of cyl test = maximum compression stress from prism test
   [Correlating prism to cylinder test]
2) $\varepsilon_{max} = 0.7 \%$ = strain at max stress
   [Average of prism tests (See Figure 6.8 and 6.9)]
3) $\sigma_{infl} = 0.87 \sigma_{max}$ = stress at inflection point
   [From prism tests on H194HS & H146D]
4) $\varepsilon_{infl} = 2.0 \%$ = strain at inflection point
   [Average of prism tests (See Figure 6.8 and 6.9)]
5) $\sigma_{plat} = 2/3 \sigma_{max}$ = plateau stress
   [From prism tests]
6) $\sigma_{tmax} = 1/3 \sigma_{plat}$ = maximum tensile stress
   From Homrich and Naaman [1987]
7) $\varepsilon_{tmax} = 0.6 \%$ = strain at maximum tensile stress
   From Homrich and Naaman [1987]
The compression stress strain curve for specimen H226 in Figure 6.6 did not reach a maximum stress. Therefore, another correlation was required. The maximum compression stress for horizontally filled SIFCON prisms averaged approximately 10% lower than the compression strength of the unreinforced slurry cylinders. Thus, the maximum compression stress for specimen H226 was estimated to be 10% lower than the compressive strength of the plane slurry. Values for specimen H118D were estimated to be a little greater than those values for specimen H146D because of the similarities in the specimens.

6.3 Hinge Testing Results

Energy absorption and ductility were calculated for each hinge specimen. These values were used to compare the performance of each specimen. The test results were quantified in an effort to determine how each variable affects the SIFCON hinge performance.

Curvature is calculated by computing the rotation of each zone and dividing it by the length of the instrumented zone. Thus, the curvature values are averages for each zone. The moments plotted on the hysteresis plots are calculated by multiplying the load by the moment arm to the center of each zone. Energy absorption is obtained by calculating the area enclosed by the moment - curvature hysteresis loops. The energy absorption for each zone, and of the full hinge, is calculated.

The ductility of each zone is calculated by dividing the maximum curvature obtained in each zone (at a moment not less than 75% of the yield moment) by the yield curvature of that zone. The yield point of each specimen is defined as the point at which the moment curvature plot becomes nonlinear. This is more accurate and consistent for all specimens than any of the methods used by previous researchers. The curvature and ductility values reported are for only the positive displacement strokes only. Sometimes this results in lower ductilities, and sometimes it results in higher ductilities. By using only positive
displacements to calculate ductility, all the variability introduced by residual tensile strains in the specimen is removed. Hysteresis plots are shown up to failure of the specimen. Failure is defined as that point at which: 1) reinforcing bar rupture occurs or, 2) load decreases 25% below the yield load. Finally, as a result of residual tensile strains in the specimen, each specimen elongated as yielding occurred. The final plot shown for each specimen is how the specimen elongated as it was loaded.

6.3.1 Summary

Table 6.4 and 6.5 shows a summary of hinge performance. Table 6.4 list overall hinge performance while Table 6.5 concentrates only on the critical failure zone of each hinge. Included in these tables are curvature ductility, energy absorption, and hinge failure mode. Five of the seven hinges tested failed within the central hinge region. Of these seven, only three failed as a result of rupture of the tension reinforcing. These three are V44, H194 and H146D. Two hinges, H190 and H118D, failed in the connection region. Figure 6.9 shows a graphical comparison of the total energy absorption of each specimen. Figure 6.10 shows a graphical comparison of the energy absorption of only the internal zones (zones ‘B’, ‘C’, and ‘D’) of each specimen. To adjust for the effect of the various moment capacities of each specimen, the energy absorption shown in Tables 6.4 and 6.5 and in Figures 6.10 and 6.11 was normalized by dividing the area enclosed by its hysteresis loops by its yield moment. The energy absorption shown in Tables 6.5 and 6.5 was further normalized by dividing by the energy absorbed by specimen V44, resulting in energy values greater than one. This aids in the quick comparison of specimen results.

The contrast between Figures 6.10 and 6.11 shows the importance of isolating the connection region from the specimen hinging. From Figure 6.10, specimen H194 appears to absorb more energy than the other specimens. Once the connection yielding is
removed from the results, **Figure 6.11** indicates that specimens H194 and H226 absorb more than twice the energy as the next closest specimen.

Energy absorption data indicate that reinforced SIFCON hinges using Grade 60 reinforcing absorb much more energy than hinges using higher strength reinforcing. This finding must be tempered with the fact that specimen H194HS failed due to compression buckling of the reinforcing, and the anchorage details used for specimen H146D did not promote distributed yielding (see Chapter 5, sections 5.6 and 5.7 for details). Thus, the full energy absorption and ductility of these two specimens were not developed in this program. However, the energy absorbed in specimens using Grade 60 reinforcing is over twice that of specimens using higher strength reinforcing. In light of this fact, it is believed that reinforced SIFCON hinges using Grade 60 reinforcing steel will exhibit better hinge behavior than SIFCON hinges using higher strength reinforcing.

Ductility comparisons of the specimens are shown in **Figures 6.12** and **6.13**. **Figure 6.12** shows the curvature ductility for the internal zones (zones ‘B’, ‘C’, and ‘D’) of the specimens. The value shown for specimen V44 is for zone ‘A’. **Figure 6.13** shows the curvature ductility for just the failure zone of each specimen. A curvature ductility of 26.4 was achieved, over a 4 inch length of the hinge, for specimen H194. A curvature ductility of 13.3 was achieved, over a 13 inch length of the hinge, for specimen H226. These values are over twice that reported in for reinforced concrete or reinforced FRC hinges.

Ductility data indicates that specimens using Grade 60 reinforcing exhibit twice the ductility of specimens using higher strength reinforcing. Again, this finding must be qualified by the premature buckling of compression reinforcing in specimen H194HS and the effect of the embedded anchor plates of specimen H146D (see section 5.7). Had reinforcing buckling been prevented in specimen H194HS or different details been used on specimen H146D, the ductilities exhibited by these two specimens would likely increase. However, the ductility would have to double to show better behavior than
specimens using Grade 60 reinforcing. This is very unlikely. Thus, this author concludes that reinforced SIFCON hinges using Grade 60 reinforcing exhibit higher ductilities than SIFCON hinges using higher strength reinforcing.

**Figure 6.14** shows the moment curvature envelope of the four most successful specimens. **Figure 6.15** shows the moment curvature plots of these same specimens. The stiffness variations between the specimens are clearly illustrated in **Figure 6.14**. The higher strength steel in specimen H194HS results in the greatest stiffness while the specimen H146D, having the least quantity of steel of the four specimens, is the softest. Apparently, the higher strength Dywidag bars were not enough to offset the lower quantity of steel. The specimen with the greatest quantity of steel, H226, also contains the lowest strength SIFCON matrix. These two properties seem to balance out, resulting in a specimen with almost identical stiffness of H194. The stiffest specimen is H194HS.

Specimen H146D exhibits the widest hysteresis loops, but the specimen softness results in a yield curvature that is almost double that for the other specimens. As a result, the ductility of specimen H146D is about half that of the other specimens. In addition, this specimen also exhibits rapid post-peak strength degradation—undesirable behavior for plastic hinges. This is likely a result of the reinforcing anchor details used for this specimen.

Specimen H194HS also shows rapid post-peak strength degradation, but this is surely a result of compression buckling of the reinforcing bars. The ultimate moment capacity H194HS is comparable to all the other specimens. Yet only H194HS failed as a result of buckling of compression reinforcing. Specimen H194HS lost confinement when a flexural crack penetrated diagonally into the depth of the specimen. No other specimen exhibited the degree of diagonal cracking observed in H194HS. It is likely that buckling of compression reinforcing would have occurred in other specimens had they developed the same degree of diagonal cracking as specimen H194HS. One-half the minimum area
of transverse ties, as required by ACI 318-95, should be used in SIFCON hinges to prevent compression buckling of compression reinforcing.

Specimens H194 and H226 exhibited the most stable yielding behavior. The moment curvature plots are very similar. Each specimen exhibits a flat yield plateau after approximately 10% strain hardening. Had specimen H226 not failed in shear, it would likely have performed better than specimen H194. It is also very likely that specimen H194HS would have exhibited a flat yield plateau had the compression reinforcing not buckled.

Figures 6.14 and 6.15 support the conclusions drawn from Figures 6.10 through 6.13. In the author’s opinion, reinforced SIFCON hinges using grade 60 reinforcing will result in the best behavior. Even if future research shows that higher strength steel results in better hinge performance, the use of higher strength reinforcing steel is impractical in the light of performance compatibility. High strength reinforcing leads to higher strength SIFCON members. At present, it is difficult to produce reinforced SIFCON hinges that exhibit low enough strength to be used as in reinforced concrete buildings. Thus, Grade 60 reinforcing may be more of a practical requirement than a requirement to optimize SIFCON hinge behavior.

Peak load versus specimen elongation plots show that all specimens elongated as they were loaded. These plots tend to closely match the shape of the moment curvature envelopes for each specimen. Thus, it is clear that specimen elongation initiates at the onset of specimen yield, and is caused by crack opening in the SIFCON matrix and residual tensile strains in the reinforcing steel lead to specimen elongation. Analytical models of the hinges, performed in Chapter 7, indicate that specimen elongation results in stiffness softening of the specimen and does not greatly effect the ultimate strength of the SIFCON hinges. This is further discussed, and compared to reinforced concrete behavior, in Chapter 7.
Potentiometer readings were taken in zone ‘B’ to show linearity of the strain profile through the depth of the specimen. The results were erratic. During early stages of loading, the strain profile remained linear. However, once the moment exceeded approximately 75% of the yield moment, potentiometer readings started indicating a nonlinear strain profile. It is believed that the close proximity of the embedded connectors may have led to erroneous readings. To eliminate the effect of the embedded connectors, the potentiometers and LVDTs in zone ‘C’ were investigated. Zone ‘C’ results were consistent with zone ‘B’. The strain profile was linear at and below approximately 75% yield moment. Once the yield moment was exceeded, the strain profile became less and less linear. There was also no correlation of the strain profile between negative and positive loading. However, a significant portion of the data showed the strain profile to be linear within a 15% error at maximum positive moment. Upon load reversal however, the strain profile varied from negative to positive strains through the depth of the specimens.

6.3.2 V44
This specimen failed due to rupture of the tensile reinforcing in zone ‘A’ at the end of the left rows of embedded connectors (the connectors attached to the bottom end plate). Specimen V44 is very lightly reinforced. The overall specimen achieved a curvature ductility of 4.6 while zone ‘A’ achieved a curvature ductility of 7.6. Figures 6.16 through 6.21 show the load displacement and the moment curvature hysteresis plots for this specimen. The hysteretic shear behavior is shown in Figure 6.22, and the specimen elongation during testing is shown in Figure 6.23. Peak load versus specimen elongation if shown in Figure 6.25. Elongation of this specimen initiated prior to reaching its yield moment. This was the only specimen to exhibit this behavior. It is believed that the poor injection, an subsequent patching (see Chapter 4), led to this behavior. The energy absorption is shown in Figure 6.25. This specimen absorbed the least energy of all specimens, with zone ‘A’ dominating the behavior. The pinching observed in the hysteresis plots is evidence of very little energy absorption—with only zone ‘A’ entering
into nonlinear behavior (compare Figures 6.18 through 6.21). As seen in Figure 6.22 shear deformation remained linear throughout loading.

Specimen V44 exhibited the least elongation of any specimen—exhibiting less than half the elongation of any other specimen. This is a result of minimal distribution of damage to regions adjacent to the primary failure crack. One side of the failure plane was a region heavily reinforced by embedded connectors that prevented any nonlinear behavior. On the other side of the failure plane, the short development length of No. 3 reinforcing bars limited the distribution of yielding. It is likely that the behavior of specimen V44 resembles that of an unreinforced SIFCON flexural region when exposed to a linear moment gradient.

6.3.3 H190
This specimen failed due to a splitting tension plane developing in the outside row of embedded connectors. As the specimen was loaded, the SIFCON over the embedded connectors split, and the connectors pulled out. This type of failure was unanticipated and undesirable. Although some data is reported for this specimen, its usefulness is questionable. The overall specimen achieved a curvature ductility of 6.2 while zone ‘A’ achieved a curvature ductility of 7.4. Figures 6.26 through 6.29 show the load displacement and the moment curvature hysterisis plots for this specimen. The hysteretic shear behavior is shown in Figure 6.30. The purpose for showing this graph is to indicate the magnitude of shear deformations expected under minimal flexural yielding. The energy absorption is shown in Figure 6.31. Although there is some nonlinear action in zones ‘B’, ‘C’, and ‘D’, it is very limited.

6.3.4 H194
Specimen H194 could be considered the most successful specimen. It failed due to rupture of the tensile reinforcing in zone ‘C’ at the end of the No. 6 reinforcing bars. The overall specimen achieved a curvature ductility of 6.9 while zone ‘C’ achieved a
curvature ductility of 26.4. This specimen withstood a maximum load of 37.9 kips. This resulted in a shear stress of 237 psi—less than $3\sqrt{f_e}$.

**Figures 6.32** through **6.39** show the load displacement and the moment curvature hysteresis plots for this specimen. **Figure 6.34** shows the hysteretic behavior of only zones ‘B’, ‘C’, and ‘D’ (interior zones). By removing any nonlinear action of the connection regions, this figure shows the behavior of only the SIFCON hinge region. Zone ‘C’ dominated the behavior and exhibited a curvature ductility of 26.4. This is a higher ductility, over a four-inch long region, than any previously reported.

The hysteretic shear behavior is shown in **Figure 6.40**. Even though the shear measurements were taken close to the failure plane, this specimen exhibited almost negligible shear deformation. A maximum shear deformation of 0.004 radians was observed. This magnitude is undetectable to the naked eye and did not affect the ductility or energy absorption.

As shown in **Figure 6.41**, the specimen elongated approximately $\frac{1}{2}$ inch during testing. Peak load versus specimen elongation is shown in **Figure 6.42**. This general shape of this curve mirrors that for the load-deformation plot (compare **Figure 6.42** to **Figure 6.32**). It is clear that residual tensile strains (specimen elongation) initiated when the specimen reached its yield moment. This is typical of any flexural member. Unrecoverable elongation occurs once the reinforcing steel exceeds its yield point. The effects of specimen elongation are discussed in section 6.3.1. To reiterate, hinge test results, when compared to analytical predictions, indicate that SIFCON compression behavior is not substantially effected by the cyclic opening and closing of tensile cracks for the compression strains induced in this study. It is important to note that maximum hinge compression strains induced in this study were estimated to be approximately 1%.

The energy absorption is shown in **Figure 6.43**. The specimen behaved as designed. However, to maximize energy absorption and ductility, more nonlinear behavior is
needed in zones ‘B’ and ‘D’. Distributed yielding was achieved throughout the specimen, but most of the damage was isolated to zone ‘C’. As compared to the two previous specimens, specimen H194 exhibits much wider (less pinched) hysteretic behavior.

6.3.5 H226
Specimen H226 failed due to diagonal cracking in zone ‘D’. The overall specimen achieved a curvature ductility of 6.9 while zone ‘D’ achieved a curvature ductility of 20.5. This specimen withstood a maximum load of 37.7 kips. This resulted in a shear stress of 236 psi—more than $3 \sqrt{f_c}$ but less than $6 \sqrt{f_c}$. Thus, according to previous research, this specimen should fail due to a combination of shear and bending [Bertero and Popov, 1977; and Scribner and Wight, 1980]. The reader is cautioned that the aforementioned research was performed on reinforced concrete hinges. Care should be exercised when applying reinforced concrete research to SIFCON specimens. Other specimens had shear stresses and strengths on the same magnitude as this specimen and were not adversely affected by shear stress.

Figures 6.44 through 6.51 show the load displacement and the moment curvature hysteresis plots for this specimen. Figure 6.46 shows the hysteretic behavior of only zones ‘B’, ‘C’, and ‘D’ (interior zones). By removing any nonlinear action of the connection regions, this figure shows the behavior of only the SIFCON hinge region. Zones ‘B’, ‘C’ and ‘D’ contribute to produce a curvature ductility of 13.3.

The hysteretic shear behavior is shown in Figure 6.52. Although the graph shows shear deformations in excess of 0.025 radians, the actual shear deformation was somewhat less. As deformations in zone ‘D’ started becoming excessive, the curvature instrumentation started returning inconsistent data. This is evidenced by the moment curvature plot of zone ‘E’, shown in Figure 6.51. Calculations are made to remove the curvature effects on shear deformation. As the curvature instrumentation started slipping, the shear deformations started contradicting other measurements. Thus, a second method to
compute shear deformations was used to check the previous results. The second method included the effects of curvature and resulted in maximum shear deformations of 0.17 radians. Thus, the actual shear deformation is closer to 0.015 radians, and not the 0.025 radians shown in Figure 6.52. Although the specimen failed as a result of diagonal cracking, it does not appear that the shear stresses significantly affected the energy absorption and ductility up to the point of halting the test.

As shown in Figure 6.53 and Figure 6.54, the specimen elongated approximately one inch during testing. This was more elongation than any other specimen. The reason specimen H226 elongated more is because damage was distributed to two zones instead of just one. The energy absorption is shown in Figure 6.55. There is a good distribution of energy between zones ‘C’ and ‘D’.

Two facts separate this specimen from all the others. First, this specimen exhibited more distribution of damage within the interior of the hinge than all the other hinges. And second is that this specimen exhibited wider hysteresis loops than all other specimens. At first glance, it appears that shear sliding may be responsible for the wide hysteresis loops. However, shear sliding shows up in other ways on hysteresis plots. Shear sliding appears as an abrupt softening then stiffening near zero moment on the loading and unloading sides of the hysteresis loops. The only other difference in this specimen is that it contains more steel and less SIFCON strength. Lower SIFCON strength leads to more distributed SIFCON damage. More steel increases the energy absorption achieved through cyclic yielding of the steel. Thus, it is unclear whether the energy absorption is the result of more steel, more SIFCON damage, or a combination of both. But it is clear that the shear sliding is not responsible for the wide hysteresis loops.

6.3.6 H118D
Specimen H118D failed by developing horizontal shear planes in zone ‘A’ when the Dywidag anchor hardware started slipping. As a result of the hardware slip, the hysteresis plots are slightly pinched when compared to other specimens. The overall
specimen achieved a curvature ductility of 3.4 while zone ‘A’ achieved a curvature ductility of 6.0. **Figures 6.56 through 6.58** show the load displacement and the moment curvature hysteresis plots for this specimen. The energy absorption is shown in **Figure 6.59**. It is obvious that zone ‘A’ dominated the hinge behavior, and very little nonlinear action developed on the interior of the hinge. Thus, the results from this specimen have questionable value except to determine how not to design a connection using Dywidag hardware.

### 6.3.7 H194HS

Specimen H194HS failed due to buckling of compression reinforcing in zone ‘D’. The overall specimen achieved a curvature ductility of 6.7 while zone ‘D’ achieved a curvature ductility of 9.1. This specimen withstood a maximum load of 46.7 kips. This resulted in a shear stress of 292 psi—more than $3 \sqrt{f_c}$ but less than $6 \sqrt{f_c}$. Thus, according to previous research, this specimen should fail due to a combination of shear and bending [Bertero and Popov, 1977; and Scribner and Wight, 1980]. However, shear did not affect the performance of this specimen. The aforementioned research was performed on reinforced concrete hinges, and is likely not applicable to SIFCON specimens.

**Figures 6.60 through 6.67** show the load displacement and the moment curvature hysteresis plots for this specimen. **Figure 6.62** shows the hysteretic behavior of only zones ‘B’, ‘C’, and ‘D’ (interior zones). By removing any nonlinear action in the connection regions, this figure shows the behavior of only the SIFCON hinge region. Closer inspecting of **Figure 6.65** indicates that as spalling occurs (induced by buckling of reinforcing), some erroneous data was collected for the curvature in zone ‘C’. This likely causes the negative curvatures observed in **Figure 6.65**.

The hysteretic shear behavior is shown in **Figure 6.68**. Although the graph shows residual nonlinear shear deformations in excess of 0.015 radians, the actual shear deformation was somewhat less. Just as in specimen H226, instrumentation slip (caused
by rebar buckling) lead to inconsistent data. Again a check was performed just as in specimen H226, resulting in shear deformations less than 0.008 radians. Thus, the actual shear deformation is closer to 0.005 radians, than the 0.015 radians shown in Figure 6.68. However, the residual nonlinear shear deformations were certainly on the order of 0.004 radians. At this level, the shear deformations are undetectable by the naked eye, and do not affect energy absorption or ductility.

Figures 6.69 and 6.70 indicate that the specimen elongated approximately 0.7 inches during testing. Again, the shape of the peak load versus elongation plot mirrors that of the load-displacement plot indication elongation initiating at the onset of specimen yielding.

The energy absorption is shown in Figure 6.71. Although this specimen absorbed almost 30% more energy than the next highest specimen, Figure 6.71 indicates that most of the energy was absorbed in the connection region, zone ‘A’. It is interesting that this specimen exhibited the most distribution of cracking and damage as any specimen. The cracking patterns were encouraging and indicated a high level of energy absorption on the interior of the hinge. However, the actual data did not show very high levels of energy absorption in the interior of the hinge. During testing, the separation between the specimen and the bottom base plate opened considerably more than specimen H146D, which used the same connection detail. This indicates that the reinforcing splices at the end plates were not the cause of the extra base separation. The reasons for the observed behavior remain undetermined. Had the reinforcing not buckled, this specimen may have equaled or exceeded the performance of H194 and H226.

6.3.8 H146D
Specimen H146D failed due to rupture of tension reinforcing in zone ‘D’. The overall specimen achieved a curvature ductility of 3.3 while zone ‘D’ achieved a curvature ductility of 12.1. This specimen withstood a maximum load of 46.7 kips. This resulted in a shear stress of 292 psi—more than 3 $\sqrt{f_c}$ but less than 6 $\sqrt{f_c}$. Thus, according to
previous research, this specimen should fail due to a combination of shear and bending [Bertero and Popov, 1975; and Scribner and Wight, 1978]. However, shear did not affect the performance of this specimen. The aforementioned research was performed on reinforced concrete hinges, and is likely not applicable to SIFCON specimens.

Figures 6.72 through 6.79 show the load displacement and the moment curvature hysteresis plots for this specimen. Additionally, Figure 6.74 shows the hysteretic behavior of only zones ‘B’, ‘C’, and ‘D’ (interior zones). By removing any nonlinear action of the connection regions, this figure shows the behavior of only the SIFCON hinge region. There was very little cracking observed except in zone ‘D’. It is believed that the embedded plates terminating the shortened No. 5 Dywidag bars between zones ‘C’ and ‘D’ (See Figure 4.9) caused localized compressive stresses in the SIFCON that prevented crack distribution into adjacent zones.

The hysteretic shear behavior is shown in Figure 6.80. The graph indicates maximum residual nonlinear shear deformations of approximately 0.013. This only occurred during the last few cycles as a result of the opening of the primary failure crack. This level of shear deformation is not detectable to the naked eye, and did not affect the ductility or energy absorption.

Specimen elongation is shown in Figures 6.81 and 6.82 while energy absorption is shown in Figure 6.83. The specimen elongated approximately 0.8 inches during testing. This is a lot of elongation considering only minimal cracking, only one primary failure crack developed, and Dywidag (low ductility) bars were used. Dywidag bars exhibit about one-half the elongation of Grade 60 and Grade 75 reinforcing bars. It is clear that zone ‘D’ dominated the energy absorption. There was little energy contribution from zones ‘B’ and ‘C’. This specimen isolated failure to one region with very little distribution of damage. The localized compression field created by the embedded end plate may have prevented damage from spreading to zones ‘B’ and ‘C’ while zone ‘D’ was yielding. It is important to note the lack of energy absorption of the end plate
connection (zone ‘A’) as compared to specimen H194HS. The end plate connection
details were almost the same for each specimen.
### Table 6.1: Tension data for reinforcing steel

<table>
<thead>
<tr>
<th>Steel Type</th>
<th>Grade</th>
<th>$f_y$ (ksi)</th>
<th>$\varepsilon_y$ (%)</th>
<th>$\varepsilon_{sh}$ (%)</th>
<th>$f_u$ (ksi)</th>
<th>Elongation in 8 in. (%)</th>
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</thead>
<tbody>
<tr>
<td>‘3’</td>
<td>60</td>
<td>75</td>
<td>0.259</td>
<td>0.259</td>
<td>105</td>
<td>11.5</td>
</tr>
<tr>
<td>‘4’</td>
<td>60</td>
<td>63</td>
<td>0.217</td>
<td>0.274</td>
<td>100</td>
<td>13.5</td>
</tr>
<tr>
<td>‘5’</td>
<td>60</td>
<td>76</td>
<td>0.262</td>
<td>0.285</td>
<td>120</td>
<td>12</td>
</tr>
<tr>
<td>‘6A’</td>
<td>60</td>
<td>66</td>
<td>0.228</td>
<td>0.269</td>
<td>105</td>
<td>15</td>
</tr>
<tr>
<td>‘6B’</td>
<td>60</td>
<td>71</td>
<td>0.245</td>
<td>0.305</td>
<td>110</td>
<td>15.5</td>
</tr>
<tr>
<td>‘7’</td>
<td>60</td>
<td>76</td>
<td>0.262</td>
<td>0.351</td>
<td>106.5</td>
<td>15.5</td>
</tr>
<tr>
<td>‘6HS’</td>
<td>75</td>
<td>89</td>
<td>0.308</td>
<td>0.461</td>
<td>109</td>
<td>16.5</td>
</tr>
<tr>
<td>‘5D’</td>
<td>ASTM A 722</td>
<td>141</td>
<td>0.486</td>
<td>0.259</td>
<td>175</td>
<td>6.5</td>
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</table>

See text or Figure 6.1 for definition of symbols.

### Table 6.2: SIFCON compression data for test cylinders.

<table>
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<tr>
<th>Specimen</th>
<th>Slurry $f_c$ (ksi)</th>
<th>$\sigma_{max}$ (ksi)</th>
<th>$\varepsilon_{max}$ (%)</th>
<th>$\sigma_{infl}$ (ksi)</th>
<th>$\varepsilon_{infl}$ (%)</th>
<th>$\sigma_{plat}$ (ksi)</th>
<th>E (ksi)</th>
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<tbody>
<tr>
<td>V44</td>
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<td>3.7</td>
<td>8.7</td>
<td>4.48</td>
<td>8.5</td>
<td>1240</td>
</tr>
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<td>H190</td>
<td>8.5</td>
<td>9.3</td>
<td>7.6</td>
<td>9.0</td>
<td>9.0</td>
<td>NA</td>
<td>1350</td>
</tr>
<tr>
<td>H194</td>
<td>9.2</td>
<td>9.8</td>
<td>5.0</td>
<td>9.2</td>
<td>6.4</td>
<td>NA</td>
<td>1550</td>
</tr>
<tr>
<td>H226</td>
<td>3.4</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>960</td>
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<td>H118D</td>
<td>7.2</td>
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<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>1870</td>
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<tr>
<td>H194HS</td>
<td>7.6</td>
<td>7.9</td>
<td>5.9</td>
<td>7.5</td>
<td>9.7</td>
<td>6.9</td>
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<tr>
<td>H146D</td>
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<td>7.9</td>
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<td>NA</td>
<td>NA</td>
<td>1570</td>
</tr>
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</table>

See text for definition of symbols.
Table 6.3: SIFCON compression data for test prisms.

<table>
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<tr>
<th>Specimen</th>
<th>Slurry $f_c$ (ksi)</th>
<th>$\sigma_{p_{max}}$ (ksi)</th>
<th>$\epsilon_{\text{max}}$ (%)</th>
<th>$\sigma_{\text{infl}}$ (ksi)</th>
<th>$\epsilon_{\text{infl}}$ (%)</th>
<th>$\sigma_{\text{plat}}$ (ksi)</th>
<th>$\sigma_{\text{max}}$ (ksi)</th>
<th>$\epsilon_{\text{max}}$ (%)</th>
<th>E (ksi)</th>
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</thead>
<tbody>
<tr>
<td>V44</td>
<td>8.4</td>
<td>8.7</td>
<td>1.05</td>
<td>7.3</td>
<td>2.0</td>
<td>5.1</td>
<td>1.7*</td>
<td>0.6*</td>
<td>2340</td>
</tr>
<tr>
<td>H190</td>
<td>8.5</td>
<td>7.4*</td>
<td>0.7*</td>
<td>6.4*</td>
<td>2.0*</td>
<td>4.9*</td>
<td>1.6*</td>
<td>0.6*</td>
<td>NA</td>
</tr>
<tr>
<td>H194</td>
<td>9.2</td>
<td>7.8*</td>
<td>0.7*</td>
<td>6.8*</td>
<td>2.0*</td>
<td>5.2*</td>
<td>1.7*</td>
<td>0.6*</td>
<td>NA</td>
</tr>
<tr>
<td>H226</td>
<td>3.4</td>
<td>3.1**</td>
<td>0.7*</td>
<td>2.7*</td>
<td>2.0*</td>
<td>2.1*</td>
<td>0.7*</td>
<td>0.6*</td>
<td>NA</td>
</tr>
<tr>
<td>H118D</td>
<td>7.2</td>
<td>6.5</td>
<td>0.7</td>
<td>5.7</td>
<td>3.0</td>
<td>4.4</td>
<td>1.5*</td>
<td>0.6*</td>
<td>NA</td>
</tr>
<tr>
<td>H194HS</td>
<td>7.6</td>
<td>6.8</td>
<td>0.6</td>
<td>6.0</td>
<td>1.5</td>
<td>4.3</td>
<td>1.4*</td>
<td>0.6*</td>
<td>1970</td>
</tr>
<tr>
<td>H146D</td>
<td>7.1</td>
<td>6.4</td>
<td>0.7</td>
<td>5.6</td>
<td>3.0</td>
<td>4.4</td>
<td>1.4*</td>
<td>0.6*</td>
<td>1870</td>
</tr>
</tbody>
</table>

* From correlation to other tests. ** 10% lower than plane slurry. + From Homrich and Naaman [1987]
Table 6.4: Specimen strength, ductility, and energy absorption summary.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>$M_y$ (ft-k)</th>
<th>$M_a$ (ft-k)</th>
<th>Yield Curvature</th>
<th>Ultimate Curvature</th>
<th>Curvature Ductility</th>
<th>Energy Absorption (k-ft)</th>
<th>Energy Absorbed Normalized to $M_y$</th>
<th>Energy Absorbed Normalized to $M_y$ &amp; V44</th>
<th>Hinge Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>V44</td>
<td>96</td>
<td>118</td>
<td>0.00138</td>
<td>0.00636</td>
<td>4.6</td>
<td>3.3</td>
<td>0.034</td>
<td>1.0</td>
<td>Bar rupture due to flex @ end of connectors</td>
</tr>
<tr>
<td>H190</td>
<td>114</td>
<td>155</td>
<td>0.00165</td>
<td>0.0102</td>
<td>6.2</td>
<td>8.8</td>
<td>0.077</td>
<td>2.26</td>
<td>Split tension @ end of connectors</td>
</tr>
<tr>
<td>H194</td>
<td>140</td>
<td>180</td>
<td>0.00171</td>
<td>0.0118</td>
<td>6.9</td>
<td>23.5</td>
<td>0.169</td>
<td>4.97</td>
<td>Bar rupture due to flex in zone ‘C’</td>
</tr>
<tr>
<td>H226</td>
<td>142</td>
<td>179</td>
<td>0.00170</td>
<td>0.0118</td>
<td>6.9</td>
<td>26.9</td>
<td>0.189</td>
<td>5.56</td>
<td>Shear in zone ‘D’</td>
</tr>
<tr>
<td>H118D</td>
<td>153</td>
<td>188</td>
<td>0.00284</td>
<td>0.00975</td>
<td>3.4</td>
<td>18.2</td>
<td>0.119</td>
<td>3.5</td>
<td>Hor shear due to rebar hardware slip</td>
</tr>
<tr>
<td>H194HS</td>
<td>175</td>
<td>222</td>
<td>0.00225</td>
<td>0.015</td>
<td>6.7</td>
<td>42.9</td>
<td>0.245</td>
<td>7.2</td>
<td>Comp rebar buckling in zone ‘D’</td>
</tr>
<tr>
<td>H146D</td>
<td>183</td>
<td>212</td>
<td>0.00265</td>
<td>0.0088</td>
<td>3.3</td>
<td>12.6</td>
<td>0.069</td>
<td>2.03</td>
<td>Bar rupture due to flex in zone ‘D’</td>
</tr>
</tbody>
</table>
Table 6.5: Individual zone strength, ductility, and energy absorption summary.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Zone</th>
<th>$M_y$ (ft-k)</th>
<th>$M_u$ (ft-k)</th>
<th>Yield Curvature</th>
<th>Yield Curvature</th>
<th>Curvature Ductility</th>
<th>Energy Absorption (k-ft)</th>
<th>Energy Abs Normalized to $M_y$</th>
<th>Energy Abs Normalized to $M_y$ &amp; V44</th>
</tr>
</thead>
<tbody>
<tr>
<td>V44</td>
<td>A</td>
<td>121</td>
<td>141</td>
<td>0.00067</td>
<td>0.0051</td>
<td>7.6</td>
<td>2.9</td>
<td>0.024</td>
<td>1.0</td>
</tr>
<tr>
<td>H190</td>
<td>A</td>
<td>144</td>
<td>170</td>
<td>0.00087</td>
<td>0.0064</td>
<td>7.4</td>
<td>5.1</td>
<td>0.035</td>
<td>1.4</td>
</tr>
<tr>
<td>H194</td>
<td>C</td>
<td>140</td>
<td>182</td>
<td>0.00033</td>
<td>0.0087</td>
<td>26.4</td>
<td>12.5</td>
<td>0.089</td>
<td>3.7</td>
</tr>
<tr>
<td>H226</td>
<td>D</td>
<td>147</td>
<td>168</td>
<td>0.00040</td>
<td>0.0082</td>
<td>20.5</td>
<td>11.1</td>
<td>0.076</td>
<td>3.2</td>
</tr>
<tr>
<td>H118D</td>
<td>A</td>
<td>170</td>
<td>220</td>
<td>0.00140</td>
<td>0.0084</td>
<td>6.0</td>
<td>17.1</td>
<td>0.101</td>
<td>4.2</td>
</tr>
<tr>
<td>H194HS</td>
<td>D</td>
<td>187</td>
<td>211</td>
<td>0.00037</td>
<td>0.0034</td>
<td>9.1</td>
<td>4.8</td>
<td>0.026</td>
<td>1.1</td>
</tr>
<tr>
<td>H146D</td>
<td>D</td>
<td>178</td>
<td>200</td>
<td>0.00047</td>
<td>0.0057</td>
<td>12.1</td>
<td>6.5</td>
<td>0.037</td>
<td>1.5</td>
</tr>
</tbody>
</table>
Figure 6.1: Calculation of reinforcing strain values from typical reinforcing tensile test.

Figure 6.2: Typical 4”x8” vertically filled cylinder, 4”x4”x8” horizontally filled prism, and 3”x3”x8” horizontally filled prism SIFCON compression specimens.
Figure 6.3: Compression tests for specimen V44.

Figure 6.4: Compression tests for specimen H190.
Figure 6.5: Compression tests for specimen H194.

Figure 6.6: Compression tests for specimen H226.
Figure 6.7: Compression tests for specimen H118D.

Figure 6.8: Compression tests for specimen H194HS.
Figure 6.9: Compression tests for specimen H146D.
Figure 6.10: Energy absorption for full specimens.

Figure 6.11: Energy absorption of internal zones (‘B’, ‘C’, and ‘D’) only.
Figure 6.12: Ductility for internal zones (‘B’, ‘C’, and ‘D’) only.

Figure 6.13: Ductility for individual failure zones only.
Figure 6.14: Overlay of the moment curvature envelopes for the internal zones (zones ‘B’, ‘C’, and ‘D’) of the four most successful specimens.
Figure 6.15: Moment curvature plots for the internal zones (zones ‘B’, ‘C’, and ‘D’) of the four most successful specimens.
Figure 6.16: Load vs displacement for specimen V44.

Figure 6.17: Moment vs curvature for specimen V44.

\[ M_y = 96 \text{ ft-k} \]
\[ \phi_y = 0.00138 \]
\[ M_u = 118 \text{ ft-k} \]
\[ \phi_u = 0.00636 \]
\[ \mu_\phi = 4.6 \]
Figure 6.18: Moment vs curvature for specimen V44, zone ‘A’.

Figure 6.19: Moment vs curvature for specimen V44, zone ‘B’.

V44, Zone 'A'

V44, Zone 'B'

$M_y = 121 \text{ ft-k}$

$\phi_y = 0.00067$

$M_u = 141 \text{ ft-k}$

$\phi_u = 0.0051$

$\mu_\phi = 7.6$
Figure 6.20: Moment vs curvature for specimen V44, zone ‘C’.

Figure 6.21: Moment vs curvature for specimen V44, zone ‘D’.
Figure 6.22: Hysteretic shear behavior of specimen V44.

Figure 6.23: Specimen V44 elongation during testing.
Figure 6.24: Peak load versus specimen elongation for specimen V44.

Figure 6.25: Energy absorption, specimen V44.
Figure 6.26: Load displacement response for specimen H190.

Figure 6.27: Moment vs curvature for specimen H190.
Figure 6.28: Moment vs curvature for specimen H190, zone ‘A’.

Figure 6.29: Moment vs curvature for specimen H190, zone ‘B’.
Figure 6.30: Hysteretic shear behavior of specimen H190.

Figure 6.31: Energy absorption for specimen H190.
Figure 6.32: Load displacement response for specimen H194.

Figure 6.33: Moment vs curvature for specimen H194.
Figure 6.34: Moment vs curvature for specimen H194, zones ‘B’, ‘C’, and ‘D’.

Figure 6.35: Moment vs curvature for specimen H194, zone ‘A’.
Figure 6.36: Moment vs curvature for specimen H194, zone ‘B’.

Figure 6.37: Moment vs curvature for specimen H194, zone ‘C’.
Figure 6.38: Moment vs curvature for specimen H194, zone ‘D’.

Figure 6.39: Moment vs curvature for specimen H194, zone ‘E’.
Figure 6.40: Hysteretic shear behavior of specimen H194.

Figure 6.41: Specimen H194 elongation during testing.
Figure 6.42: Peak load versus specimen elongation for specimen H194.

Figure 6.43: Energy absorption, specimen H194.
Figure 6.44: Load displacement response for specimen H226.

Figure 6.45: Moment vs curvature for specimen H226.
Figure 6.46: Moment vs curvature for specimen H226, zones ‘B’, ‘C’, and ‘D’.

Figure 6.47: Moment vs curvature for specimen H226, zone ‘A’.

H226, Zonal ‘B’, ‘C’, and ‘D’

\[ M_y = 156 \text{ ft-k} \]
\[ \phi_y = 0.00080 \]
\[ M_u = 181 \text{ ft-k} \]
\[ \phi_u = 0.01062 \]
\[ \mu = 13.3 \]

H226, Zone ‘A’

\[ M_y = 156 \text{ ft-k} \]
\[ \phi_y = 0.00080 \]
\[ M_u = 181 \text{ ft-k} \]
\[ \phi_u = 0.01062 \]
\[ \mu = 13.3 \]
Figure 6.48: Moment vs curvature for specimen H226, zone ‘B’.

Figure 6.49: Moment vs curvature for specimen H226, zone ‘C’.
Figure 6.50: Moment vs curvature for specimen H226, zone ‘D’.

Figure 6.51: Moment vs curvature for specimen H226, zone ‘E’.
Figure 6.52: Hysteretic shear behavior of specimen H226. (Actual deformation is less. See text).

Figure 6.53: Specimen H226 elongation during testing.
Figure 6.54: Peak load versus specimen elongation for specimen H194.

Figure 6.55: Energy absorption, specimen H226.
Figure 6.56: Load displacement response for specimen H118D.

Figure 6.57: Moment vs curvature for specimen H118D.
Figure 6.58: Moment vs curvature for specimen H118D, zone ‘A’.

Figure 6.59: Energy absorption, specimen H118D.
Figure 6.60: Load displacement response for specimen H194HS.

Figure 6.61: Moment vs curvature for specimen H194HS.
Figure 6.62: Moment vs curvature for specimen H194HS, zones ‘B’, ‘C’, and ‘D’.

Figure 6.63: Moment vs curvature for specimen H194HS, zone ‘A’.
Figure 6.64: Moment vs curvature for specimen H194HS, zone ‘B’.

Figure 6.65: Moment vs curvature for specimen H194HS, zone ‘C’.
Figure 6.66: Moment vs curvature for specimen H194HS, zone ‘D’.

Figure 6.67: Moment vs curvature for specimen H194HS, zone ‘E’.
Figure 6.68: Hysteretic shear behavior of specimen H194HS. (Actual deformation is less. See text).

Figure 6.69: Specimen H194HS elongation during testing.
Figure 6.70: Peak load versus specimen elongation for specimen H194HS.

Figure 6.71: Energy absorption, specimen H194HS.
Figure 6.72: Load displacement response for specimen H146D.

Figure 6.73: Moment vs curvature for specimen H146D.
Figure 6.74: Moment vs curvature for specimen H146D, zones ‘B’, ‘C’, and ‘D’.

Figure 6.75: Moment vs curvature for specimen H146D, zone ‘A’.
Figure 6.76: Moment vs curvature for specimen H146D, zone ‘B’.

Figure 6.77: Moment vs curvature for specimen H146D, zone ‘C’.
Figure 6.78: Moment vs curvature for specimen H146D, zone ‘D’.

Figure 6.79: Moment vs curvature for specimen H146D, zone ‘E’.
Figure 6.80: Hysteretic shear behavior of specimen H146D.

Figure 6.81: Specimen H1146D elongation during testing.
Figure 6.82: Peak load versus specimen elongation for specimen H146D.

Figure 6.83: Energy absorption, specimen H146D.
7 ANALYTICAL MODEL OF HINGES

This chapter will discuss the analytical methods used to determine yield and ultimate strength of the hinges. The analytical results will be compared to the test results to determine if SIFCON hinge behavior can be accurately modeled. Analytical predictions best match experimental results when the test setup involves few discontinuities and is instrumented across a constant moment region. The specimens used in this testing program contained many discontinuities and the setup induced a linearly varying moment. For that reason, average results over instrumented zones will be compared to the analytical program output.

7.1 Analytical Program

A nonlinear Fortran program was written to predict the moment versus curvature behavior of reinforced SIFCON flexural regions. This program is shown in the Appendix. Force equilibrium and strain compatibility are used to calculate the moment and curvature of the SIFCON hinges from zero moment up to reinforcing tensile rupture. In the program, the depth of the member is divided into 100 horizontal slices, and a linear strain profile is assumed for the cross-section. The average strain for each horizontal slice is used to calculate SIFCON tension and compression stresses using the models discussed in Chapter 2. The model suggested by Sergin [1971] (discussed in Section 2.10 and shown in Figure 2.9) was used to calculate the reinforcing steel stress. Using the stresses and contributory areas, forces are summed to satisfy equilibrium. If force equilibrium is not satisfied, the assumed strain profile is adjusted and the process continues. Once force equilibrium is satisfied, the moment and curvature is calculated from the forces and strain profile. Then the maximum strain is increased, and the process is repeated until tensile rupture occurs in the reinforcing. A sample output of the program is shown in Figure 7.1. This figure shows the moment capacity of specimen H194 in zone ‘D’ when two No. 6 reinforcing bars are used in the top and bottom of the specimen. Also plotted in Figure 7.1 are the reinforcing steel tension stress, reinforcing steel compression stress
and SIFCON tension stress as they vary with the moment and curvature of the member cross-section.

7.2 Analytical Results

To better correspond with the applied linear moment gradient, the hinge cross-section changes throughout the length of the hinge. Figures 4.1 through 4.9 show the discontinuities in the reinforcement of each hinge. The moment curvature program was used to predict the moment capacity every time the reinforcing changed.

Figures 7.2 through 7.8 show the moment capacity plots for all specimens. Moment capacity varies directly with the discontinuities in the reinforcing. These plots are approximations based upon perfect embedment of reinforcing steel. The SIFCON material properties used in the analysis are shown in Table 6.3. An initial tension modulus (E_t) of 1000 ksi was used for all specimens. An initial compression modulus (E) of 1800 ksi was used for all specimens except 1200 ksi was used for specimen H226. The material properties shown in Table 6.1 were used for the reinforcing.

7.3 Zones of Comparison

Several factors make comparing the analytical and experimental results difficult. First, the specimen must undergo flexural yielding. Secondly, the instrumentation often bridges reinforcing discontinuities within the hinges. Zone ‘D’ is the only zone instrumented in which there were no discontinuities. And thirdly, failure in one zone often leads to crack opening in an adjacent zone. This is the case with specimen H194 where failure occurred at the end of the hooked No. 6’s in zone ‘D’, but the resulting crack opened in zone ‘C’. Analytical versus experimental comparisons will only be made using specimens V44, H194, H226, H194HS, and H146D. The remaining two specimens showed no yielding in the moment curvature plots prior to failure in the connection regions.
Specimen zones were defined in Figure 3.7. For each specimen, one zone of experimental results is compared to the analytical prediction. The analytical prediction must also occur in the same region as the plotted experimental data. For example, specimen V44 was dominated by the behavior in zone ‘A’. The region just outside of the embedded connectors dominated zone ‘A’ behavior. Consequently, the experimental results from zone ‘A’ are used for comparison, and the analytical model is based upon the cross-section of the specimen just outside of the embedded connectors.

7.3.1 Specimen V44
Specimen V44 failed in zone ‘A’ at the end of the embedded connectors as described in Chapter 6, section 6.3.2. The moment curvature plot for this specimen is shown in Figure 6.13. The experimental moment shown occurs at the center of zone ‘A’, and is computed by multiplying the test load by the moment arm to the center of zone ‘A’. However, the yielding occurred at the end of the connectors (2.5 inches farther up the specimen). Adjusting the test yield and ultimate moments to the end of the connectors, by decreasing the moment arm from the loading point, results in moments of 118 ft-k and 138 ft-k respectively.

The test curvature values in zone ‘A’ are greatly influenced by the junction between the SIFCON specimen and the end plate. There is no tension capacity across this junction except that supplied by the embedded connectors. As a result, the stiffness values obtain for zone ‘A’ are lower than the stiffness at the failure plane at the end of the connectors. Thus, stiffness values for this specimen cannot be compared to analytical values. Yield and ultimate moments are compared for this specimen.

7.3.2 Specimen H194
Specimen H194 failed in zone ‘D’, but crack opening occurred in zone ‘C’. This is described in Chapter 6, section 6.3.4. Thus to compare the predicted and experimental capacities, the analytical moment capacity of zone ‘D’ (at the end of the hooked No. 6’s)
must be compared to the results obtained from the instrumentation of zone ‘C’. Figure 6.31 shows the moment curvature test results for zone ‘C’. The yield and ultimate moments given in this figure are to the center of zone ‘C’. However, failure actually occurred at the end of the hooked No. 6’s (one inch into zone ‘D’). Adjusting the yield and ultimate test moments to the end of the hooked No. 6’s gives moments of 133 ft-k and 173 ft-k respectively.

7.3.3 Specimen H226
Specimen H226 failed due to diagonal shear cracking in zone ‘D’. This failure is described in Chapter 6, section 6.3.5. However, yielding occurred prior to the development of extensive diagonal cracking. Thus, zone ‘D’ yield data can be compared to the analytical prediction. The tested moment curvature behavior for zone ‘D’ is seen in Figure 6.43. The test yield moment shown in this figure occurs at the center of zone ‘D’. Translating this moment to the end of the hooked No. 6’s results in a yield moment of 153 ft-k.

7.3.4 Specimen H194HS
Specimen H194HS failed due to buckling of compression reinforcing in zone ‘D’. This failure is described in Chapter 6, section 6.3.7. However, inspection of the moment curvature plot indicates that the ultimate moment also developed prior to reinforcing buckling. Thus, both yield and ultimate data will be compared. The tested moment curvature behavior for zone ‘D’ is shown in Figure 6.58. The test moment shown in this figure occurs at the center of zone ‘D’. Translating these moments to the end of the hooked No. 6’s results in a yield moment and ultimate moment of 192 ft-k and 217 ft-k respectively.

7.3.5 Specimen H146D
Specimen H146D failed as a result of flexural yielding in zone ‘D’. This failure is described in Chapter 6, section 6.3.8. The tested moment curvature plot for this zone is shown in Figure 6.69. Translating the yield and ultimate moments to the end of the
terminated 5/8” diameter Dywidag bars results in test yield and ultimate moments of 183 ft-k and 206 ft-k respectively.

7.4 Analytical versus experimental comparison

Table 7.1 shows the comparison between the predicted and tested yield moment, ultimate moment, yield curvature, ultimate curvature, and stiffness for the five specimens.

7.4.1 Moment comparisons

Figures 7.9 through 7.13 show the analytical moment curvature plot for the appropriate zone of each specimen plotted with the experimental moment curvature envelope of the same zone. The moment predictions for all specimens fell within 15% of the experimental results, with average results within 8% for yield moments and 11% for ultimate moments. There was no real pattern in how the error was distributed among the specimens. Equal prediction error was shown for specimens using Grade 60, Grade 75, and Dywidag reinforcing. The results are very good considering that SIFCON tensile properties were approximated from compression data and much of the compression properties were approximated from vertically filled SIFCON compression cylinders.

As a result of the cyclic nature of the testing, experimental ultimate moments were expected to be less than the monotonic ultimate moment predictions from the model. The results proved otherwise. For specimens using higher strength reinforcing (H194HS and H146D), the ultimate experimental moments were about 12% greater than the monotonic ultimate moment predictions. For specimens using Grade 60 reinforcing (V44 and H194), the ultimate experimental moments were about 10% less than the monotonic ultimate moment predictions. There are four possible explanations for this behavior.

The first explanation is that the SIFCON material behavior data was inaccurate. However, numerous variations of the tension and compression properties were considered, but none resulted in better moment predictions for specimens H226, H194HS,
and H146D without an equal increase in error for specimens V44 and H194. It is unlikely that inaccurate SIFCON properties led to the poor predictions for specimens H194HS and H146D.

The second possible explanation is the inaccuracies inherent in both the SIFCON tension model and the strain hardening model for used for the reinforcing steel. There are small errors in the predicted behavior for both models. High strength reinforcing steel reaches its ultimate stress at lower strains than that predicted by the stress-strain model used for the reinforcing steel. Naaman and Homrich [1989] flattened the sharp peak of the tensile load elongation curve when they developed the SIFCON stress-strain model in tension (See Figure 2.7). It is likely that the peak tensile stress in the SIFCON coincides, more closely than predicted, with the ultimate tensile stress in high strength reinforcing. This would result in predicted ultimate moment capacities less than those observed in testing for high strength reinforcing.

The third possible explanation is that the cyclic loading inherently introduces some error when compared to the predictions of a monotonic model. Cyclic, post-yield loading causes residual tensile strains in the specimens and results in specimen elongation. This characteristic is discussed in Chapter 6. Through depth flexural cracks open as specimens elongate. Larger curvatures are required to close these cracks and form the compression zone. As larger curvatures are induced, the effective compression zone migrates out to the extreme compression flange—resulting in an increase in effective member depth and increased compressive strains. In reinforced concrete, this usually results in compression zone spalling, and reduced ultimate moment strength. In reinforced SIFCON, this compression zone migration may enhance the ultimate moment capacity. SIFCON’s large compressive strain capacity prevents compression zone spalling, facilitating larger curvatures and larger ultimate moment capacities. Cyclic loading one SIFCON flexural members may enhance ultimate moment capacities. This could explain the low predictions for specimens H194HS and H146D. However, this explanation does not explain the high predictions for specimens V44 and H194.
Additionally, cyclic loading should not affect the yield moment capacities. It is likely that the analytical model underestimated reinforced SIFCON cyclic ultimate moment capacity, but loading up to yield should be unaffected by cyclic loading. From the data, no clear conclusion can be drawn on the cyclic loading effects on the yield and ultimate moment strengths.

The fourth possible explanation is found upon close examination of reinforcing bar strain data. This data reveals that the yield point on the experimental moment curvature hysteresis does not necessarily coincide with yield strain in the reinforcing. Table 7.2 shows the tensile reinforcing strain at the experimentally determined yield point. The analytical strain shown is taken from tensile tests on the reinforcing bars. The strains shown in Table 7.2 are taken directly from strain gage data at the failure planes after the experimental yield moment had been determined. Specimen V44 is not reported because the strain gages did not function properly on this specimen. As seen in the table, simultaneous yielding of the bar and the specimen only occurred for specimen H194 (steel strain at specimen yield exceeds the reinforcing yield strain by 10.2%). All other specimens resulted in reinforcing strains exceeding the reinforcing yield strain by 33 to 52% at specimen yield. Analytical moment curvature plots show specimen yields occurring simultaneous to reinforcing yield. The experimental moment curvature plots show specimen yields occurring at reinforcing strains up to 52% greater than reinforcing yield strain.

The difference in reinforcing strain at specimen yield may account for the differences between the analytical and experimental moment capacities of hinges H226, H194HS, and H146D. The slurries used for specimen H194 and specimens H194HS and H146D are not different enough to expect dissimilar behavior. Therefore, the author does not feel the reason why one prediction is good and another is poor is the result of using two different slurries. It is likely that the combined effects of all four possibilities interact to produce over predictions for some specimens and under predictions for others.
7.4.2 Ductility comparisons
Analytical ductility is calculated by determining the yield and ultimate curvatures of a planar section. Experimental ductility is calculated using instrumentation that measures the deformations of a finite length of a specimen. In this project, this length varied from 4 inches to 8 inches. Additionally, the moment varied linearly over this length. It is impossible to compare results between these two different methods. This is proven by the dissimilar ductility results shown in Table 7.1. The experimental curvature ductility of 26.4 shown for specimen H194 obviously includes distributed cracking and distributed yielding. It is impossible to account for this distribution of damage with the analytical model. There is little agreement between the analytically predicted and the experimentally measure results for ductility.

7.4.3 Stiffness comparisons
Zone ‘D’ is the only constant region instrumented on all specimens. Therefore, zone ‘D’ will be used exclusively to compare analytical and experimental stiffness. Experimental results will be the average stiffness measured in zone ‘D’. Stiffness values for specimen V44 are not listed because failure occurred in zone ‘A’. Table 7.1 shows the experimental and analytical stiffness of each specimen. The values shown are the slope of the moment curvature plots.

Analytical yield curvatures compare favorably to experimental yield curvatures. Specimen H146D shows the largest variation (20%) between predicted and experimental yield curvature. All other predictions fall within approximately 10% of measured values.

Analytical and experimental stiffness can be computed from the slope of the moment curvature plots. The slope of the moment curvature plot is equal to EI (Modulus of elasticity multiplied by the moment of inertia). Two values of EI are shown in Table 7.1 for each specimen. One value is the secant stiffness at 0.75My, and the other value is the secant stiffness at My. My is the yield moment of the specimen, and is defined as the
point at which the moment curvature plot (either analytical or experimental) indicates clear yielding.

As seen in Table 7.1, the analytical stiffness at 0.75M_y is approximately 25% softer than the experimental stiffness. As a result of the restraining effect of the fibers, a pattern of very fine shrinkage cracks developed as each specimen cured. It was anticipated that these cracks would lead to softer SIFCON hinges than the analysis showed. Experimental results proved otherwise. The analytical secant stiffness at yield moment matched the experimental results within an average of 14% (ranging from 4% to 21%). Just as with moment predictions, predicted yield stiffness values for higher strength reinforcing exhibit more error than that for specimens using Grade 60 reinforcing. Thus, it can be safely concluded that the model under predicts SIFCON hinge yield stiffness by at least 10%.

Comparing the data to known reinforced concrete properties, SIFCON flexural members exhibit approximately 35% of the stiffness of moderately reinforced concrete beams with similar moment capacities. A 10”x16” reinforced concrete beam, using 7 ksi concrete and a reinforcing ratio (ρ) equal to 2/3rd of the maximum allowed by ACI 318 results in an analytical yield stiffness (EI) equal to 13,207,000 k-in². This is approximately three times the stiffness of the SIFCON specimens shown in Table 7.1. Even a more accurate method proposed by Park and Ruitong [1988] results in a concrete beam stiffness (EI) of 11,143,000 kin². Thus SIFCON beams exhibit between 35% and 40% of the stiffness of comparable strength reinforced concrete beams.

The effective moment of inertia can be calculated by dividing the stiffness, EI, by the Young’s modulus (as determined from uniaxial SIFCON compression tests). A conservative modulus of 2000 ksi was used for all specimens except H226. For specimen H226 a modulus of 1200 ksi was used. The results for both 0.75M_y and M_y are shown in Table 7.1. The experimental results indicate an effective moment of inertia that is greater than the gross moment of inertia at 0.75M_y. However, at yield the moment of
inertia was approximately 70% of the gross moment of inertia (excluding specimen H226, and H194HS). As a result of the very weak matrix, specimen H226 exhibited a moment of inertia greater than the gross moment of inertia even at yield moment. Specimen H194HS exhibited much greater stiffness (91% of gross) than the other specimens as a result of the large quantity of high strength steel. Just as in reinforced concrete members, the moment of inertia of SIFCON flexural members is directly related to the reinforcing steel quantity and strength. The moment of inertia of lightly reinforced SIFCON members exhibit approximately 75% to 85% of the gross moment of inertia, and approximately 10% more than that predicted by the analytical model.

It is clear from the stiffness results that the high fiber content of SIFCON specimens results in stiffness behavior much different than that expected when using reinforced concrete specimens. Even at yield, reinforced SIFCON hinges can exhibit moment of inertia values greater than the gross moment of inertia.
Table 7.1: Analytical and experimental hinge capacities.

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Failure Zone</th>
<th>Analytical</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$M_y$ (ksi)</td>
</tr>
<tr>
<td>V44</td>
<td>Bar rupture at end of connectors in zone ‘A’</td>
<td>120</td>
</tr>
<tr>
<td>H194</td>
<td>Bar rupture due to flexure in zone ‘D’</td>
<td>146</td>
</tr>
<tr>
<td>H226</td>
<td>Diagonal shear cracking in zone ‘D’</td>
<td>139</td>
</tr>
<tr>
<td>H194HS</td>
<td>Compression rebar buckling in zone ‘D’</td>
<td>163</td>
</tr>
<tr>
<td>H146D</td>
<td>Bar rupture due to flexure in zone ‘D’</td>
<td>175</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Name</th>
<th>Failure Zone</th>
<th>Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td>V44</td>
<td>Bar rupture at end of connectors in zone ‘A’</td>
<td>118</td>
</tr>
<tr>
<td>H194</td>
<td>Bar rupture due to flexure in zone ‘D’</td>
<td>133</td>
</tr>
<tr>
<td>H226</td>
<td>Diagonal shear cracking in zone ‘D’</td>
<td>153</td>
</tr>
<tr>
<td>H194HS</td>
<td>Compression rebar buckling in zone ‘D’</td>
<td>192</td>
</tr>
<tr>
<td>H146D</td>
<td>Bar rupture due to flexure in zone ‘D’</td>
<td>183</td>
</tr>
</tbody>
</table>

* $I_g = 3413$ in^4  ** E = 1200 ksi
Table 7.2: Difference between analytical and experimental reinforcing strain at specimen yield.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Failure Zone</th>
<th>$P_y$ (kips)</th>
<th>$\varepsilon_s @ P_y$ Analytical (%)</th>
<th>$\varepsilon_s @ P_y$ Experimental (%)</th>
<th>% Rebar strain exceeds yield (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H194</td>
<td>Zone ‘C’</td>
<td>30.0</td>
<td>0.245</td>
<td>0.27</td>
<td>10.2</td>
</tr>
<tr>
<td>H226</td>
<td>Zone ‘D’</td>
<td>33.0</td>
<td>0.262</td>
<td>0.35</td>
<td>33.6</td>
</tr>
<tr>
<td>H194HS</td>
<td>Zone ‘D’</td>
<td>41.9</td>
<td>0.308</td>
<td>0.47</td>
<td>52.6</td>
</tr>
<tr>
<td>H146D</td>
<td>Zone ‘D’</td>
<td>38.1</td>
<td>0.259</td>
<td>0.35</td>
<td>35.1</td>
</tr>
</tbody>
</table>
Figure 7.1: Analytical moment and stress versus curvature at end of No. 6 bars in specimen H194.
Figure 7.2: Moment capacity plot and elevation of specimen V44.
Figure 7.3: Moment capacity plot and elevation of specimen H190.
Figure 7.4: Moment capacity plot and elevation of specimen H194.
Figure 7.5: Moment capacity plot and elevation of specimen H226.
Figure 7.6: Moment capacity plot and elevation of specimen H118D.
Figure 7.7: Moment capacity plot and elevation of specimen H194HS.
Figure 7.8: Moment capacity plot and elevation of specimen H146D.
Figure 7.9: Analytical moment and stress versus curvature plot with experimental moment envelope for specimen V44, zone ‘A’.

Figure 7.10: Analytical moment and stress versus curvature plot with experimental moment envelope for specimen H194, zone ‘D’.
Figure 7.11: Analytical moment and stress versus curvature plot with experimental moment envelope for specimen H194HS, zone ‘D’.

Figure 7.12: Analytical moment and stress versus curvature plot with experimental moment envelope for specimen H226, zone ‘D’.
Figure 7.13: Analytical moment and stress versus curvature plot with experimental moment envelope for specimen H146D, zone ‘D’.

$EI_y = 3,864,000$ k-in$^2$
A nonlinear time step analysis was performed on the prototype structure using the software package Ruaumoko [Carr, 1998]. Second order effects (P-Delta or hinge elongation) and slab effects were not considered in this analysis. There were two purposes for the analysis. The first purpose was to determine if the hysteretic behavior of the SIFCON hinges can be modeled with reasonable accuracy. The second purpose was to determine the effect of SIFCON hinges on the overall performance of a concrete moment frame exposed to severe seismic loading.

The Elcentro and North Ridge earthquake acceleration records were chosen for this analysis. These two records compliment each other in that they are representative of two separate types of earthquakes. The Elcentro record, as seen in Figure 8.1a, is reflective of longer duration ground motion with approximately the same amplitude and frequency. The North Ridge record, as seen in Figure 8.1b, is reflective of a more random frequency earthquake ground motion with a larger amplitude occurring over a shorter duration. These accelerations, together with dead load nodal masses, are used to compute the inertial forces in the structure.

An additional analysis was performed to determine the effect of stronger earthquakes. For this analysis the earthquake records are scaled to 150% of the ground accelerations. This creates earthquake ground accelerations that are 50% stronger than the recorded data.

### 8.1 Analysis Assumptions and Limitations

Each beam was divided into three distinct sections for the time-step analysis. These sections are shown in Figure 8.2. The outer three feet of each beam was modeled as 6 ksi reinforced SIFCON. The interior of each beam and the columns were modeled as 6
ksi reinforced concrete. Plastic hinges were placed on the outer 12 inches of each SIFCON region, the outer 12 inches of the concrete beams, and on each column 11 inches above and below the girders (See Figure 8.2). In the analysis performed, once the yield moment is reached on any portion of the plastic hinge, the whole hinge is considered to be yielding, and a constant moment is imposed on the full length of the hinge. A preliminary analysis was performed with yield moment strengths of each plastic hinge set equal to 5% more than the ultimate applied moment as determined from the equivalent lateral force analysis. The 5% increase is to account for the fact that members are always slightly over-designed. For the final analysis, the yield moment strengths were adjusted to 30% and 50% greater for the concrete beams and columns respectively.

Young’s moduli of 4400 and 2500 ksi were used for the concrete and the SIFCON regions respectively. Five percent structural damping was assumed for the structure. The moment of inertia for the concrete columns, concrete beams, and SIFCON beam region was set at 0.7I_g, 0.5I_g, and 0.6I_g respectively. As a result of the lower modulus of SIFCON, this made the SIFCON regions 68% softer than the connected concrete beams.

The Takeda model is used to model all plastic hinge regions. The Takeda model [Takeda, et. al., 1970] is used by many researchers to model the hysteretic behavior of reinforced concrete. The model performs adequately except that it does not accurately model hysteretic pinching. The experimental component of this research program showed that SIFCON flexural regions exhibit much less pinching than reinforced concrete flexural regions. Thus, the Takeda model should more closely model the hysteretic behavior of SIFCON than that of reinforced concrete.

To properly model the prototype SIFCON regions, the SIFCON region must be 45 inches long, contain 12 inch rigid connection zones at each end, and include a 21 inch long central region that yields at two separate moment magnitudes. This was impossible to model given the restrictions of the analysis program. Thus, analytical modifications were
necessary to match, as close as possible, the prototype structure. The modifications are discussed below.

A Giberson beam element [Giberson, 1967] is used for all flexural members. In this element type, a predefined length of plastic hinging is input for each end of the member. Ductility demand, and overall element behavior, are computed based upon the inputted plastic hinge length and yield moment. Additionally, once yielding occurs at any point in the hinge, yielding is enforced on the full length of the hinge and the hinge is considered to be under a constant moment. Thus, to model a moment gradient, two plastic hinge regions, each with different yield moments, were required in the SIFCON. Inputted plastic hinge lengths are also important. Numerical instability may develop if the entire SIFCON region is allowed to yield. Thus it was decided to allow a 12 inch plastic hinge length (1/3 of the member length) on each end of the SIFCON regions. (A sensitivity analysis indicated no instability, but the twelve-inch long SIFCON hinges were used to ensure against potential instability). This resulted in a total plastic hinge length of 24 inches—approximately equal to the 45 inch long SIFCON region minus the 12 inch rigid connection zones on each end. Plastic hinges were placed at the ends of each beam and column (adjacent to the SIFCON regions) to determine if plastic hinging penetrated into the concrete beams or columns.

8.2 Results

Preliminary analysis revealed two things. First, the critical earthquake record was Elcentro. Displacement and ductility demand were much greater using the Elcentro record over the North Ridge record. Thus, final analysis was performed using only the Elcentro accelerogram. The second thing that preliminary analysis revealed is that more hinging occurred in the concrete beams than the SIFCON regions unless the strength and stiffness of the concrete beams and columns were increased above that required by the equivalent lateral force analysis.
8.2.1 Structural Response

The results of the analysis are tabulated in Table 8.1. Table 8.1 shows the period of the fundamental mode of vibration for the structure, the maximum displacement during excitation, and the maximum ductility demands for the SIFCON hinges, concrete beam hinges, and column hinges. Results for six analyses are shown. These include:

1. Prototype structure as designed from the equivalent lateral force analysis exposed to the base Elcentro earthquake record.
2. Prototype structure with concrete beams and columns over designed by a factor of 30% and 50% respectively. The structure was exposed to base Elcentro record.
3. Prototype structure with concrete beams and columns over designed by a factor of 30% and 50% respectively. The structure was exposed to Elcentro earthquake record scaled to 150% of the base record.
4. Prototype structure with concrete beams and columns over designed by a factor of 30% and 90% respectively. The structure was subjected to the Elcentro earthquake record scaled to 150% of the base record.
5. Structure constructed exclusively from 6 ksi concrete with 24 inch long plastic hinges place at each ends of the girders. Beams and columns were over designed by a factor of 30% and 50% respectively. The structure was exposed to the Elcentro record scaled to 150% of the base record.

Analysis ‘1’ showed more hinging in the concrete beams and columns than in the SIFCON. It appeared that the softness of the SIFCON led to larger beam end rotations, and distributed the moment to the concrete beams and columns. Deformation compatibility at the SIFCON-to-concrete connection required the stiffer reinforced concrete to resist most of the earthquake forces. To reduce concrete beam and column yielding, larger concrete beams and columns were required. Analysis ‘2’ showed that increasing the strength and stiffness of the concrete beams 30% reduced concrete beam ductility demand below 1. However, even with a 50% stronger and stiffer column, the concrete column ductility demand decreased to only 3.2. Analysis ‘3’ was performed to
model the structural behavior when exposed to very large seismic activity. The results show that the concrete beam ductility demand increased to 1, while the ductility demand required of the SIFCON and concrete column increase to 3.3 and 6.3 respectively. The ductility demand for the columns occurs near adjacent to the beam-column joint and increased more than for the SIFCON hinges. Analysis ‘4’ indicates that the ductility demand on the columns cannot be practically reduced below 1. Even if 90% over designed, the ductility demand on the columns is still over 2. Analysis ‘5’ shows the behavior of the plane concrete structure when exposed to severe seismic forces. Stiffer concrete leads to a shorter natural period, which increases inertial forces. The result is larger beam and column ductility demands.

8.2.2 Hysteretic response
Numerous moment curvature hysteresis curves were plotted from the analysis. Figure 8.3 shows one of these plots. The Takeda model input parameters for this response are shown on the plot. Each plot compared favorably to the response exhibited during testing. The largest variation between the experimental behavior and the Takeda prediction is the pinching exhibited by the test specimens. However, the effect of this error is minimized by the fact that the analytical model indicates a maximum ductility demand of 3.4. With such limited nonlinear, post yield action, the importance of an accurate hysteretic model is arguable. In the opinion of the author, the Takeda model, with the parameters used in this analysis, adequately models the hysteretic behavior of SIFCON hinges at ductilities less than 5. There are more inaccuracies and omissions associated to the overall structural model than with the hysteretic model applied to the SIFCON hinges.

8.3 Summary
Reinforced SIFCON flexural members are approximately 50% as stiff as comparably sized reinforced concrete members. The stark contrast in rigidity of the two materials can lead to difficulties when used simultaneously in a structure. The analysis showed that
behavior is not necessarily improved by using SIFCON hinges in concrete moment frames. SIFCON hinges can actually increase ductility demand on the reinforced concrete regions of a structure. When using SIFCON hinges the reinforced concrete regions must be over designed to decrease the ductility demand on the concrete regions and force yielding in the SIFCON regions. Although yielding can be eliminated within the reinforced concrete beams, it is not practical to eliminate yielding in the reinforced concrete columns.

The nonlinear analysis resulted in a curvature demand of approximately 0.006. The average maximum curvature capacity of test specimens was approximately 0.01. Thus, the SIFCON hinges tested exhibited approximately twice the curvature capacity required by the analysis. It is important to note that the prototype structure was intentionally undersized to promote nonlinear behavior during the time-stepped analysis. Consequently, it is unlikely that curvature demands of 0.005 would be placed on a real structure that meets the UBC drift limitations. However, as a result of the softness of SIFCON hinges, it would be very difficult to meet the drift limitations of the Uniform Building Code using reinforced SIFCON hinges. The UBC allows the drift limitations to be exceeded if a higher order analysis is performed. Thus, a higher order analysis, including second order effects, will be required when using reinforced SIFCON hinges in concrete moment frames.
Table 8.1: Nonlinear, time step prototype analysis summary.

<table>
<thead>
<tr>
<th>Structure</th>
<th>Analysis number</th>
<th>Design Magnification Factor</th>
<th>Fundamental Period (sec)</th>
<th>Top Floor Displacement $\Delta_{\text{max}}$ (inches)</th>
<th>Maximum Curvature Ductility Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Beams</td>
<td>Columns</td>
<td>Earthquake</td>
<td></td>
</tr>
<tr>
<td>Prototype</td>
<td>‘1’</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>3.3</td>
</tr>
<tr>
<td>Prototype</td>
<td>‘2’</td>
<td>1.3</td>
<td>1.5</td>
<td>1.0</td>
<td>3.1</td>
</tr>
<tr>
<td>Prototype</td>
<td>‘3’</td>
<td>1.3</td>
<td>1.5</td>
<td>1.5</td>
<td>3.1</td>
</tr>
<tr>
<td>Prototype</td>
<td>‘4’</td>
<td>1.3</td>
<td>1.9</td>
<td>1.5</td>
<td>3.0</td>
</tr>
<tr>
<td>Concrete</td>
<td>‘5’</td>
<td>1.3</td>
<td>1.5</td>
<td>1.5</td>
<td>2.6</td>
</tr>
</tbody>
</table>
Figure 8.1: Elcentro and North Ridge ground acceleration records.

- a) S00E component of May 1940 Elcentro

- b) S90E component of January 1994 Northridge

Figure 8.1: Elcentro and North Ridge ground acceleration records.
Figure 8.2: Beam and column model used in nonlinear analysis.

Figure 8.3: Takeda hysteresis model output from Ruaumoko.
9 SUMMARY AND CONCLUSIONS

9.1 Summary of Research

Efforts have been made to improve the seismic resistance of reinforced concrete moment frames using hinges for over 30 years. Such a hinge must exhibit stable, ductile, post yield cyclic behavior. Ordinary reinforced concrete is not well suited to this performance criteria. However, recent advancements in high performance, fiber reinforced concrete hold potential for use in flexural hinge regions for reinforced concrete moment frames. One such material is slurry infiltrated fiber concrete (SIFCON).

This research investigated the use of precast reinforced SIFCON flexural hinges to increase the seismic resistance of reinforced concrete moment frames. The main thrust was to investigate the effect of different variables on the nonlinear, cyclic, flexural behavior of these hinges, and to determine how to best optimize hinge performance. The variables investigated include:

1. SIFCON compressive strength.
2. Reinforcing yield and ultimate tensile strength.
3. Reinforcing yield and ultimate tensile strain.
4. Reinforcing geometry.
5. Ratio of strength provided by reinforcing steel to that provided by SIFCON.

In addition to optimizing hinge behavior, a conceptual analysis was performed to determine the compatibility of using reinforced SIFCON hinges in concrete structures. In this portion of the research, a time-stepped, nonlinear analysis was performed on the prototype structure. The analysis had two purposes. The first purpose was to determine the resistance of the prototype structure to severe seismic loading. The second purpose of the analysis was to determine the compatibility of using reinforced SIFCON hinges at the ends of reinforced concrete beams.
Seven 10”x 26”x 26” long reinforced SIFCON hinge specimens were designed and fabricated, then tested under quasi-static cyclic loading. All specimens were fabricated using between 9 and 11%, by volume, Dramix 30/50 fibers, made by the Bekaert Corporation. Grade 60, Grade 75, and ASTM A722 (Dywidag) bars were used, in combination with three different SIFCON compression strengths. Additionally, various end connection details were used in testing three different reinforcing arrangements. Two specimens failed in the connection regions, one specimen failed in shear, and one specimen failed as the result of buckling of the compression reinforcing.

Ductilities never before achieved were recorded in this study. The maximum curvature ductility achieved was 26.4 over a 4-inch long interior region of a specimen. The curvature ductility of this hinge specimen, when taken over the full 26 inch length, was 10.5.

9.2 Conclusions and Observations

Based upon the results of this research, the following conclusions can be drawn.

9.2.1 SIFCON Material Behavior

Although the basic behavior of SIFCON was not a significant aspect of this research, several conclusions can be made based upon the research conducted.

1. Young’s modulus for unreinforced SIFCON in compression is between 1800 and 2000 ksi for SIFCON using a 7 ksi slurry and 10% volume fraction of Dramix ZL 30/50 fibers oriented parallel to loading. The same SIFCON using a 3.5 ksi slurry resulted in a Young’s modulus of 1000 ksi. This is approximately 50% less than that of comparable strength of normal weight concrete.

2. Fiber orientation is the most significant variable affecting SIFCON behavior. Size effects and fabrication techniques greatly effect fiber orientation. Two individuals following the same procedure will likely produce elements with notably different
material properties. This suggests that caution should be exercised when using analytical models to predict the behavior of SIFCON structural members.

3. Strain at maximum compressive stress is at least 0.5% and sometimes exceeds 15% strain. Several specimens were tested in excess of 15% strain without reaching their maximum compressive stress.

4. Strain at maximum tensile stress exceeds the yield strain of mild reinforcing steel.

9.2.2 SIFCON Hinge Behavior

This research demonstrated that reinforced SIFCON hinges can exhibit superior performance as compared to reinforced concrete hinges. Many problems encountered with reinforced concrete hinges do not occur when using SIFCON hinges. There are three primary advantages of using reinforced SIFCON flexural hinges in place of reinforced concrete hinges.

1. Greater shear strength and toughness prevent shear sliding on through-depth flexural cracks in reinforced SIFCON. In contrast, reinforced concrete hinges develop a through-depth flexural crack. As loading progresses, sliding occurs on this plane, quickly degrading structural integrity. However, the SIFCON hinges tested in this program rarely developed any through-depth cracks. If a through-depth flexural crack developed, it never opened enough to degrade the shear capacity of the section.

2. Compression strain capacity does not limit ductility of reinforced SIFCON hinges. However, reinforced concrete hinge ductility is limited by concrete compression strain capacity. In reinforced concrete hinges, concrete spalling and rupture usually occurs prior to tensile steel rupture. As a result, section strength is significantly decreased. The ability of SIFCON to undergo compressive strains in excess of 15% enables SIFCON hinges to develop the full ductility of the reinforcing steel. As a result, properly designed SIFCON hinge ductility is limited to the ductility of the reinforcing steel. Reinforced concrete hinge ductility is limited by the compressive strain capacity of the concrete.

3. Although SIFCON enables the reinforcing to undergo cyclic yielding without buckling, minimal confining steel may be required. Reinforced concrete hinges
require longitudinal and transverse confining steel to not only confine the flexural reinforcing, but also to confine the concrete core.

Other conclusions on SIFCON hinge behavior and compatibility are as follows.

1. Reinforced SIFCON hinges can withstand stable flexural yielding up to curvature ductilities of 10.5 when distributed over a yield region equal to the specimen depth. A curvature ductility of 26.4 was achieved over a 4 inch long region of one specimen.

2. Distributed yielding and distributed damage is a key design factor in hinge performance. This is true for reinforced SIFCON or reinforced concrete hinges.

3. Developing more than one primary failure plane is very difficult. However, a very good distribution of cracking and damage can be achieved prior to developing a singular primary failure crack.

4. Comparing energy absorption to previous research is difficult. Different loading patterns and member sizes are just two variables to consider when comparing hinge test results. Additionally, much of the previous research reported load deformation behavior, and not moment curvature results. However, visual comparisons to moment curvature and load displacement plots indicate that the specimens tested in this research exhibited at least 30 percent more energy absorption than previously reported results for reinforced concrete, fiber reinforced concrete, and reinforced SIFCON hinges.

5. In most of the hinges tested in this study, the tensile reinforcing yielded through the full length of the hinge. As a result, the whole hinge contributed to the ductility of the specimen. However, most of the ductility was the result of behavior within a small length of hinge—always within four inches of only one side of the primary failure crack.

6. SIFCON flexural members are much stronger than concrete members of the same size. A SIFCON beam with reinforcement ratio ($\rho$) of 0.006 can withstand approximately the same yield moment as the same sized concrete beam with a
reinforcement ratio of $0.5\rho_b$. Another way to state this is that a SIFCON beam with twice the ACI minimum reinforcing ratio exhibits comparable yield moments to that of a concrete beam with a reinforcing ratio of $0.5\rho_b$. This capacity difference must be considered when using SIFCON members adjacent to reinforced concrete members.

7. Young’s modulus of SIFCON in compression is approximately one-half that of comparable strength concrete. This results in the flexural stiffness of SIFCON, as determined by the slope of the moment curvature plots, equal to approximately one-half that of comparable strength concrete beams. SIFCON flexural members can withstand approximately twice the pre-yield rotation of a comparably reinforced concrete member.

8. Moment of inertia, at yield, for SIFCON specimens tested in this study is approximately 75% to 85% of $I_g$ (gross moment of inertia). All specimens tested in this study were lightly reinforced. Thus, $I_y = 0.75 I_g$ is a lower bound for practically reinforced SIFCON flexural members. These values are based upon the slope of the moment versus curvature plots.

9. SIFCON tensile strain at maximum tensile stress is approximately twice the yield strain of mild reinforcing steel. Thus, reinforced SIFCON hinges do not exhibit significant flexural cracking until after the reinforcing steel has exceeded its yield strain. Again, fiber orientation greatly affects the tensile properties of SIFCON.

10. Data suggests that reinforced SIFCON hinges do not necessarily reach yield moment when the reinforcing steel reaches yield. The reinforcing strain in several specimens exceeded yield strain prior to reaching the yield moment of the SIFCON hinge. Other hinge specimens, however, exhibited yield points occurring simultaneously with yield of the reinforcing steel. Strain gage readings indicated that reinforcing bars exceeded yield strains by 10 to 50 percent at the onset of flexural yielding. Analytical moment curvature predictions indicate the hinge flexural yield point is reached when the tensile reinforcing reaches yield strain. This may be why yield and ultimate moment predictions were up to 13 percent lower that those observed in testing.
11. When SIFCON compression strength exceeded 6 ksi, few cracks developed prior to yielding of the tensile reinforcing. Significant specimen cracking did not develop until the reinforcing reached strains in excess of 6000 microstrain (approximately 3 times greater than the yield strain).

12. The best hinge performance was achieved using Grade 60 reinforcing steel. One specimen using Grade 75 reinforcing failed due to compression buckling of the reinforcing. The embedded end plate anchorage limited ductility of specimens using Dywidag reinforcing. It is possible that better results can be achieved using higher strength reinforcing if embedded anchorage plates, and buckling of compression reinforcing can be avoided. However, it is clear that specimens using Grade 60 reinforcing exhibit more stable failure, which is a desirable characteristic of plastic hinges.

13. Hinges using Grade 75 reinforcing steel produce better distributed cracking, but compression buckling of the reinforcing prevented testing up to tensile rupture of the reinforcing. Energy absorption data indicates that much more energy is absorbed when using Grade 60 reinforcing. However, this may be a direct result of the premature failure of the specimen using Grade 75 reinforcing.

14. When using high strength reinforcing, damage is more extensive prior to reinforcing yield than with Grade 60 reinforcing. High strength reinforcing exhibits higher yield strains. Thus, the SIFCON hinge must undergo larger tensile strains prior to reaching the yield point of the reinforcing. Larger tensile strains produce larger and more distributed cracks.

15. Hinges using larger bars with weaker SIFCON may provide the best overall performance. One specimen tested matched this description, but failed due to diagonal shear cracking. It is important to note that calculations did not indicate the specimen was overstressed in shear. It is believed that problems associated with the manufacturing technique led to diagonal fiber orientation, and thus, the shear failure.

16. No shear reinforcing is required for lightly reinforced SIFCON specimens. This is true only if primary fiber orientation is parallel to the longitudinal axis of the flexural member.
17. Transverse ties should be used to prevent buckling of compression reinforcing. Minimal cover and longitudinal fiber orientation can lead to loss of confinement at advanced stages of cracking. This is especially true when using high strength reinforcing. Had transverse ties been used in the Grade 75 reinforcing specimen, it may have exhibited the best performance.

18. The Takeda model is an adequate representation of the hysteretic behavior of reinforced SIFCON hinges.

19. Reinforced SIFCON hinges can elongate up to 3 percent as a result of nonlinear cyclic loading. The average specimen elongation was 2.3 percent.

20. The use of embedded end plates to anchor reinforcing in the middle of hinges (as in specimen H146D) is detrimental to hinge ductility and energy absorption. Embedded plates promote the development of localized compression fields on tension faces of flexural members. Localized compression fields prevent good distribution of cracking and damage—resulting in less than optimal behavior.

21. When designing reinforced SIFCON hinges, the effect of localized reinforcing stresses on overall hinge behavior must be considered.

9.2.3 Structural Behavior
The following conclusions can be made concerning the effect of using SIFCON hinges in reinforced concrete moment frames.

1. Good hinge-to-concrete frame connection details are critical to realizing the full performance of SIFCON hinges. Connection capacity was a significant concern in this research project. A few full-scale connections have been designed for lightly reinforced SIFCON hinges. As a result of the inherent strength of SIFCON, it is doubtful that heavily reinforced SIFCON hinges can be adequately connected to adjacent concrete beams.

2. Stiffness differences between concrete and SIFCON must be considered when designing for SIFCON hinges. Interior concrete beams must be over designed by a factor of 30% in order to force yielding into the SIFCON hinges and out of the
concrete beams. Concrete column hinging could not be prevented with reasonably over-designed columns. However, column curvature ductility demand was decreased to 2.5 from 6.3 by using a 90% over-designed column in place of a 50% over-designed column.

3. SIFCON hinge elongation may significantly effect the performance of concrete moment frames utilizing SIFCON hinges. Consequently, hinge elongation must be considered when modeling a structure using SIFCON hinges. Hinge elongation will push columns away from each other, resulting in compression forces on the hinges. This could enhance hinge performance.

4. The nonlinear time-step analysis of the prototype structure indicated that columns demand as much ductility as beams. Reinforced SIFCON hinges can be used effectively in beams and columns.

9.3 Recommendations for Future Research

This project was initiated with the intent of optimizing reinforced SIFCON hinge performance by investigating the effect of different variables and concepts of design. In the process of answering questions, many more unknowns surfaced. The following is a list of topics that require further research to adequately address the unknown issues associated with using SIFCON hinges in concrete moment frames.

1. SIFCON compression and tension behavior is highly variable. The behavior depends upon fiber type, fiber orientation, and slurry strength. Results from this project indicate that fabrication technique, specimen size, and specimen shape effect fiber orientation. There is some question as to whether 4”x4” SIFCON prisms adequately reflect the compression behavior of 10” wide by 16” deep SIFCON hinges. More research is needed on basic SIFCON behavior. Present knowledge enables yield and ultimate moment predictions within approximately 20 percent of observed strengths. It is likely that more accurate predictions are needed before precast SIFCON hinges can be used effectively.
2. This research only considered a few possible issues that need to be addressed when using SIFCON hinges in concrete moment frames. More in-depth analysis is required to fully explore these and other issues.

3. A research program is needed to determine the effect of slabs on the overall performance of reinforced SIFCON hinges in moment frames.

4. Different types of reinforcing geometry need to be investigated. Sloped reinforcing, or straight, unhooked bar terminations may lead to significant improvements in SIFCON hinge behavior.

5. A test method is needed for determination or verification of fiber orientation in SIFCON specimens.

6. Full-scale connections need to be tested prior to using SIFCON hinges. There is a great potential for removing damaged hinges and replacing them with new hinges after earthquake damage has occurred.


11 APPENDIX
11.1 Moment Curvature Program

This program calculates the moment curvature data for a pure SIFCON region with up to 20 layers of reinforcing.

Program modified on 6-13-98 to handle up to 10 layers of steel.
Program modified on 7-30-98 to print stress in extreme SIFCON fiber on moment curvature plot (stress is plotted to 10X scale).
Program Modified on 6-4-99 to write data files for SIFCON compression and tension curves (removed comments and recompiled).

PROGRAM SIFCONM

DIMENSION DEPTH(500), STRAIN(500), SIFSTRESS(500), FORCE(500),
+ DEPTHM(500), FORCEMAX(500), AS(20), D(20),
+ FY(20), STY(20), STSH(20), FULT(20), STULT(20),
+ ESO(20), ESH(20), IFLAG2(10)
CHARACTER*15 INPUT, OUTPUT, OUTPUT2
WRITE(*,*) 'ENTER THE INPUT FILE NAME.'
READ(*,'(A15)') INPUT
WRITE(*,*) 'ENTER THE MOMENT DATA OUTPUT FILE NAME.'
READ(*,'(A15)') OUTPUT
WRITE(*,*) 'ENTER THE CROSS SECTION OUTPUT FILE NAME.'
READ(*,'(A15)') OUTPUT2
OPEN(20,FILE=INPUT,STATUS='OLD')
OPEN(50,FILE='MIRROR.OUT')
OPEN(21,FILE=OUTPUT)
OPEN(22,FILE=OUTPUT2)
OPEN(30,FILE='COMP.DAT')
OPEN(31,FILE='TENS.DAT')
C OPEN(32,FILE='REBAR.DAT')
C
C B = SECTION WIDTH (INCHES)
C H = SECTION HEIGHT (INCHES)
C D = DEPTH TO BOTTOM REBAR FROM TOP FIBER (INCHES)
C N = NUMBER OF LAYERS SECTION IS DIVIDED INTO
C AS = AREA OF BOTTOM STEEL
C D1 = DEPTH TO TOP REBAR FROM TOP FIBER (INCHES)
C AS1 = AREA OF TOP STEEL
C FCMAX = MAX COMP STRESS (KSI)
C STCMAX = STRAIN AT FCMAX
C FCINF = COMP STRESS AT INFLECTION POINT (KSI)
C STCINF = STRAIN AT FCINF
C FCPL = COMP PLATEAU OR PLASTIC STRESS (KSI)
C ECO = INITIAL COMPRESSION MODULUS (KSI)
C FTMAX = MAX TENS STRESS (KSI)
C STCMAX = STRAIN AT FTMAX
C FTPL = TENSILE PLATEAU STRESS (KSI)
C STTULT = ULT TENSILE STRAIN OF SIFCON
C ETO = INITIAL TENSION MODULUS (KSI)

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C FCULT = \( f_c' \); MAX SLURRY COMP STRESS (KSI)
C VF = FIBER VOLUME FRACTION (%)
C FL = FIBER LENGTH (INCHES)
C FD = FIBER DIAMETER (INCHES)
C FY = STEEL YIELD STRESS (KSI)
C STY = STRAIN AT FY
C STSH = STRAIN AT ONSET OF STRAIN HARDENING
C FULT = ULTIMATE STEEL TENSILE STRESS (KSI)
C ESO = INITIAL REBAR MODULUS OF ELASTICITY (KSI)
C ESH = REBAR MODULUS AT STRAIN HARDENING (KSI)
C NLS = NUMBER OF LAYERS OF STEEL REINFORCING
C
READ(20,*) B, H, N, NLS
READ(20,*) FCMAX, STCMAX, FCINFL, STCINFL, FCPL, ECO
READ(20,*) FTMAX, STTMAX, FTPL, STTULT, ETO
READ(20,*) FCULT, VF, FL, FD
READ(20,*) D(I), AS(I)
FOR I=1, NLS
READ(20,*) FY(I), STY(I), STSH(I), FULT(I), STULT(I),
+ ESO(I), ESH(I)
CONTINUE
C ROUTINE TO CHECK COMP, TENS, AND REBAR CURVE
C
WRITE(30,*) 'SIFCON COMPRESSION STRESS / STRAIN CURVE'
WRITE(30,*) ' STRAIN STRESS (KSI)'
WRITE(30,*) '
WRITE(31,*) 'SIFCON TENSION STRESS / STRAIN CURVE'
WRITE(31,*) ' STRAIN STRESS (KSI)'
WRITE(31,*) '
WRITE(32,*) 'REBAR STRESS / STRAIN CURVE'
WRITE(32,*) ' STRAIN STRESS (KSI)'
C
DO 100, I=1,800
STSIF = FLOAT(I)/8000.0
CALL SIFCONC(FCMAX,STCMAX,FCINFL,STCINFL,FCPL,ECO,
CALL SIFCONT(FTMAX,STTMAX,STTULT,ETO,STRESSST,STSIF, FL,H,IFLAG1)
WRITE(30,*) STSIF, STRESSC
WRITE(31,*) STSIF, STRESST
CONTINUE
C
C      DO 110, I=-400,400
C        STSTEEL = FLOAT(I)/4000.0
C        CALL STEEL(FY,STY,ESO,STSH,ESH,FULT,STULT,STRESSS,STSTEEL,
C     +              IFLAG)
C        IF(IFLAG .EQ. 2) THEN
C          WRITE(32,*) 'REBAR RUPTURE; SECTION FAILURE'
C        ENDIF
C        WRITE(32,*) STSTEEL, STRESSS
C 110  CONTINUE
C
C     START ITERATION FOR MOMENT CALCS
C     DIVIDE SECTION INTO 100 SLICES
C     SUM MOMENTS ABOUT THE TOP OF THE SECTION
C     ASSUME A TENSILE STRAIN AT THE BOTTOM AND ASSUME AN X
C     CALCULATE TOP STRAIN BASED ON X AND BOTTOM STRAIN
C     CALCULATE THE TENSION AND COMPRESSION WITH THAT STRAIN PROFILE
C     IF C = T, THEN WE HAVE CONVERGANCE.
C     IF C <> T, THEN ADJUST X, LEAVE BOTTOM STRAIN ALONE, AND TRY AGAIN
C     IF BOTTOM STRAIN > SIFCON ULT TENSILE STRAIN, STOP THE PROGRAM
C     IF STEEL RUPTURES, STOP THE PROGRAM
C
ICOUNT = 0
DX = H / FLOAT(N)
DSTRAIN = STTMAX / 20.0
WRITE(21,502)
502  FORMAT(T2,'CURVATURE',T13,'MOMENT (K-FT)',T25,'TOP STL STRESS',
     T39,'BOT STL STRESS',T52,'100x SIFCON STRESS',T66,
     + '100x SIFC COMP STRESS')
WRITE(21,*)
X = 0.5*H
XOLD = 0.0
SMOMMAX = 0.0
5000 CONTINUE
ICOUNT = ICOUNT + 1
STBOT = DSTRAIN * FLOAT(ICOUNT)
IF(STBOT .GE. FLOAT(ICOUNT)) GOTO 6000
ITER = 0
C     X = 0.5*H
C     XOLD = 0.0

4000  CONTINUE
C     DIVIDE SIFCON SECTION INTO SLICES AND USING STRAIN
C COMPATIBILITY, FIND STRAIN, AND THEN STRESS IN EACH SLICE

ITER = ITER + 1
IF(ITER.GT.100000) THEN
    WRITE(*,*),'NO CONVERGENCE AT STBOT = ',STBOT
    WRITE(*,*),'NO CONVERGENCE AT NA = X =',X
    WRITE(*,*),'********** END PROGRAM **********'
WRITE(21,*)'NO CONVERGENCE AT STBOT = ',STBOT
WRITE(21,*)'NO CONVERGENCE AT NA = X =',X
WRITE(21,*)'********** END PROGRAM **********'
GOTO 6000
ENDIF
SMOMENT = 0.0
SFORCE = 0.0
SFORCEOLD = 0.0
C STBOT = STTULT
STTOP = X * STBOT/(H-X)
DO 200 I=1,N
   DXT = DX * FLOAT(I)
   C DXT = H / FLOAT(I)
   IF(I .EQ. 1) THEN
      STRAIN1 = STTOP
   ENDIF
   STRAIN2 = STTOP - DXT*(STTOP/X)
   AVGSTRAIN = (STRAIN1 + STRAIN2)/2.0
   STRAIN1 = STRAIN2
   IF(AVGSTRAIN .GE. 0.0) THEN
      CALL SIFCONC(FCMAX,STCMAX,FCINFL,STCINFL,FCPL,ECO,
                  + STRESSSIF,AVGSTRAIN)
      STRESS = STRESSSIF
   ELSE
      TSTRAIN = ABS(AVGSTRAIN)
      CALL SIFCONT(FTMAX,STTMAX,STTULT,ETO,STRESSSIF,
                   + TSTRAIN,FL,H,IFLAG1)
      STRESS = -STRESSSIF
   ENDIF
   IF(I .EQ. N) THEN
      EFSTRESS = STRESS*100.0
   ENDIF
   IF(I .EQ. 1) THEN
      SIFCOMPSTRESS = STRESS*10.0
   ENDIF
   DEPTH(I) = DXT - DX/2.0
   STRAIN(I) = AVGSTRAIN
   SIFSTRESS(I) = STRESS
   FORCE(I) = STRESS * DX * B
C COMPRESSION CAUSES POSITIVE MOMENT; CLOCKWISE ROTATION
C ABOUT THE TOP FIBER IS POSITIVE MOMENT.
C COMPRESSION FORCE IS CONSIDERED POSITIVE
SMOMENT = SMOMENT + ABS(FORCE(I) * (X - DEPTH(I)))
SFORCE = SFORCE + FORCE(I)
CONTINUE

C GET REINFORCING STRESS, STRAIN, AND FORCE
C STSTEEL = STRAIN IN STEEL
C STRESSS = STRESS IN STEEL
C COMPRESSION STRESS AND STRAIN IS POSITIVE
C FORCESTL = FORCE IN STEEL

IF(NLS .EQ. 0) GOTO 110
DO 120 I=1, NLS
STSTEEL = STTOP/X*(X-D(I))
CALL STEEL(FY(I),STY(I),ESO(I),STSH(I),ESH(I),FULT(I),
+ STULT(I),STRESSS,STSTEEL,IFLAG2(I))

C ONLY STOP THE PROGRAM IF IFLAG2 = 2 AFTER CONVERGENCE OF MOMENT

FORCESTL = AS(I) * STRESSS
IF(I .EQ. 1) THEN
FORCESTLT = FORCESTL
STRESSST = STRESSS
ENDIF
IF(I .EQ. NLS) THEN
FORCESTLB = FORCESTL
STRESSSB = STRESSS
ENDIF
SMOMENT = SMOMENT + FORCESTL * (X-D(I))
SFORCE = SFORCE + FORCESTL
120 CONTINUE
110 CONTINUE

C ************** CHECK FOR CONVERGENCE **************
C ************** CHECK FOR T = C; KIPS **************

write(50,*) 'iteration = ', iter
write(50,202)
202 FORMAT(T5,'X',T10,'SMOMENT',T20,'SFORCE',T30,'STRN TOP',
+ T40,'STRN BOT',T50,'STL F TOP',T60,'STL F BOT')
WRITE(50,201) x,abs(smoment/12.0),sforce,sttop,stbot,
+ forcestlt, forcestlb
+ T50,F7.3)

C ***** SET TOLERANCE TO 100th OF SECTION DEPTH *****
TOL = H/100
IF(SFORCE .GT. TOL) THEN
C TOO MUCH COMPRESSION FORCE; C > T; MUST DECREASE X
XNEW = X - ABS(X-XOLD)/2
IF(SFORCE .GT. 30.0) THEN
XNEW = X - (0.005*H)
XOLD = X
X = XNEW
GOTO 4000
ELSE IF(SFORCE .GT. 50.0) THEN
XNEW = X - (0.002*H)
XOLD = X
X = XNEW
GOTO 4000
ELSE IF(SFORCE .GT. 10.0) THEN
XNEW = X - (0.0005*H)
XOLD = X
X = XNEW
GOTO 4000
ELSE IF(SFORCE .GT. 5.0) THEN
XNEW = X - (0.0001*H)
XOLD = X
X = XNEW
GOTO 4000
ELSE
XNEW = X - (0.00001*H)
XOLD = X
X = XNEW
GOTO 4000
ENDIF
ELSE IF(SFORCE .LT. -TOL) THEN
C         C < T; MUST INCREASE X
C            XNEW = X + ABS(X-XOLD)/2
IF(SFORCE .LT. -50.0) THEN
XNEW = X + (0.005*H)
XOLD = X
X = XNEW
GOTO 4000
ELSE IF(SFORCE .LT. -10.0) THEN
XNEW = X + (0.002*H)
XOLD = X
X = XNEW
GOTO 4000
ELSE IF(SFORCE .LT. -5.0) THEN
XNEW = X + (0.0005*H)
XOLD = X
X = XNEW
GOTO 4000
ELSE IF(SFORCE .LT. -1.0) THEN
XNEW = X + (0.0001*H)
XOLD = X
X = XNEW
GOTO 4000
ELSE
XNEW = X + (0.00001*H)
XOLD = X
X = XNEW
GOTO 4000
ENDIF
ENDIF
C       CONVERGENCE; T = C; WRITE RESULTS AND GOTO NEXT STRAIN
C       INCREMENT.
C
C INPUT FLAG TO DETERMINE IF STEEL HAS RUPTURED AFTER A
C FULLY CONVERGED STEP. STEEL IS ALLOWED TO YIELD WITHIN
C AN ITERATION BUT NOT AFTER A CONVERGED STEP.
C CHECK FOR STEEL RUPTURE ; I.E. IF IFLAG2 = 2

DO 130 I=1,NLS
  IF(IFLAG2(I) .EQ. 2) THEN
    IFLAG3 = 2
  ENDIF
130 CONTINUE

CURVATURE = STTOP/X

IF(ABS(SMOMENT) .GT. SMOMMAX) THEN
  SMOMMAX = ABS(SMOMENT)
  XMAX = X
  FSTLTOPMAX = FORCESTLT
  FSTLBOTMAX = FORCESTLB
  SFORCEMAX = SFORCE
  DO 500 K=1,N
    DEPTHM(K) = DEPTH(K)
    FORCEMAX(K) = FORCE(K)
  500 CONTINUE
ENDIF

WRITE(21,501) CURVATURE, ABS(SMOMENT)/12.0, STRESSST, STRESSSB,
+ EFSTRESS, SIFCOMPSTRESS
501 FORMAT(T2,E10.3,T13,E10.3,T25,E10.3,T39,E10.3,T52,E10.3,T66,
+ E10.3)
WRITE(22,*), '=============================================='
WRITE(22,*), '       MOMENT = ', ABS(SMOMENT)/12.0
WRITE(22,*), 'N. A. (X in.) = ', X
WRITE(22,*), 'STEEL RUPTURE'
ENDIF

WRITE(22,*), 'SECTION DEPTH (in)       STRAIN    ,
+ 'STRESS (ksi)'
DO 300 J = 1,N
  WRITE(22,*), DEPTH(J), STRAIN(J), SIFSTRESS(J)
300 CONTINUE

IF(IFLAG3 .EQ. 2) THEN
  WRITE(21,*), 'STEEL RUPTURE AT STBOT = ', STBOT
  WRITE(21,*), 'CURVATURE, MOMENT=', CURVATURE, SMOMENT/12.0
  WRITE(22,*), 'STEEL RUPTURE AT STBOT = ', STBOT
  WRITE(22,*), 'CURVATURE, MOMENT=', CURVATURE, SMOMENT/12.0
  GOTO 6000
ENDIF
GOTO 5000

6000 CONTINUE

C     MAXIMUM TENSILE STRAIN HAS BEEN REACHED OR
C     REINFORCING HAS RUPTURED

C     PRINT OUT FORCES AND MOMENTS FOR MAX MOMENT

WRITE(22,*)

C     THIS SECTION ADDED TO KEEP TRACK OF THE NA. AND STRAIN PROFILE
C     AT MAXIMUM MOMENT (ADDED IN JAN - FEB 1998)

WRITE(22,*)
WRITE(22,*) '==================================================================='
WRITE(22,*)
WRITE(22,*) '  FORCES AT MAXIMUM MOMENT = ',SMOMMAX/12.0, 'KFT'
WRITE(22,*) 'NEUTRAL AXIS AT MAX MOMENT = ',XMAX, 'INCHES'
WRITE(22,*) 'DELTA X = ',DX, 'INCHES'
WRITE(22,*) 'SUM OF FORCES = ', SFORCEMAX
WRITE(22,*)
WRITE(22,*) 'DEPTH (inches)    FORCE (kips) '
WRITE(22,*) '--------------------------------------'
DO 510 I=1,N
WRITE(22,*) DEPTHM(I), FORCEMAX(I)
510  CONTINUE
WRITE(22,*)
WRITE(22,*) 'DEPTH TO TOP AND BOT STL. =',D(1),D(NLS), 'INCHES.'
WRITE(22,*) 'TOP COMP. STEEL FORCE = ', FSTLTOPMAX, 'KIPS.'
WRITE(22,*) 'BOT. TENS. STEEL FORCE = ', FSTLBOTMAX, 'KIPS.'

STOP
END

C        ====================================== 
C        =   SIFCON COMPRESSION SUBROUTINE    = 
C        ====================================== 

SUBROUTINE SIFCONC(FCMAX,STCMAX,FCINFL,STCINFL,FCPL,ECO,
+                   STRESSC,STSIF)
       IF(ICOUNT .NE. 1) THEN
A = ECO * STCMAX / FCMAX
P = FCMAX - FCPL
RM = 1.0/(1.0+LOG((FCINFL-FCPL)/P))
B = (RM - 1.0)/(RM*(STCINFL - STCMAX)**RM)
ENDIF
       IF(STSIF .LE. STCMAX) THEN
STRESSC = FCMAX*(1.0-(1.0-STSIF/STCMAX)**A)
ELSE
EXP1 = -B*(STSIF**RM)*((STSIF/STCMAX-1.0)**RM)
STRESSC = P*EXP(EXP1) + FCPL
ENDIF
ICOUNT = 1
RETURN
END

C==================================================================================
C = SIFCON TENSION SUBROUTINE =
C==================================================================================

SUBROUTINE SIFCONT(FTMAX,STTMAX,STTULT,ETO,STRESST,STSIF,FL,H, +
                   IFLAG1)
IF(ICOUNT .NE. 1) THEN
A = ETO*STTMAX/FTMAX
CK = 1.0
ENDIF
IF(STSIF .LE. STTMAX) THEN
STRESST = FTMAX*(1.0-(1.0-STSIF/STTMAX)**A)
IFLAG = 0
ELSE IF(STSIF .LE. STTULT) THEN
DELTA = H * (STSIF - STTMAX/2.0)
STRESST = FTMAX*(1.0-CK*2.0*DELTA/FL)**2.0
IF (STRESSLAST .LE. STRESST) THEN
STRESST = 0.0
ENDIF
IFLAG = 0
ELSE
STRESST = 0.0
IFLAG1 = 2
ENDIF
ICOUNT = 1
STRESSLAST = STRESST
RETURN
END

C==================================================================================
C = REINFORCING STRESS SUBROUTINE =
C==================================================================================

SUBROUTINE STEEL(FY,STY,ESO,STSH,ESH,FULT,STULT,STRESSS, +
                  STSTEEL,IFLAG)
C IF(IFLAG .EQ. 2) THEN
C STRESSS = 0.0
C GOTO 100
C ENDIF
IF(ABS(STSTEEL) .LE. STY) THEN
IFLAG = 0
ELSE IF(ABS(STSTEEL) .LE. STSH) THEN
STRESSS = ESO*STSTEEL
ELSE IF(ABS(STSTEEL) .LE. STSH) THEN
IFLAG = 0
ELSE IF(STSTEEL .LE. 0.0) THEN
STRESSS = 0.0
ENDIF
ENDIF
IFLAG = 0
ELSE
STRESSS = 0.0
IFLAG1 = 2
ENDIF
ICOUNT = 1
STRESSLAST = STRESST
RETURN
END
STRESSS = -FY
ELSE
  STRESSS = FY
ENDIF
ELSE IF(STSTEEL .LE. -STSH .AND. STSTEEL .GE. -STULT) THEN
  IFLAG = 0
  STRESSS = -(FY + ESH*(ABS(STSTEEL)-STSH)*
            (1.0-ESH*(ABS(STSTEEL)-STSH)/(4.0*(FULT-FY))))
ELSE IF(ABS(STSTEEL) .LE. STULT) THEN
  IFLAG = 0
  STRESSS = FY + ESH*(STSTEEL-STSH)*
            (1.0-ESH*(STSTEEL-STSH)/(4.0*(FULT-FY)))
ELSE
  STRESSS = 0.0
  IFLAG = 2
C       STEEL RUPTURE
ENDIF
100 CONTINUE
RETURN
END