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APPLICATION OF BAMBOO FOR FLEXURAL AND SHEAR REINFORCEMENT IN CONCRETE BEAMS

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APPLICATION OF BAMBOO FOR FLEXURAL AND SHEAR REINFORCEMENT IN CONCRETE BEAMS

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ABSTRACT

As the developing world is industrializing and people migrate to cities, the need for infrastructure is growing quickly and concrete has become one of the most widely used construction materials. One poor construction practice observed widely across the developing world is the minimal use of reinforcement for concrete structures due to the high cost of steel. As a low-cost, high-performance material with good mechanical properties, bamboo has been investigated as an alternative to steel for reinforcing concrete. The goal of this research is to add to the knowledge base of bamboo reinforced concrete (BRC) by investigating a unique stirrup design and testing the lap-splicing of flexural bamboo reinforcement in concrete beams. Component tests on the mechanical properties of Moso bamboo (*Phyllostachys edulis*) were performed, including tensile tests and pull-out tests. The results of the component tests were used to design and construct 13 BRC beams which were tested under monotonic gravity loading in 3 and 4-point bending tests. Three types of beams were designed and tested, including shear controlled, flexure controlled, and lap-spliced flexure controlled beams. The test results indicated that bamboo stirrups increased unreinforced concrete beam shear capacities by up to 259%. The flexural bamboo increased beam capacities by up to 242% with an optimal reinforcement ratio of up to 3.9%, assuming sufficient shear capacity. Limitations of the bamboo reinforcement included water absorption as well as poor bonding capability to the concrete. The test results show that bamboo is a viable alternative to steel as tensile reinforcement for concrete as it increases the ultimate capacity of the concrete, allows for high deflections and cracks, and provides warning of impending structural failure.

DEDICATION

I would like to dedicate this to my Lord and Savior, Jesus Christ. Anything I am ever able to accomplish in life is only by His grace. I pray that my life would bring honor to Him and glory to God until the day He calls me home.

> Blessed be the God and Father of our Lord Jesus Christ, who has blessed us in Christ with every spiritual blessing in the heavenly places, even as he chose us in him before the foundation of the world, that we should be holy and blameless before him. In love he predestined us for adoption as sons through Jesus Christ, according to the purpose of his will, to the praise of his glorious grace, with which he has blessed us in the Beloved. In him we have redemption through his blood, the forgiveness of our trespasses, according to the riches of his grace, which he lavished upon us, in all wisdom and insight making known to us the mystery of his will, according to his purpose, which he set forth in Christ as a plan for the fullness of time, to unite all things in him, things in heaven and things on earth.
>
> (Ephesians 1:3-10 ESV)

I would also like to dedicate this to my family who have supported and prayed for me from half the world away. Though far away from each other, we are unified in the Holy Spirit through the bond of peace in Christ! I love you all very much.

ACKNOWLEDGEMENTS

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Chapter 1 – INTRODUCTION

1.1 Research Overview

As the developing world is becoming more industrialized and people are migrating to the cities, the need for infrastructure is growing quickly and concrete has become one of the most widely used construction materials. The construction industry has expanded rapidly and is in a haphazard and disorganized state in many developing countries. This problem, coupled with many other critical challenges faced by developing countries has led to sub-standard and dangerous construction practices that have severe consequences when tested by natural disasters. The 7.0 magnitude earthquake that hit Port-au-Prince, Haiti in January 2010 was a shocking reality of the devastation occurring due to poor construction practices. One poor construction practice observed widely across the developing world is the minimal use of reinforcement for concrete structures. Concrete requires tensile reinforcement, traditionally steel rebar, due to its brittle and low tensile strength nature. In many cases however, little to no rebar is provided to reinforce the concrete. This is the result of a lack of experience and knowledge on the part of the contractors as well as the inability of the owner to afford steel rebar - the cost of rebar for a home can be very expensive relative to the average income.

The high cost of steel rebar as well as the increasing emphasis on sustainable construction materials has led researchers to investigate alternatives to steel reinforcement. One promising alternative for steel reinforcement is bamboo. Bamboo is part of the grass family and is currently categorized into over 1,200 species (Judziewicz et al., 1999). In many regions of the world, especially in India and Southeast Asia, bamboo is being used as a common construction material for scaffolding, housing and bridges. Bamboo's fast growth rate and high tensile strength make it a good candidate for reinforcing concrete.

The physical and mechanical properties of bamboo have been studied extensively. Bamboo reinforced concrete (BRC) has been studied at Pontifical Catholic University of Rio de Janeiro (PUC-Rio) in Brazil, Clemson University and the University of Texas Arlington in the United States and Fukuyama University and Kumamoto University in Japan, among other places. However, more practical demonstration is required before bamboo gains enough credibility to be used to reinforce concrete structures around the world.

The goal of this research is to add to the knowledge base of bamboo reinforced concrete (BRC) by focusing on a unique stirrup design and testing the lap-splicing of flexural reinforcement. Tests of the mechanical properties of bamboo were performed including the tensile strength and bond strength between bamboo and concrete. These results were used in the design and construction of bamboo cages to reinforce concrete beams. This research presents the results of tensile tests and pull-out tests on Moso (*Phyllostachys edulis*) bamboo harvested from the Clemson University Experimental Forest. Using the tensile and pull-out test results, 13 full-scale BRC beams were constructed and tested under monotonic gravity loading to assess the performance of bamboo as shear reinforcement, flexural reinforcement and lap-splicing. The preliminary test results show that bamboo is a viable alternative to steel as tensile reinforcement for concrete structures.

1.2 Research Problems

1. Variability of both physical and mechanical properties from one bamboo culm to another
2. Low bond strength between bamboo and concrete
3. Water absorbing quality of bamboo
4. Brittle failure of bamboo which may prevent ductile failure of BRC beams

1.3 Objectives and Scope of Research

Since there are over 1,200 species of bamboo with varying physical and mechanical properties, the results from this research may not apply directly to all species of bamboo. This research was conducted using Moso bamboo (*Phyllostachys edulis*) grown in Clemson, SC.

1.3.1 Tensile strength

1. Further investigate the tensile strength and tensile failure modes of Moso bamboo.
2. Determine the relationship of bamboo nodes (locations on the bamboo plant which separate the hollow segments and where the branches grow from) to bamboo tensile strength.

1.3.2 Bond strength

3. Investigate the effect of nodes and water proofing agents on the bonding between bamboo and concrete.

4. Determine the required development length needed to achieve full tensile capacity of the bamboo.

1.3.3 Bamboo reinforced concrete beams

5. Determine a design using bamboo reinforcement that addresses its brittle failure mode.
6. Investigate the shear capacity of bamboo reinforced concrete beams provided by closed-loop bamboo stirrups and determine the optimum stirrup spacing.
7. Investigate the flexural capacity of bamboo reinforced concrete beams with varying percentages of bamboo and determine the optimum percentage of flexural reinforcement.
8. Investigate the lap-splicing of flexural bamboo reinforcement and determine the minimum allowable lap-splice length.
9. Determine an efficient and practical method for constructing bamboo cages for reinforcing concrete in the field.

Chapter 2 – LITERATURE REVIEW

The following section presents a literature review on the topic of bamboo reinforcement for concrete in developing countries. The introduction of the literature review covers the properties of concrete and the function of steel as reinforcement. Construction problems in developing countries are addressed, with a specific look into Haiti. An investigation into the cause of building failures from the 2010 Haiti earthquake concludes that the lack of proper reinforcement due to its high cost was a major problem. This introduces bamboo as a low cost alternative to steel rebar.

The physical structure and mechanical properties of bamboo are introduced with an emphasis on bamboo properties relating to its use as reinforcement. The economic and environmental impacts of bamboo are briefly discussed as well as common applications of bamboo around the world.

The final sections of the literature review provide information on the past studies of bamboo reinforced concrete emphasizing the results from the test specimens and actual field construction of several bamboo reinforced concrete structures. Finally, various issues with bamboo reinforcement are presented.

2.1 Concrete as a construction material

Concrete is the most common, man-made, building material that is being used for many applications including the construction of residential houses, high-rise buildings, bridges, streets, and dams. As the developing world is becoming more industrialized and people are migrating to the cities, the need for infrastructure is growing quickly and concrete has become the primary construction material. Concrete has many advantageous properties including high compressive strength, durability, fire-resistance, low maintenance and energy efficiency.

However, one disadvantage of concrete is its relatively low tensile strength. For this reason, concrete must be reinforced with high tensile strength materials to carry the tensile loads. Masonry structures and mortar have similar properties to concrete, and must also be reinforced. The most common reinforcing material for concrete and masonry structures is steel rebar.

Rebar, also known as reinforcing steel bar, is commonly made of mild steel, although for special corrosion resistant applications it can be made of stainless steel. Most rebar is made from scrap steel from automobiles, farm equipment and other discarded steel products, which are processed into the final steel bars. The milling process is well monitored with high levels of quality control. This ensures uniform strength, size and shape from rebar to rebar.

The idea of reinforced concrete was developed in 1849 by Joseph Monier, a Parisian gardener. Monier made garden pots and concrete basins reinforced with an iron mesh. He patented the idea in 1867, displaying his discovery at the Paris exposition the same year. Monier soon realized the potential for steel reinforcement for concrete in other engineering applications including bridges, floors, arches and pipes (Joseph Monier, n.d.).

2.2 Construction problems in Haiti

As many developing countries are rapidly urbanizing, and their infrastructures are expanding, often times little oversight is given to the local contractors and engineering designs are not used in construction. A prime example of this is in Port-au-Prince, Haiti. The majority of buildings in Port-au-Prince consist of low-rise, non-engineered, concrete block masonry structures which are used for single-family dwellings and small businesses. The roofs and floors are generally reinforced concrete slabs, 4-6 inches thick, with a single layer of reinforcement. The concrete frames of the buildings are lightly reinforced with slender columns which contain unreinforced concrete masonry walls (International Code Council [ICC], 2010).

According to Disaster's Emergency Committee (DEC), around 86% of people in Port-au-Prince live in slum conditions of tightly packed, poorly-built, concrete buildings. Typical low income housing in Port-au-Prince can be seen in Figure 2.1.

Figure 2.1 Typical low income housing in Port-au-Prince
(AFP photo)

Most homeowners construct their houses themselves or hire local contractors who have little engineering experience. While most Caribbean countries use the Caribbean

Uniform Building Code (CUBiC), Haiti has no building code. Occasionally engineers will use U.S. or European building codes, but only voluntarily, as no codes are enforced by the government. In recent years, building permits have been required for some buildings, but construction inspections are extremely rare. As a result, most of Haiti's infrastructure is built without engineering code enforcement and without quality control of construction methods and materials (ICC, 2010). The consequences of the poor construction were fully realized after the earthquake in 2010.

2.3 Haiti earthquake

In January 2010, a 7.0 magnitude earthquake occurred 10 miles south-west of Port-au-Prince. According to the DEC, 3.5 million people were affected by the earthquake and over 220,000 are estimated to have died. Close to 200,000 houses were badly damaged and 100,000 houses were destroyed, leaving 1.5 million people homeless. The quake crippled the fragile infrastructure of Port-au-Prince destroying 60% of government buildings, including the national palace, and 80% of schools. At the peak, 1.5 million people lived in refugee camps at critical risk from storms, flooding and diseases. The cholera outbreak in October, 2010 left over 6,000 dead and thousands more infected (Haiti Earthquake Facts and Figures, 2010).

Immediately following the earthquake, international engineering teams traveled to Port-au-Prince to investigate the nature and cause of the structural failures. The catastrophic damage resulted from poor construction methods and materials, including improper use of steel reinforcement. Unreinforced concrete masonry walls were observed to have collapsed under out-of-plane loading. The damage from the earthquake can be seen in Figure 2.2.

Figure 2.2 Earthquake damage
(LA Times)

The average compressive strength of concrete in the United States ranges from 2,500 psi for residential structures to 4,000 psi and higher for commercial applications. According to ACI 318-11, concrete used for seismic design must have a compressive strength of 3,000 psi. In Haiti, the average compressive strength of concrete is 1,300 psi with a standard deviation of 530 psi according to a report by Georgia Tech (DesRoches, 2011). The high variability of concrete strength in Haiti stems from poor construction practices including improper mixing, inconsistent mix designs, poor aggregate quality and insufficient cement.

An article from National Society of Professional Engineers (NSPE) highlights some of the findings of several earthquake engineers. According to Peter Coats, a senior engineer who volunteered through Earthquake Engineering Research Institute (EERI), smooth rebar was the standard in Europe until the 1950s and 1960s. This rebar was used in Haitian construction until it was phased out around the year 2000, and as a result, the majority of reinforced buildings in Port-au-Prince were reinforced with smooth rebar having low bond strength to the concrete (Leiserson, 2010).

2.4 Rebar alternatives

The main deterrent to using rebar in Haitian construction is its cost. There is currently no production of rebar in Haiti, so all rebar must be imported. Being an island country also drives up the cost of imports. In a country where over 70% of the population lives on less than $2 USD per day, it would cost many months wages to purchase the required steel to properly reinforce a house. Rather than spend their income to reinforce their house, many spend their money on more immediate needs such as food and education and hope that an earthquake or hurricane does not happen.

The high cost of rebar as well as the increasing emphasis on sustainable construction materials has led researchers to investigate alternatives to steel reinforcement. One promising alternative to steel reinforcement is bamboo, which has been studied extensively at Pontifical Catholic University of Rio de Janeiro (PUC-Rio) in Brazil, Clemson University and the University of Texas Arlington in the United States, Eindhoven University of Technology in Holland and Fukuyama University and Kumamoto University in Japan, among other places.

2.5 Introduction to bamboo

2.5.1 Physical structure

Bamboo is part of the grass family and is currently categorize into over 1,200 species which can be divided into two main groups: woody and herbaceous bamboos. Herbaceous bamboos are commonly called pygmy bamboos and resemble grasses with broader leaves than woody bamboo. Woody bamboos are the more commonly known bamboos which can grow to extraordinary sizes (Botanical Features of Bamboo, 2011).

Bamboo grows from seeds or a rhizome system which provides the foundation for the bamboo plant underground. The underground bamboo structure can be seen in Figure 2.3. Bamboo shoots grow from the rhizome and become mature culms, which are the woody stems of bamboo. By the time bamboo culms appear, they have already reached their full diameter. Young bamboo culms are protected by a layer of sheaths which fall off as the culms mature. Most species of bamboo are hollow but are partitioned by diaphragms which are denoted by a ring around the culm. The diaphragm and outside ring form a “node” from which branches and protective sheaths grow. The portion of the culm in between nodes is known as the “internode.” Both the diameter and wall thickness of bamboo is largest at the base and gradually decreases from bottom to top. Internodal length however, is largest in the middle of the culm. Figure 2.4 labels the various portions of the bamboo plant (Janssen, 2000).

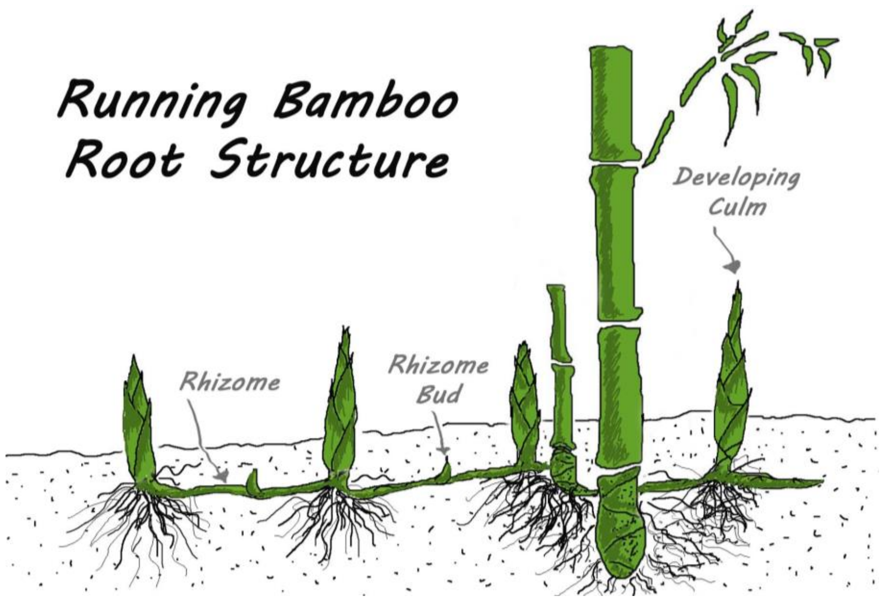

Figure 2.3 Bamboo structure
(Garden, 2012)

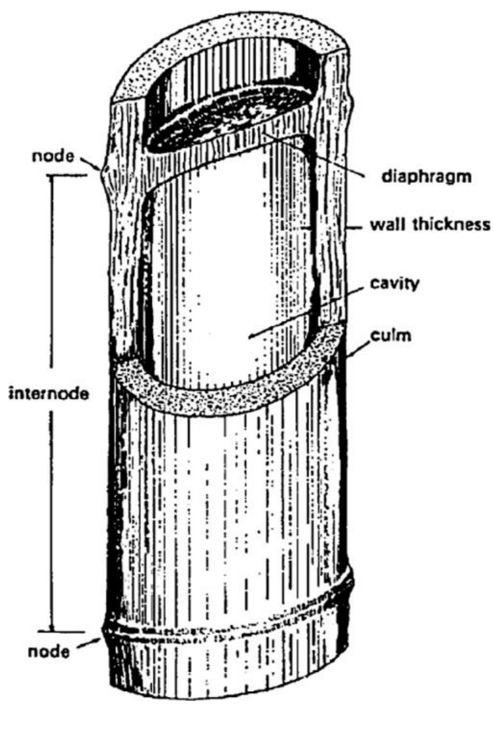

Figure 2.4 General features of a bamboo culm
(Subrahmanyam, 1984)

Bamboo has very fast growth rates and has set the Guinness World Record for the fastest growing plant, some species growing over 30 inches (76.2-cm) in day under ideal conditions. It can reach its full growth in just a few months and gain maximum strength in just a few years, giving it significant economic advantage over other construction materials (Ghavami, 2005).

2.5.2 Growth distribution

Bamboo has an extremely wide geographic distribution and can be found in a variety of habitats. Bamboo is indigenous to every continent except for Antarctica and Europe and can be found from 46°N to 47°S latitude. Figure 2.5 shows the geographic distribution of bamboo around the world. Bamboo also has a high altitudinal range and can grow from sea level to heights of 10,000 to 13,000 feet (3,000 to 4,000 meters) in the Himalayan and Andes mountain ranges (Botanical Features of Bamboo, 2011).

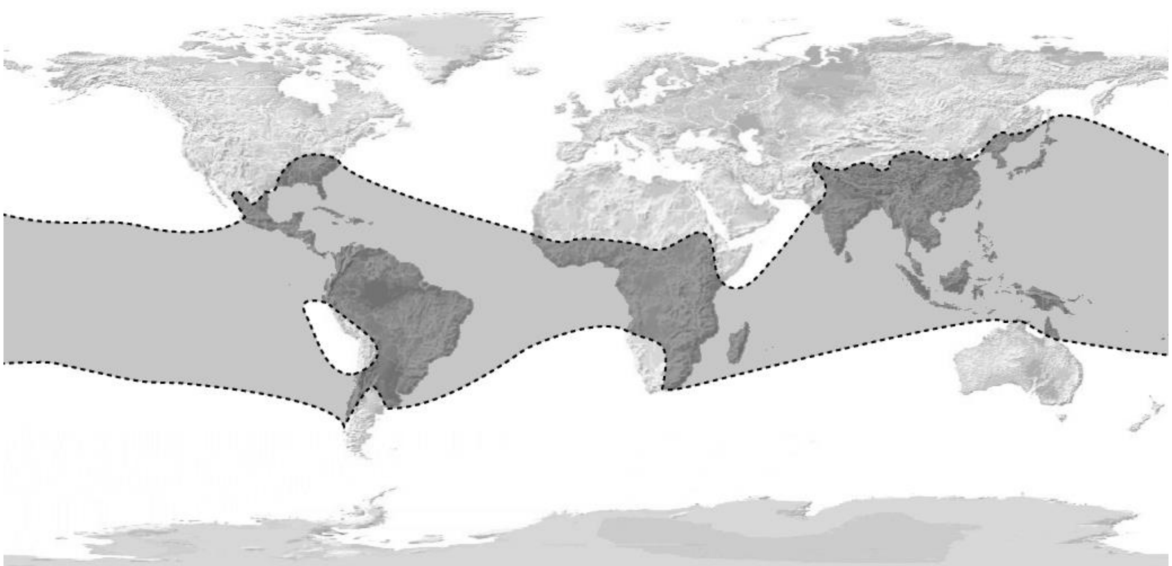

Figure 2.5 Geographic distribution of bamboo
(Wikimedia)

2.5.3 Natural durability

According to Janssen (2000), the service life of bamboo is lower than most woods partially due to the absence of certain preservative chemicals and also because of bamboo's hollow, thin walled structure. Even a small amount of destruction to the bamboo wall could seriously compromise the bamboo's structural strength. Janssen provides rough guidelines on the service life of untreated bamboo:

- 1-3 years in the open and in contact with soil;
- 4-6 years under cover and free from contact with the soil; and
- 10-15 years under very good storage/use conditions.

2.5.4 Harvesting and treatment

Proper treatment of bamboo can dramatically increase its service life. Janssen (2000) suggests that bamboo should be harvested in the season when starch content is low as this decreases the chance of fungal and insect attacks. Bamboo should be kept free from soil and dry, with sufficient space between culms for good air flow. Youssef (1976) found that green bamboo possessed only 60% of the tensile strength, and between 30% and 35% of the compressive strength compared to seasoned bamboo, which has a much lower moisture content. Brink and Rush (1966, 2000) provide guidelines for selecting bamboo to be used as reinforcement in concrete structures:

1. Use only bamboo showing a pronounced brown color. This will insure that the plant is at least three years old.
2. Select the longest large diameter culms available.

3. Do not use whole culms of green, unseasoned bamboo.
4. Avoid bamboo cut in spring or early summer. These culms are generally weaker due to increased fiber moisture content.

Bamboo's structure makes it difficult to be preserved. Both the outer and inner skin of bamboo are impermeable so preservatives can only enter through the conducting vessels which make up less than ten percent of the cross-section. These conducting vessels close within 24 hours of harvesting, so any preservation needs to be completed within this short time limit. Preservation can increase the service life of bamboo and includes traditional and chemical methods.

Traditional methods of treating bamboo are popular since they can be applied without any capital investment with the use of unskilled labor. Various traditional preservation methods include air curing, smoking, soaking and seasoning, and lime-washing (Janssen, 2000).

Chemical preservation methods are necessary for bamboo to be used in modern industry or in large-scale building projects. Effective and safe chemicals are based on the element boron. Two chemical preservation methods are the modified Boucherie process for whole green culms and dip-diffusion for split culms. In the modified Boucherie process, the preservative is passed through the culm vessels under pressure from one end of the bamboo to the other. This must be applied to bamboo within the first 24 hours of harvesting. After treatment, the culms must be dried under shade. In the dip-diffusion process, the bamboo culm is immersed in the preservative so that a slow diffusion can take place. This process only works on split bamboo strips because whole culms are impermeable and would not allow the preservative to penetrate (Janssen, 2000).

2.5.5 Mechanical properties

The mechanical properties of bamboo are described in the following section. It should be noted that there are many species of bamboo and the mechanical properties can vary greatly between species. For any values given in the following sections, an attempt will be made to name the associated bamboo species. If no species name is provided, the value given should be assumed to be an average for common bamboo species.

2.5.5.1 Fiber distribution

The outside of a bamboo culm wall is a thin, but very dense layer containing silica. Bamboo is composed of cellulose fibers together with vessels running in the longitudinal direction and surrounded by parenchyma. The cellulose fibers are strong and stiff and act as reinforcement in the parenchyma matrix. Bamboo is approximately made up of 40% fibers, 10% vessels and 50% parenchyma by volume (Janssen, 2000).

Ghavami (2005) describes bamboo as an orthotropic material with variations in mechanical properties along its radial, circumferential and axial directions. The fiber distribution on the cross-section of bamboo is more concentrated closer to the outside perimeter, according to the stress distribution caused by bending due to wind forces. The fiber distribution on a cross-section of bamboo can be clearly seen in Figure 2.6. According to Janssen (2000), the cellulose fibers in bamboo act as reinforcement to the culm structure. Amada (1997) has described the fiber distribution in the nodes to be chaotic, which is evidence for their brittle behavior. Amada also suggests that the nodes prevent buckling due to bending and also may help prevent axial cracks. It is evident that bamboo is much stronger parallel to the fibers and weaker in the perpendicular direction. Due to the structure of the bamboo culm, its strength is greatest at the base where there is a larger concentration of fibers and decreases towards the top.

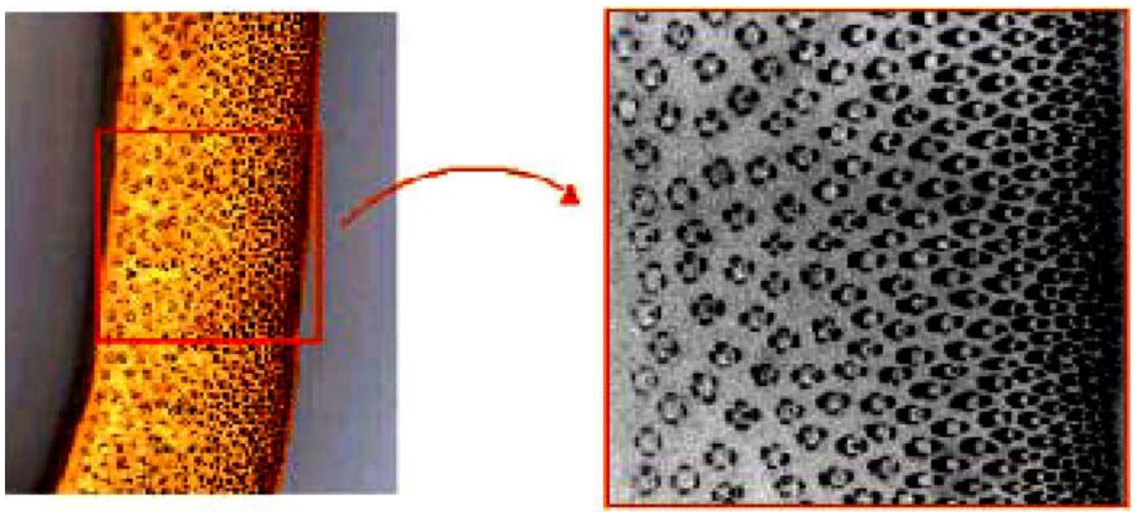

Figure 2.6 Non-uniform fiber distribution on bamboo cross section
(Ghavami, 2005)

2.5.5.2 Modulus of elasticity

According to Ghavami (2005), bamboo has a structural advantage over other engineering materials in terms of modulus of elasticity and density. Janssen (2000) has given the average modulus of elasticity of bamboo to be around 2500 ksi which is similar to the value given by Brink and Rush (1966, 2000).

A study by Khare (2005) showed the nodal region to have a brittle behavior, while the internodal region to have a more ductile behavior. The stress-strain curve of bamboo samples suggested that internodal regions elongated until reaching a limiting value, at which point the load was transferred to the node.

2.5.5.3 Tensile strength

The tensile strength of bamboo is known to be very high, but the average strength varies from species to species. When addressing bamboo's tensile strength it is important to differentiate between the culm tensile strength and the fiber tensile strength. A study by Cao and Wu (2008) showed fiber tensile strengths to range from 18 to 131 ksi (124 to 903 MPa) with an average strength of 53 ksi (365 MPa). Ghavami (2005) has listed the tensile strength of bamboo as high as 54 ksi (372 MPa), but it is unclear whether this is for the culm or fibers. Amada (1997) has concluded that the fiber strength for Moso bamboo is around 87 ksi (600 MPa) while the culm strength is only around 7 ksi (48 MPa).

A study done by Stamford University in Bangladesh showed tensile strengths of bamboo culm to be around 18 ksi (124 MPa). This study also described the difficulties of gripping the bamboo during the tensile tests and explained how spiraled wire wrapped around each end prevented slipping at the grips (Sabbir, Mamun & Fancy, 2012).

According to the International Standard Organization (ISO), the tensile strength of a node region is only 30% of an internode region (ISO, 2004). Results from Glenn (1950) and Khare (2005), also show that nodes are weaker than internodes.

ISO has developed a laboratory manual for determining physical and mechanical properties of bamboo. This manual covers testing methods for moisture content, specimen volume, shrinkage, compression, bending, shear and tension.

Wahab et al. (2012) used a variation from ASTM D 143-94, the Standard Test Methods for Small Clear Specimens of Timber, to test the tensile strength of bamboo parallel to the grain. Four different species of Gigantochloa were tested with tensile strengths ranging from 14.9 ksi to 17.7 ksi. The tensile strength of dried bamboo was found to be 20 ksi compared to 13 ksi for green bamboo (Wahab et al., 2012).

2.5.6 Environmental impacts

Bamboo is a natural plant that has many positive environmental impacts. According to Ghavami (2005), the energy required to produce steel is 50 times that of bamboo. The carbon footprint of bamboo is significantly less than that of rebar as an acre of bamboo can sequester up to 40 tons of carbon dioxide from the atmosphere (Bamboo & Global Warming, 2011).

According to Geiger (2006), bamboo produces about 20 times more timber than trees on the same acreage. Bamboo can be harvested in only 3 years compared to a fir tree which takes up to 40 years to be harvested and after being harvested, new bamboo can continue to grow from the same underground root structure. Bamboo's quick growth rates make it an ideal candidate for reforestation especially in areas where timber is cut and used for charcoal. Bamboo can also be used to make charcoal which burns more efficiently and cleanly than wood charcoal. Mingjie (2004), provides a manual for bamboo charcoal production and utilization.

The extensive root system of bamboo is extremely useful in combating soil erosion. An INBAR (International Network for Bamboo and Rattan) project in Allahabad, India planted bamboo to reclaim land whose topsoil had been depleted by the brick industry. After only five years, farmers could once again cultivate the land (Rosenberg, 2012).

2.5.7 Applications of bamboo

Bamboo is commonly known as ‘poor man’s timber’ because it is cheap, readily available, and is used in all aspects of life. Bamboo has been used since ancient times for a variety of applications including pulp and paper, construction and structural applications, furniture, weapons, food and even clothing.

Bamboo’s unique mechanical properties make it an ideal construction material. In many countries, bamboo is the primary material for house construction. INBAR estimates that one billion people live in bamboo dwellings. Scaffolding is a major use of bamboo around the world, but especially in India and Southeast Asia. In Hong Kong, many of the skyscrapers are constructed with bamboo scaffolding, some reaching up to 80 stories.

A bridge made from bamboo composites that can support a 16-ton truck has been built in China. The bamboo composites for this structure were formed into timber-like beams by a technique similar to cross laminated timber (CLT) in which many small strips of bamboo are arranged in alternating layers and glued together (Inman, 2007).

In 2004, the International Code Council (ICC) certified that Structural Bamboo Poles produced by Bamboo Technologies met International Building Code (IBC) standards. These structural poles are from the bamboo species *Bambusa Stenostachya* which is found in Vietnam. Despite positive advances made by Bamboo Technologies, bamboo still has a ways to go to be accepted by the IBC for modern construction.

2.6 Bamboo reinforcement

2.6.1 History

Bamboo reinforced concrete (BRC) has been studied at Pontifical Catholic University of Rio de Janeiro (PUC-Rio) in Brazil, Clemson University and the University of Texas Arlington in the United States and Fukuyama University and Kumamoto University in Japan, among other places. During the 1930's several experiments were performed in Germany and Italy testing the performance of cement concrete beams reinforced with bamboo. However, the earliest comprehensive information on bamboo reinforcement is found in a report carried out by Clemson Agricultural College under the direction of Professor H.E. Glenn (Glenn, 1950).

In 1943, Clemson Agricultural College of South Carolina[1] was awarded with undertaking the research of bamboo reinforcement in Portland cement concrete. This research was funded by the War Production Board (W.P.B.) during WWII in an attempt to find substitutes for steel reinforcement. The Clemson study consisted of laboratory research followed by field construction.

Ghavami has conducted much research on bamboo as a concrete reinforcement at PUC-Rio in Brazil. His work includes research of the mechanical properties of bamboo, water repellent treatment methods as well as investigation of bamboo reinforced concrete beams, columns and slabs (Ghavami, 2005).

[1] Clemson Agricultural College of South Carolina ceased to exist as of 1964; now it is known as Clemson University

Khare (2005) from the University of Texas Arlington conducted a study on the mechanical properties of Moso and Tonkin bamboo and investigated the performance of six bamboo reinforced concrete beams (Khare, 2005).

Terai and Minami (2012) from Fukuyama University in Japan presented their bamboo reinforced concrete research at the 15th World Conference on Earthquake Engineering (WCEE) in Lisbon, Portugal. Their research focused on the mechanical properties of bamboo, including the effects of alkali contact on bamboo durability and tensile strength as well as the bond strength of bamboo in concrete. Terai and Minami also tested the flexural failure of BRC slabs reinforced with whole bamboo culms (Terai & Minami, 2012).

Yamaguchi, Murakami, and Takeda (2013) from Kumamoto University in Japan presented on the flexural performance of BRC beams using bamboo as main rebar and stirrups. This presentation and corresponding paper was delivered at the Third International Conference on Sustainable Construction Materials and Technologies (SCMT) at the Kyoto Research Park in Kyoto, Japan.

2.6.2 Design principles for bamboo reinforced concrete

Brink and Rush (1966, 2000) outline several design principles for bamboo reinforced concrete. According to Brink and Rush, bamboo reinforced concrete design can be performed similarly to steel reinforcing design with the mechanical properties of bamboo substituted for the steel. Bamboo reinforced concrete design examples are provided for beams and girders, columns, ground-supported slabs, and walls.

2.6.3 Bamboo reinforced concrete beams

There have been many studies applying bamboo reinforcement to a concrete beam. The first well-documented research was conducted by the Clemson Agricultural College[2] who performed studies on rectangular beams, T-beams and slabs. The study concluded that the bamboo reinforcement in concrete beams increased the load capacity with increasing percentages of bamboo reinforcement up to an optimum value of three to four percent (Glenn, 1950). Very little literature is available on lap splicing bamboo. Glenn suggests to lap splice bamboo a minimum distance of 25 inches (64-cm) and to securely tie the splices.

Ghavani (2005) has performed several studies on bamboo reinforcement and concluded that the ultimate strength of a concrete beam reinforced with bamboo is approximately 4 times that of an unreinforced concrete beam of the same dimensions.

Khare (2005) conducted a study at the University of Texas at Arlington on the performance evaluation of bamboo reinforced concrete. This study comprised of tensile tests on three types of bamboo (Moso, Solid and Tonkin), followed by the preparation of six reinforced concrete beams with variation in *a/d* (shear span to depth) ratios, percentages of reinforcement, and bamboo type. Test results indicated that bamboo reinforcement increased the beam capacity by 250% as compared to the initial crack in the beam. A direct relationship was found between percentage of reinforcement and capacity; however, the beam with 4% bamboo reinforcement produced an over-reinforced failure mode.

[2] Clemson Agricultural College of South Carolina ceased to exist as of 1964; now it is known as Clemson University

Rahman et al. (2011) conducted tests on plain concrete beams as well as singly and doubly reinforced concrete beams. The results indicate that bamboo reinforcement increases the ultimate load carrying capacity of the beam by 2 for the singly reinforced beam and by 2.5 for the doubly reinforced beam. The maximum deflections of the singly and doubly reinforced beams were 4.5 and 8 times that of the plain concrete beam.

Hidalgo (1996) studied bamboo cables as reinforcement for concrete. The bamboo cables were made from woven strips of bamboo taken from the outer layer of the bamboo culm which is almost three times stronger than the inner part. The outer layer of the bamboo absorbs a minimum amount of water compared with the inner portion and therefore does not have to be waterproofed. In addition, the helical shape of the cables increase the bonding between the concrete and the bamboo. These benefits make the bamboo cables a promising method for using bamboo reinforcement which has been tested in slab foundations (Hidalgo, 1996). The application of the bamboo cables is further addressed in Section 2.6.5.

2.6.4 Bamboo reinforced concrete masonry shear walls

Moroz (2014) conducted tests on seven squat concrete masonry walls under quasistatic, in-plane, cyclic loading. One of the seven walls was reinforced with traditional steel reinforcement vertically and horizontally in the bond beams. The other six walls were reinforced with varying amounts of Tonkin cane bamboo reinforcement both vertically to resist flexural and sliding failure and horizontally to resist diagonal shear failure. The results indicated that bamboo reinforcement in concrete block shear walls enhanced the shear capacity and ductility compared to unreinforced masonry. The bamboo reinforced shear wall with four vertical cores reinforced with bamboo behaved very similarly to the steel reinforced shear wall with four vertical cores and three bond beams reinforced with steel.

2.6.5 Bamboo reinforced concrete structures

There are only a few application of bamboo reinforcement in concrete structures. The most extensive study was completed by the Clemson Agricultural College[3] which included the field application of laboratory research. This study consisted of the construction of three separate units comprising four structures. Further descriptions of each unit with designs and construction procedures can be found in the report by Glenn (1950).

Unit No. 1, also known as the "Planner Building", is 32 feet by 32 feet, and was used as a workshop for the making of pre-fabricated parts for Unit No. 3. Unit No. 1 was a box type structure of the conventional beam and girder design. The footings, piers, columns, girders, beams and floor slab were constructed with cast in place bamboo reinforced concrete. The walls were made entirely of precast concrete panels, and the roof comprised of both cast in place and precast concrete reinforced with bamboo. This building is still partially standing, although one side has collapsed. A few concrete samples with bamboo reinforcement were inspected and no bond between the bamboo and concrete was observed.

Unit No. 2 was constructed as the press box for the Clemson College Memorial Stadium. The stadium was partially designed by Professor H.E. Glenn who was also one

[3] Clemson University

of the primary investigators of the bamboo research. Unit No. 2 was three stories tall and constructed entirely with cast in place bamboo reinforced concrete. This unit was much more complex than Unit No.1 as it contained nearly every type of load-carrying member. The press box was 60 feet long by 18 feet wide and provided space for stairs and toilets. This unit was demolished when the stadium was expanded.

Unit No. 3 consisted of eight structures, only one of which was completed. This structure was a five-room residence constructed entirely out of pre-fabricated units of bamboo reinforced concrete. The residence was inhabited shortly after completion but it is uncertain whether this structure is still standing.

Brink and Rush (1966, 2000) prepared a report to assist field personnel in the design and construction of bamboo reinforced concrete. The information in the report was compiled from various research documents including the Clemson study and a study conducted at the U.S. Army Engineer Waterways Experiment Station in 1964. This report includes the selection and preparation of bamboo reinforcement, construction principles for bamboo reinforced concrete, design procedures and charts for bamboo reinforced concrete, and six design examples. It must be noted that the original report is not available to the public and the replicated document has not been verified for accuracy. Nevertheless, this document provides much useful information regarding bamboo reinforced concrete construction.

Another example of a bamboo reinforced house is found in Auroville, India. The house was designed and constructed by Alok Mallick in 2009. The roof and floor slabs of the house were constructed with bamboo reinforced concrete but the walls were made with a mud and coconut fiber mix reinforced with bamboo. The tensile strengths of various bamboo species were tested and a factor of safety of 50% of the average was used to determine the amount of bamboo reinforcement that would be required. All materials and labor was local, and the workers had to be trained for the construction of the bamboo reinforced floor and roof slabs. The house was still in excellent condition after two years of intense monsoons and hot summers, but the current condition of the house is unknown (Mallick, 2011).

Hidalgo (1996) utilized bamboo cables to reinforce the footing around a slab foundation for a prefabricated housing program in Guayaquil city. Ten years after the construction, the houses with the bamboo reinforced slab foundations did not show any cracks (Hidalgo, 1996).

2.6.6 Issues with bamboo reinforced concrete

Although bamboo has many good properties that make it a strong candidate as an alternative reinforcement, it has qualities which cause several issues when using bamboo as reinforcement for concrete elements. These issues are addressed in the following sections.

2.6.6.1 Environmental damage

Bamboo's low durability is a major drawback for structural use. Like timber, bamboo is susceptible to damage from the environment, insects and mold. Ghavami (2005) claims there is a positive relation between starch content in the bamboo and insect attacks. To reduce starch content, Ghavami suggests curing and treatment by immersion, heating or smoke. Janssen (2000) indicates that high moisture content increases the likelihood of fungal attacks and outlines various traditional and chemical preservation methods to protect the bamboo. The long term durability of bamboo encased in concrete is a concern, but Ghavami (2005) has addressed this issue. The first BRC beam tested at PUC-Rio in 1979 was exposed to the open environment in the university campus. It was observed that the bamboo reinforcement – treated against insects and waterproofed – was still in satisfactory condition after 15 years (Ghavami, 2005).

2.6.6.2 Low stiffness

The modulus of elasticity of bamboo is much lower than that of steel and is also lower than that of concrete which is approximately 2000 – 6000 ksi (14,000 – 16,000 MPa) depending on its compressive strength. In a technical paper on bamboo reinforced concrete pavements, Rolt (2008) concludes that bamboo cannot prevent load induced cracking because its modulus of elasticity is too low to reduce the tensile stresses that cause cracking. This conclusion is also confirmed by the Glenn (1950), Brink and Rush (1966, 2000), Janssen (2000), and Rahman et al. (2011). Sherwood (2008) explains how extensive flexural cracking could cause pre-mature shear failure, but this has not been addressed by previous studies on crack widths.

2.6.6.3 Alkaline resistivity

Janssen (2000) suggests that the alkaline environment of the concrete would deteriorate the bamboo over time, but Ghavami contradicts this theory. According to Ghavami, the first bamboo reinforced concrete beam tested at PUC-Rio was exposed to open air for 15 years. After this time it was observed that the bamboo segment of the beam reinforcement was still in satisfactory condition (Ghavami, 2005).

2.6.6.4 Differential thermal expansion

The differential thermal expansion of bamboo with respect to concrete poses several problems. Ghavami (2005) suggests that the differential thermal expansion may lead to cracking of the concrete during service life and research by Hebel, Heisel, and Javadian (2013) indicates that de-bonding will occur.

2.6.6.5 Moisture absorption

The tendency for bamboo to absorb moisture leads to several serious problems including pre-loading cracking and loss of bond strength. Like timber, the engineering properties of bamboo are highly sensitive to different moisture contents, absorbing or releasing moisture depending on its environment. Research has shown that bamboo can absorb up to 100% of its dry weight in water. The lateral expansion ranges from 2% to 5%, and the longitudinal expansion is around 0.05%. The moisture absorption is initially quite high, causing the bamboo to swell until reaching its fiber saturation point, after which there is almost a negligible change in volume (Subrahmanyam, 1984).

Jiang et al. (2012) tested the sensitivity of select mechanical properties of Moso bamboo to moisture content (MC) change and bamboo age. It was determined that bamboo age has little effect on the sensitivity of the tensile modulus and bending modulus to MC change (Jiang et al., 2012).

The problems of moisture absorption and cracking were observed by Moroz, Lissel and Hagel (2014) in their study of bamboo reinforced concrete masonry shear walls. One of their wall specimens had extensive pre-cracking which was attributed to the expansion of the horizontal bamboo during the curing period (Moroz, 2014).

According to Janssen (2000), untreated bamboo will absorb water from the concrete mix and shrink as the concrete cures. The shrinkage of bamboo can be four times that of concrete, and as the bamboo shrinks it will pull away from the concrete, resulting in little to no bond. This is illustrated in Figure 2.7, taken from Ghavami's paper (Ghavami, 2005)

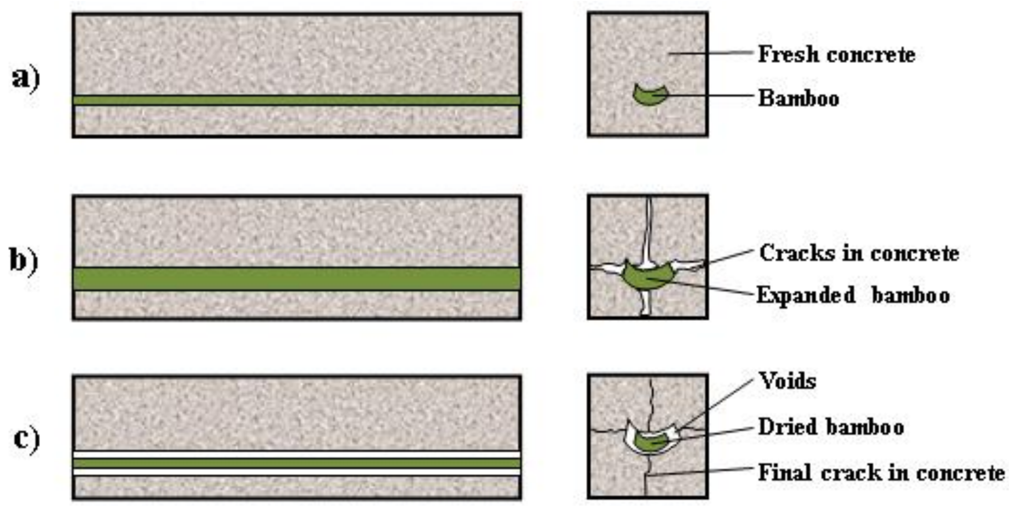

Figure 2.7 Behavior of untreated segment bamboo as reinforcement in concrete
(a) bamboo in fresh concrete, (b) bamboo during curing of concrete, and
(c) bamboo after cured concrete (adapted from Ghavami, 2005)

The bond stress of uncoated and water proofed bamboo has been determined by Sakaray, Togati, and Reddy (2012) and compared to 8-mm HYSD steel bars. The bond stress values of uncoated bamboo range from 0.9 to 1.74 N/mm^2 (131 to 252 psi) which was 4.7 times less than for the steel (Sakaray et al., 2012).

Kankam and Perry (1989) studied the variability of bond strength between bamboo and concrete. The presence of embedded nodes, surface roughening and the number of waterproofing coatings were the variables. They concluded that natural protrusions at the embedded nodes, waterproofing then sanding of the splints as well as higher concrete strengths led to greater bond strength. For 4-week seasoned bamboo splints with a single embedded node in the concrete, the ultimate bond strength non-waterproofed was 2.04 N/mm^2 (296 psi). The corresponding bond strength for waterproofed splints was 2.6 N/mm^2 (377 psi).

The presence of embedded nodes increased bond strength by 81 up to 257 percent. The application of waterproofing paint, followed by a dusting with river sand, increased the ultimate bond strengths by 12 and 27 percent, respectively, over the strength of specimens in which the paint was not used (Kankam & Perry, 1989). Kankam and Perry conclude the reason for this being the fact that the untreated bamboo strips swell after absorbing moisture from the wet concrete and subsequently dried and shrank when the concrete had hardened, with consequent reduction in bond. This confirms the conclusions of Ghavami (Ghavami, 2005).

Braga et al. (2010) experimented with improving the bamboo-concrete bond by means of nailing. In total eight beams were tested, two of them being reference beams reinforced with bamboo splints without pins and the remaining six beams being reinforced with nailed bamboo splints. Both steel and bamboo pins were used and the results from the nailed bamboo beams showed an increase in stiffness but a reduction in ultimate load capacity since the nails locally reduced the transversal section of the bamboo splint.

2.6.6.6 Variable mechanical properties

Since bamboo is a natural material its characteristics are extremely variable. This makes it difficult to specify design provisions for construction with bamboo reinforcement and any designs must have appropriate factors of safety.

2.6.7 Bamboo composite reinforcement

Dirk Hebel, an assistant professor of architecture and construction at the Future Cities Laboratory, in Singapore, is investigating a bamboo fiber composite as a substitute for steel reinforcement in concrete. Natural, untreated bamboo, has variable characteristics and is vulnerable to insect and fungal growth. One of the greatest problems with bamboo reinforcement is its poor bond strength to concrete which develops as the bamboo swells and shrinks as it absorbs moisture from the fresh concrete and dries. The resulting loss of bond strength minimizes its advantages as a reinforcement for concrete, since a good bond is required to transfer stresses from the concrete to the reinforcement (Cardno, 2014).

Hebel’s current bamboo composite comprises naturally processed bamboo fibers and an adhesive. The fibers, after being carbonized to minimize sugar content and then dried to reduce moisture content, bound together with a water-based resin. After being mixed with the adhesive, the strands are pressed into a mold to achieve the desired shape and thickness. The bamboo composite is approximately 80 percent bamboo and 20 percent adhesive. Advantages of the bamboo composite over natural bamboo include increased density, water resistance, durability, controlled thermal expansion, and the ability to be pressed into any shape. Through the manufacturing process, Hebel wishes to fully control the properties of the bamboo composite, so that it can be certified by local standardization organizations (Cardno, 2014).

Hebel envisions the bamboo composite could find a niche in the concrete market for application in which a noncorrosive reinforcement is required. Hebel also sees great potential for the bamboo fiber composite to replace steel in urban construction in developing countries. This would allow cities to produce their own construction materials and reduce their reliance on foreign steel and allowing for local economic growth. By 2016, Hebel hopes to have financing to take the research from the laboratory and start pilot projects where the bamboo composite can be tested in the field (Cardno, 2014).

Chapter 3 – TESTING PROCESS SUMMARY

The goal of this research is to test the application of bamboo as shear, flexural and flexural lap-spliced reinforcement for concrete. The bamboo tested was Moso bamboo (*Phyllostachys edulis*) and was harvested from Fant's Grove in the Clemson Experimental Forest in Clemson, SC. The harvesting and splitting of the bamboo is described in Chapter 4. In order to design bamboo reinforced concrete beams, the properties of the bamboo were investigated including the tensile strength, modulus of elasticity and bond strength with concrete. The details of these component tests and their results can be found in Chapter 5.

Once the mechanical properties of the bamboo were determined, 13 Bamboo Reinforced Concrete (BRC) beams were designed and constructed with varying configurations to test the application of bamboo as shear, flexural, and flexural lap-spliced reinforcement. The design of the BRC beams was done by following the Building Code Requirements for Structural Concrete (ACI 318-11) and replacing the properties of rebar with the predetermined properties of bamboo. Fabrication of the BRC beams included bamboo cage preparation, strain gauge installation, and casting of the beams. The testing setup varied depending on the type of beam being tested. The details of the beam design, fabrication and testing setup can be found in Chapter 6.

The testing took place in the Wind and Structural Engineering Research (WiSER) Facility at Clemson University with a reaction frame incorporating a 150-kip actuator. A total of 13 BRC beams were tested and the strain in the concrete and in the bamboo reinforcement at strategic locations was measured. Each beam was loaded until failure and the failure mechanism of each beam was investigated. After testing, each beam was broken apart with a sledge hammer to expose the bamboo reinforcement to further explain the failure mechanism. The results of the beam tests can be found in Chapter 7.

A summary of the testing process is given below:

1. Investigate the properties of Moso bamboo harvested in Clemson, SC by conducting the following component tests on bamboo specimens:
    a. Tensile strength
    b. Modulus of elasticity
    c. Bond strength
2. Design BRC beams to test the application of bamboo as a reinforcement. Vary beam dimensions and reinforcement configurations to assess the performance of bamboo as shear reinforcement (Table 3.1), flexural reinforcement (Table 3.2) and lap-spliced reinforcement (Table 3.3).

Table 3.1 Shear controlled beam configurations

Shear Beams													
Beam ID	# Flexural Rows	Stirrup Spacing, s		Shear span, a		depth, d		a/d	A_{bamboo}		ρ_{bamboo}	Splice length	
		(in)	(mm)	(in)	(cm)	(in)	(cm)		(in^2)	(cm^2)		(in)	(mm)
S1	4	4.0	101.6	18.0	45.7	16.3	41.4	1.1	3.7	24.0	2.3%	n/a	n/a
S2	4	4.0	101.6	18.0	45.7	15.8	40.2	1.1	3.7	24.0	2.4%	n/a	n/a
S3	4	6.0	152.4	18.0	45.7	15.7	39.9	1.1	3.7	24.0	2.4%	n/a	n/a
S4	4	6.0	152.4	18.0	45.7	15.1	38.4	1.2	3.7	24.0	2.5%	n/a	n/a
S5	4	8.0	203.2	18.0	45.7	15.4	39.1	1.2	3.7	24.0	2.4%	n/a	n/a
S6	4	8.0	203.2	18.0	45.7	16.2	41.0	1.1	3.7	24.0	2.3%	n/a	n/a

Table 3.2 Flexure controlled beam configurations

Flexure Beams														
Beam ID	# Flexural Rows	Stirrup Spacing, s		Shear span, a		depth, d		a/d	A_{bamboo}		ρ_{bamboo}	Splice length	
		(in)	(mm)	(in)	(cm)	(in)	(cm)		(in^2)	(cm^2)		(in)	(mm)
F1	3	6.0	152.4	25.0	63.5	17.0	43.2	1.5	2.8	18.0	1.6%	n/a	n/a
F2	4	6.0	152.4	25.0	63.5	15.8	40.1	1.6	3.7	24.0	2.4%	n/a	n/a
F3	5	6.0	152.4	30.0	76.2	15.5	39.4	1.9	4.7	30.0	3.0%	n/a	n/a
F4	6	6.0	152.4	38.0	96.5	14.5	36.7	2.6	5.6	36.0	3.9%	n/a	n/a

Table 3.3 Lap-splice beam configurations

Lap-spliced Beams													
Beam ID	# Flexural Rows	Stirrup Spacing, s		Shear span, a		depth, d		a/d	A_{bamboo}		ρ_{bamboo}	Splice length	
		(in)	(mm)	(in)	(cm)	(in)	(cm)		(in^2)	(cm^2)		(in)	(mm)
L1	4	6.0*	152.4	25.0	63.5	14.7	37.3	1.7	3.7	24.0	2.5%	12	30
L2	4	6.0*	152.4	25.0	63.5	15.4	39.1	1.6	3.7	24.0	2.4%	18	46
L3	4	6.0*	152.4	25.0	63.5	15.5	39.4	1.6	3.7	24.0	2.4%	24	61

** Note: Lap-splice lengths are surrounded by stirrups with a spacing of 4.0 inches (102 mm)*

3. Construct BRC beams including bamboo cage preparation, strain gauge installation, and casting of the beams.
4. Test BRC beams by loading each beam to failure and measure the strain in the bamboo reinforcement at strategic locations. Investigate the failure mechanism of the beam as well as the bamboo reinforcement.
5. Assess the performance of bamboo as a reinforcement for concrete.
    a. Optimum stirrup spacing
    b. Optimum percentage of flexural reinforcement
    c. Minimum allowable lap-splice length

Chapter 4 – HARVESTING and SPLITTING

4.1 Harvesting

The type of bamboo used in this research was Moso bamboo which is known by the scientific name of *Phyllostachys edulis*. All the bamboo was harvested from the Fant's Grove Moso bamboo site in the Clemson Experimental Forest (CEF) located at 34°37'51" N and 82°49'24" W. A special permit was required to harvest the bamboo and was obtained from the CEF Forest Manager. The bamboo was harvested multiple times from July to September 2013.

Harvesting was done with a hand saw and a machete. The skin of the bamboo is very hard and the machete is ineffective to cut through the entire culm. The machete was used to remove the small branches protruding out from the nodal regions of the culm. When cutting the bamboo it is important to cut just to the outside of a node. This ensures that the final bamboo stalk has nodes very near both of its ends. The nodes, which contain the diaphragms separating the internodal regions, prevent longitudinal cracks from emerging and spreading over the length of the culm. The reinforcement provided by the nodes is important when splitting the bamboo. While harvesting, the longitudinal bamboo was cut to roughly 8 feet (2.5-m) and the stirrup bamboo was cut to approximately 5 feet (1.5-m). Later, the bamboo was cut down to its required length. Figure 4.1 shows the bamboo being harvested from the Clemson Experimental Forest.

Figure 4.1 Harvesting Moso bamboo from the Clemson Experimental Forest

When harvesting bamboo, there are a few important things to look for. First the age of the bamboo is a factor because young bamboo is not as strong as three to four year old bamboo. Young bamboo can be identified by softer skin with chalky, white bands around the culm. During bamboo’s initial growth the culms and branches are protected by sheaths which are usually found near the base of the plant. The size and overall health of the bamboo culm are also important factors affecting the selection of bamboo. Only healthy looking bamboo with no obvious visual defects was harvested. The size of the bamboo harvested was dictated by the dimensions of the splitters. The maximum diameter bamboo which could fit into the largest available splitter was 3.5 inches (90-mm).

4.2 Splitting

Bamboo splits very easily along its fibers which run parallel to its length. There are many methods of splitting bamboo with relatively any sharp blade, but for this research the splitting was done with manufactured bamboo splitters. The splitters were purchased from Hida Tool & Hardware Co.[4] A picture of the two bamboo splitters used can be seen in Figure 4.2. The 6-cut splitter had an inside diameter of 4 inches (102-mm) and the 8-cut splitter had an inside diameter of 4.5 inches (114-mm). The largest diameter bamboo which could be split was 3.5 inches (90-mm) in the 8-cut splitter with inside diameter of 4.5 inches (114-mm). During the splitting process, the bamboo spreads out, so the bamboo must be at least 1 inch (25-mm) smaller than the inside diameter of the splitter to prevent it from getting stuck. Both splitters were used to achieve uniform strip widths of about 1 inch (25-mm) for the longitudinal bamboo and about 0.75 inches (19-mm) for the stirrups. The average cross-sectional area of the longitudinal bamboo was 0.31 square inches (200-mm^2) which is the same cross sectional area as an imperial #5 rebar. The average cross sectional area of the stirrups was 0.20 square inches (129-mm^2) which is the same cross sectional area as an imperial #4 rebar. Splitting should be done within a few days after harvesting to prevent the bamboo from drying out which causes longitudinal cracks to form.

[4] www.hidatool.com

Figure 4.2 Bamboo splitters, 6-cut and 8-cut

To split the bamboo, two people are needed. One holds the bamboo culm steady against an immovable object while the other person hammers the bamboo splitter into the end of the culm to start the splitting. It is necessary to center the splitter on the culm or the result will be strips of varying widths. Also, since the splitters are made from cast iron, it is necessary to not strike the splitter directly with a hammer, but rather to strike a wooden block placed on the splitter. After the initial split has started, the splitter should be held tightly and the bamboo culm should be forcefully pounded into the ground or an immovable object to finish the splitting. The splitting technique can be seen in Figure 4.3.

Figure 4.3 Splitting technique

After the bamboo was split, it was stored in a closed environment until it was used for the component tests and beam reinforcement cages. The bamboo was allowed to dry for at least 2 weeks before being used for the tensile strength tests and at least 1 month before being used for the bond strength tests and the cage fabrication. The moisture content of the bamboo was not measured, but the equilibrium moisture content (EMC) of Moso bamboo is known to be around 10.3% in a 20°C climate of 65% relative humidity (RH) from a report conducted by Larenstein University in the Netherlands (de Vos, 2010).

Chapter 5 – COMPONENT TESTS

The general properties of bamboo have been extensively studied by a number of researchers including Yu, Jiang, Hse and Shupe (2008), Cao and Wu (2008), Ghavami (2005), Khare (2005), Amada (1997), and Kankam and Perry (1989) to name a few. However, the purpose of the component tests were to determine the mechanical properties of the Moso bamboo obtained from the Clemson Experimental Forest which was the same bamboo used to reinforce the concrete beams. In order to design bamboo reinforced concrete beams, the properties of the bamboo were investigated including the tensile strength, modulus of elasticity and bond strength with concrete.

5.1 Tensile strength tests

The tensile strength is a critical factor in choosing a reinforcement for concrete. Typical concrete has a tensile strength of roughly 10-15% of its compressive strength. Since concrete has such low tensile strength, it will crack quickly under tension. In tension zones, the reinforcement is engaged primarily when a crack occurs and, in fact, a crack needs to form for the reinforcement to be fully engaged. Once the crack forms, the stresses are transferred from the concrete to the reinforcement. A traditional reinforcement used is ASTM A615 rebar made of grade 60 steel which has a tensile strength of 60 ksi (414 MPa).

The tensile strength of bamboo is known to be very high, but the average strength varies from species to species. It is also important to note that there is a significant difference between fiber tensile strength and culm tensile strength of bamboo. In this study, the culm tensile strength was tested because 1 inch (25-mm) wide split sections of the culm were used as reinforcement instead of individual fibers.

5.1.1 Specimen preparation

After the bamboo culms were harvested and seasoned they were cut into 15 to 18-inch (380 to 450-mm) long sections. It is important to note that two nodes were included in each specimen so as to give a good representation of the bamboo culm including nodal and internodal regions. Each section was split into 1 in (25-mm) strips with the bamboo splitter as described in Chapter 4. After the initial split, each strip was then split again into 0.5 inch (12-mm) strips with a machete.

Once the general size of the specimen was obtained, a chisel was used to carve away the inner portion of specimen to achieve a “dog-bone” shape as shown in Figure 5.1. After the rough cutting was done with the chisel, a circular sander was used to smooth the faces and achieve as uniform thickness as possible. The specimens were given a “dog-bone” shape to force the failure to occur in between the grips where the cross-sectional dimensions were clearly measured. Grip failure does not give an accurate representation of the specimen tensile strength and should be avoided. The thickness and width were measured at 3 locations on each specimen – left node, middle, right node – and were used to determine the cross-sectional area of the specimen. The initial length between grips was also measured prior to testing. These measurements were used to determine the average stress and strain in each specimen.

Figure 5.1 Tensile sample

Aluminum tabs with a thickness of 0.1 inch (2.5-mm) were epoxied to the ends of the bamboo specimens to prevent the UTM grips from crushing the bamboo. The width and length of the aluminum end tabs varied but were roughly 0.5 inch (12.5-mm) wide and 2-inch (50-mm) long. Fully extended, the grips only had an opening of 0.5 inch (12.5-mm) so the bamboo specimens had to be less than 0.3 inch (7.5-mm) thick to ensure they would fit in the grips with the aluminum tabs epoxied on. The epoxy used was Devcon 5-minute, two-part epoxy with a tensile strength of 1500 psi (10.34 MPa). The end tabs can be seen in Figure 5.2 and Figure 5.3. In total, 30 specimens were tested.

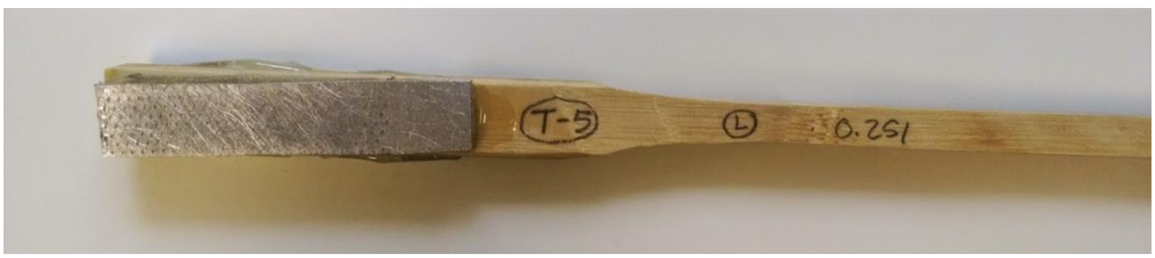

Figure 5.2 Epoxied end tab, front view

Figure 5.3 Epoxied end tab, side view

5.1.2 Test setup

The specimens were tested in a 10-kip (50 kN) Tinius Olsen UTM, as shown in Figure 5.4, with V-grips. The V-grips are designed to apply more pressure as the specimen is pulled in tension. An enlarged picture of a grip can be seen in Figure 5.5.

The specimens were loaded continuously at a rate of 0.05 in/min (0.02-mm/s) until failure. The resulting force-displacement data was recorded through a MATLAB script written by Dr. Weichiang Pang.

Figure 5.4 Tinius Olsen 10-kip UTM

Figure 5.5 Close-up of a V-grip

5.1.3 Results

All tensile specimens failed in a brittle manner at a node. This was expected based on reviewing previous literature. Amada (1997) has described the fiber distribution in the nodes to be chaotic, which is an explanation for their brittle behavior. The failure mechanisms of the tensile samples included clean break through node, diagonal split from node to node, and diagonal split from node to tab. The failure modes are shown in Figure 5.6.

Figure 5.6 Tensile specimen failure modes

The results of the tensile tests are summarized in Table 5.1 and the stress-strain curves for the tensile specimens are shown in Figure 5.7. The probability density functions (PDFs) and cumulative density functions (CDFs) for both stress and strain can be seen in Figure 5.8 through Figure 5.11. The comprehensive tensile test data can be found in Appendix A.

Table 5.1 Summary of Tensile Tests

	Modulus of Elasticity		Tensile Strength		Rupture Strain	
	(ksi)	(MPa)	(ksi)	(MPa)	(in/in)	%
Mean	1145	7891	18.7	129.1	0.016	1.64
Standard Deviation	223	1536	2.67	18.4	0.004	0.38
Coefficient of Variation	0.19		0.14		0.230	
Minimum	688	4742	11.3	78.1	0.007	0.74
Maximum	1577	10872	23.0	158.7	0.027	2.70

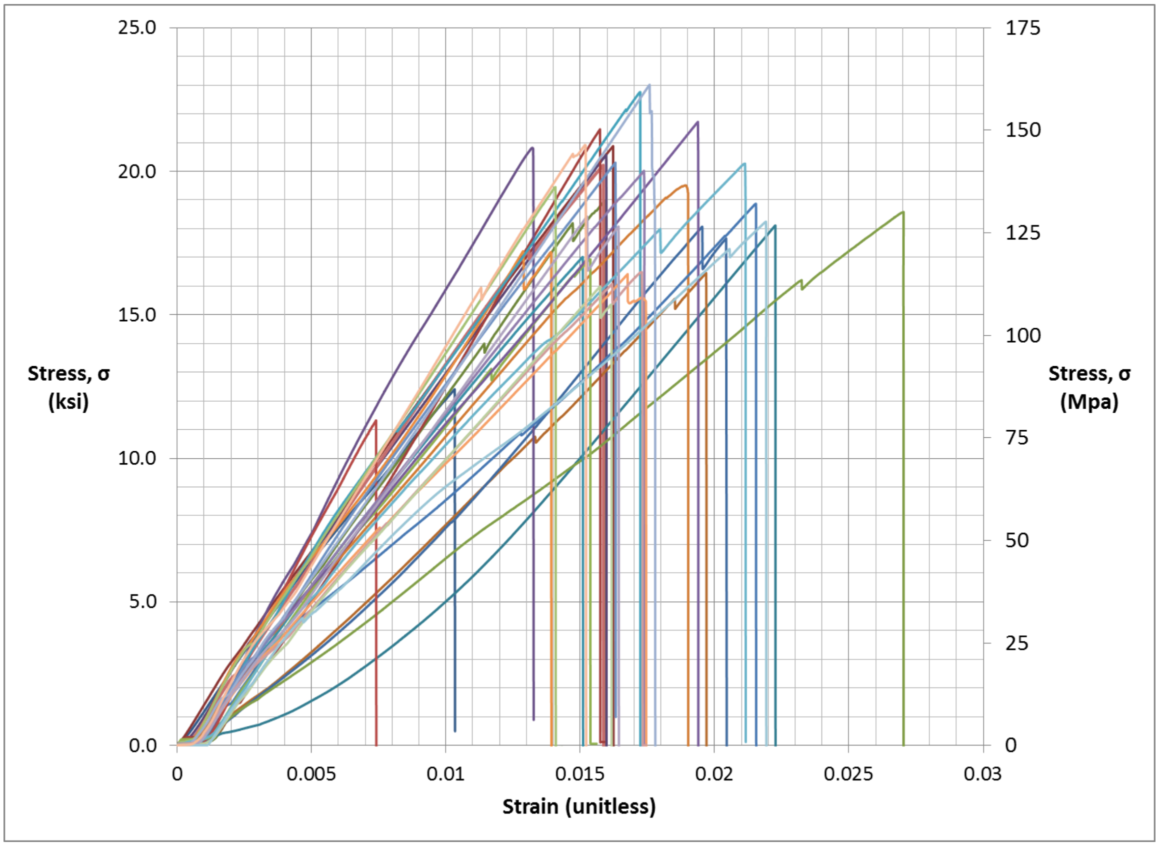

Figure 5.7 Stress strain curves for bamboo tensile samples

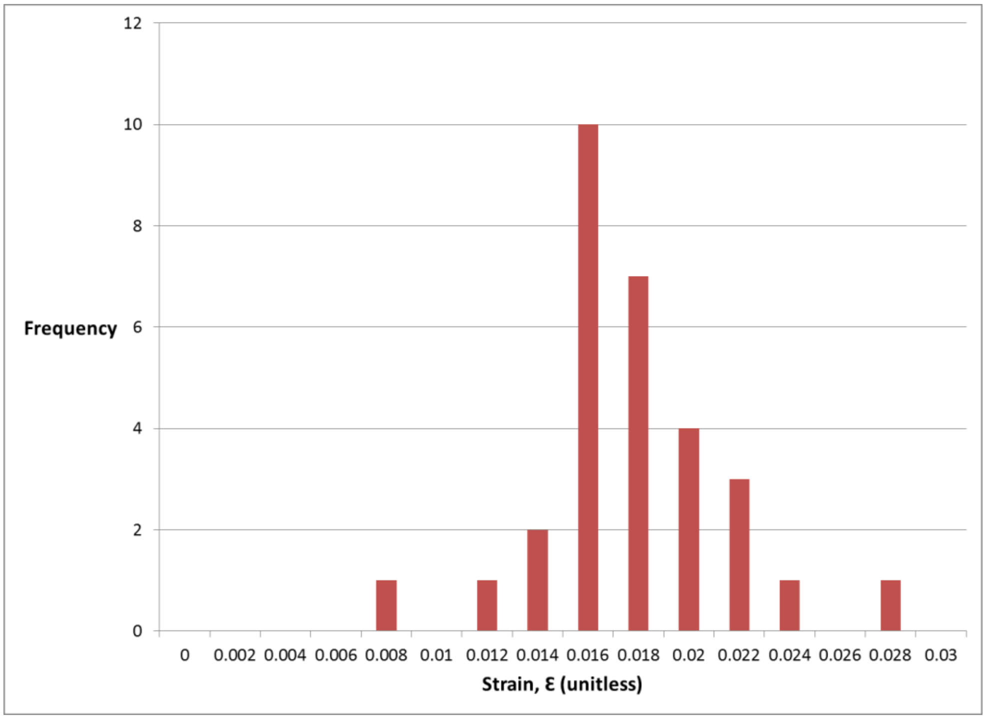

Figure 5.8 PDF of rupture strains

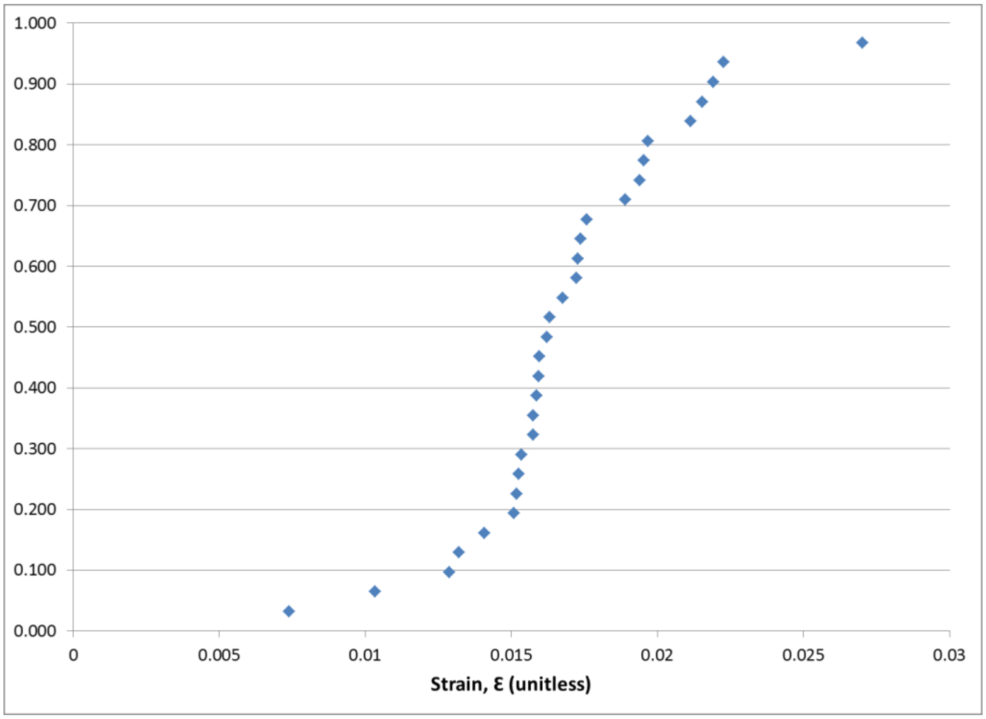

Figure 5.9 CDF of rupture strains

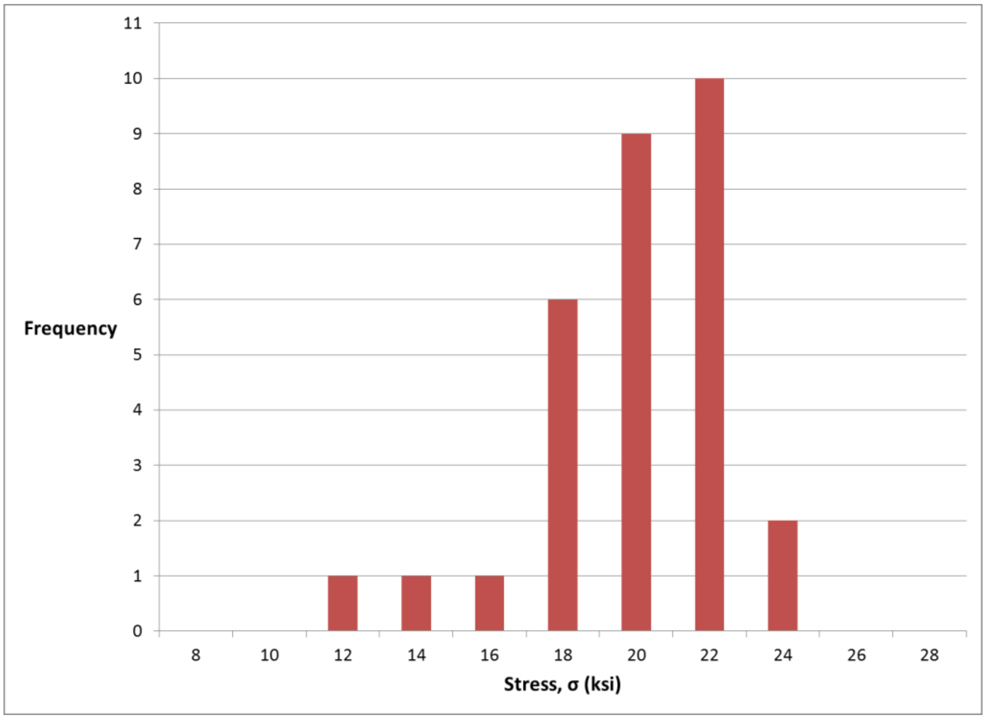

Figure 5.10 PDF of failure stresses

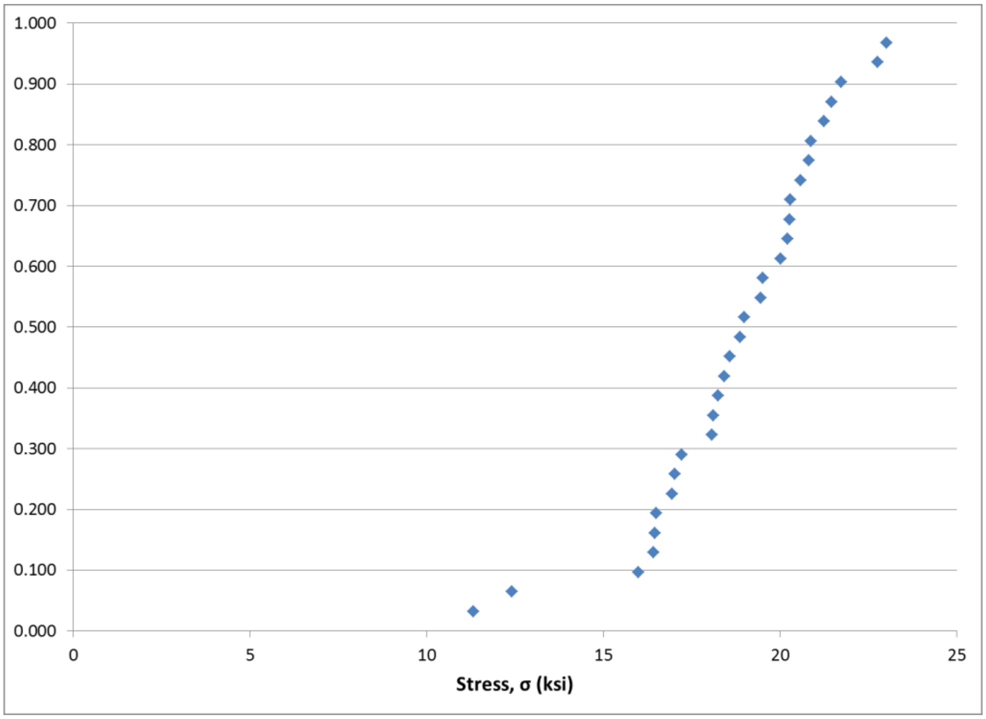

Figure 5.11 CDF of failure stresses

Upon failure, the maximum load was divided by the area of the corresponding failure location to obtain the maximum stress in the bamboo. The calculation for stress is given in Equation 5.1 where F is the force exerted and A is the cross-sectional area. The tensile strengths of the bamboo specimens ranged from 11.3 – 23.0 ksi (78 – 159 MPa), with a mean of 18.7 ksi (129 MPa) and a standard deviation of 2.67 ksi (18 MPa). These tests indicated that the tensile strength of the Moso bamboo was about one-third that of a typical grade 60 steel rebar.

$$\sigma = \frac{F}{A} \qquad 5.1$$

The rupture strain for each specimen was calculated by dividing the maximum displacement of the grips at failure by the length of the bamboo specimen. The calculation for strain can be found in Equation 5.2 where ΔL is the change in length of the specimen and L_0 is the initial length. The rupture strain of each specimen ranged from 0.007 to 0.027 with a mean of 0.016 and a standard deviation of 0.004.

$$\varepsilon = \frac{\Delta L}{L_0} \qquad 5.2$$

A comparison between the stress-strain curves of bamboo and low carbon steel rebar can be seen in Figure 5.12. It is important to notice the ductile nature of the steel compared to the brittle nature of the bamboo.

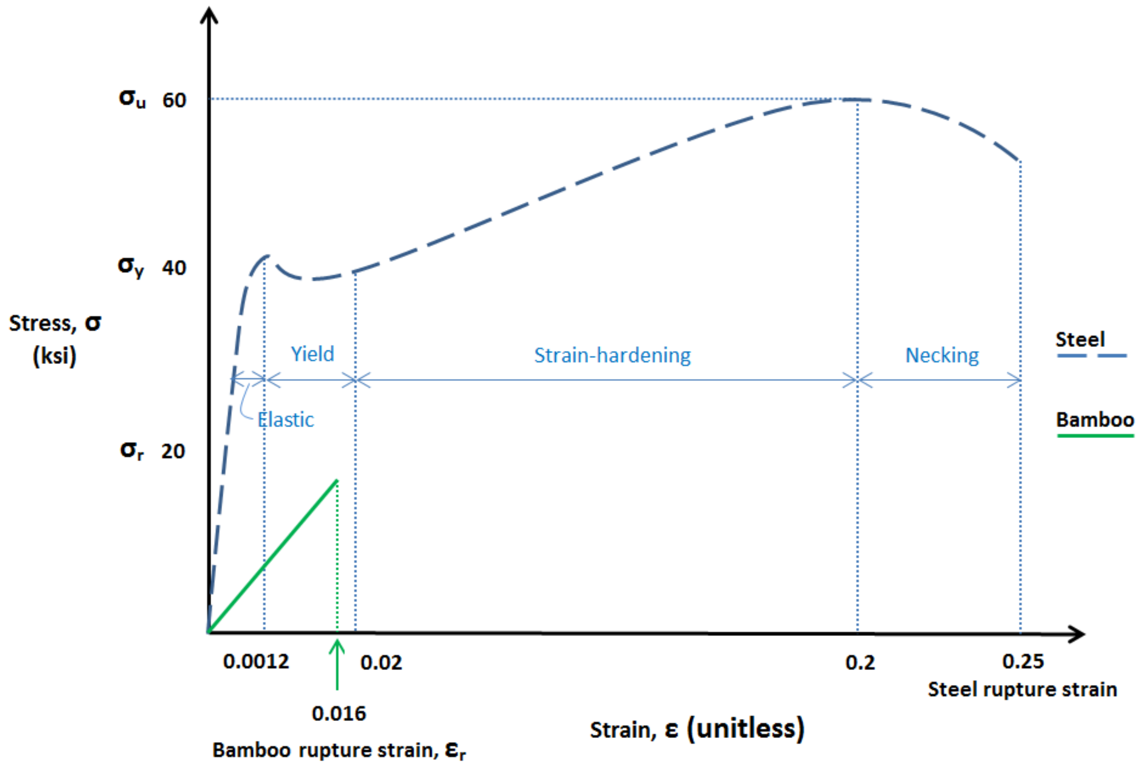

Figure 5.12 Comparison of stress-strain curves for low carbon steel and bamboo

Steel has four regions of response to increased strain. The linear portion of the steel's stress-strain curve is known as the elastic region. In this region the ratio of stress to strain (modulus of elasticity) is constant. After the elastic limit is reached, low carbon steel will yield until reaching a strain of 0.02. In between a strain of 0.02 and 0.2, the steel will gain strength through a process known as strain-hardening. After a strain of 0.2, the steel will begin necking until it ruptures at a strain of 0.25.

As evident from the stress strain curves in Figure 5.7, bamboo is not a ductile material, but rather exhibits brittle failure. The rupture failure mode of the bamboo is problematic for design. For traditional steel reinforced concrete design using ACI 318-11, the steel is designed to reach the yielding phase and carry a constant stress, known as its yield stress. Section 10.3.4 of ACI 318-11 states, "Sections are tension-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or greater than 0.005 when the concrete in compression reaches its assumed strain limit of 0.003." Tension-controlled failure is preferred as there is ample warning of failure with excessive deflection and cracking.

Unlike steel, bamboo does not yield upon reaching its critical strain. Rather, bamboo will fail in rupture upon reaching a strain of about 0.016. This rupture failure mode presents numerous problems when using ACI 318-11 to design bamboo reinforced concrete elements. In steel reinforced concrete design there is no upper limit to the strain in the steel reinforcement, because after the steel yields it will continue to gain strength through strain hardening and the yield stress of the steel is used for design. In bamboo reinforced concrete design there must be an upper limit to the strain in the bamboo reinforcement to avoid rupture failure. At the same time there must be a lower limit to the strain in the bamboo so as to ensure that the bamboo is carrying sufficient stress.

It is the author's thought that although single strips of bamboo exhibit brittle failure, a bundle made up of multiple strips of bamboo may exhibit ductile failure. Since each bamboo strip will have varying ultimate stresses and rupture strains, as the strain in the bamboo bundle increases the strips will fail at different levels of strain. When one strip fails, the stresses will be transferred to the remaining strips. This will continue until all the strips have failed which will correspond to the failure of the bundle. This theory is shown in Figure 5.13, where the average stress for all the tensile samples was taken at each strain level. When a bamboo sample reached its rupture strain its stress value was taken as zero and still calculated into the average stress. The effects of bamboo's brittle failure nature will be further investigated in the beam tests.

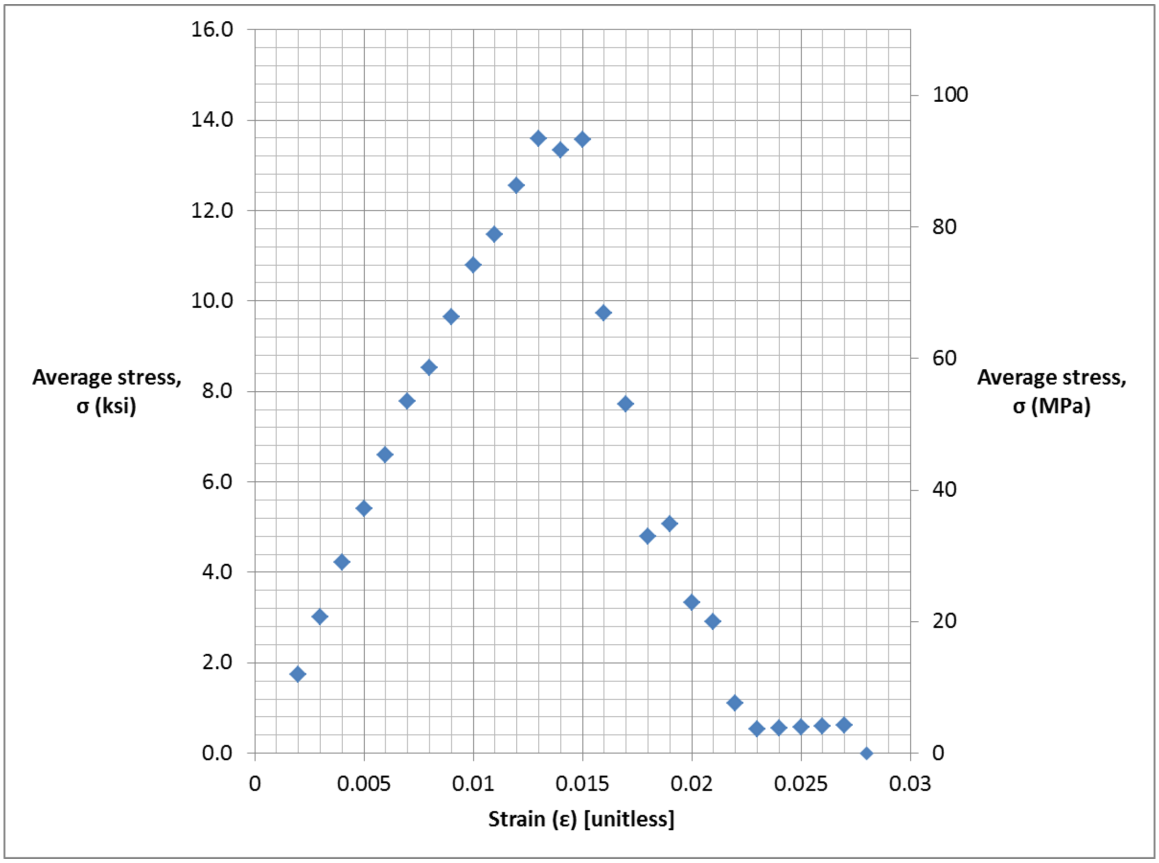

Figure 5.13 Average stress of bamboo specimens under strain

The variability of the tensile strength and rupture strain of bamboo is evident from the stress-strain curves in Figure 5.7 and the relatively high coefficients of variation shown in Table 5.1. The variability in bamboo specimens is expected because bamboo is a natural material and many factors affect its mechanical properties. However, the variability presents a challenge in design and appropriate safety factors will have to be determined when designing bamboo reinforced concrete members.

The correlation between the Moso bamboo's mechanical properties, including stress, strain and modulus of elasticity, was also investigated. From Figure 5.14, a moderately positive correlation can be observed between stress and strain indicating that specimens with higher rupture strains generally had higher stresses. The coefficient of correlation between stress and strain is 0.38. A weak positive correlation exits between the modulus of elasticity and stress with a coefficient of correlation value of 0.18 and is shown in Figure 5.15. There is a strong negative coefficient of correlation between the modulus of elasticity and strain equaling -0.81 and shown in Figure 5.16. This indicates that bamboo with a high rupture strain generally has lower stiffness. A summary of the coefficients of correlation and the covariance between each property is provided in Table 5.2.

Table 5.2 Correlation between bamboo properties

	Stress vs. Strain	MoE vs. Stress	MoE vs. Strain
Coefficient of Correlation	0.38	0.18	-0.81
Covariance	0.004	106.060	-0.657

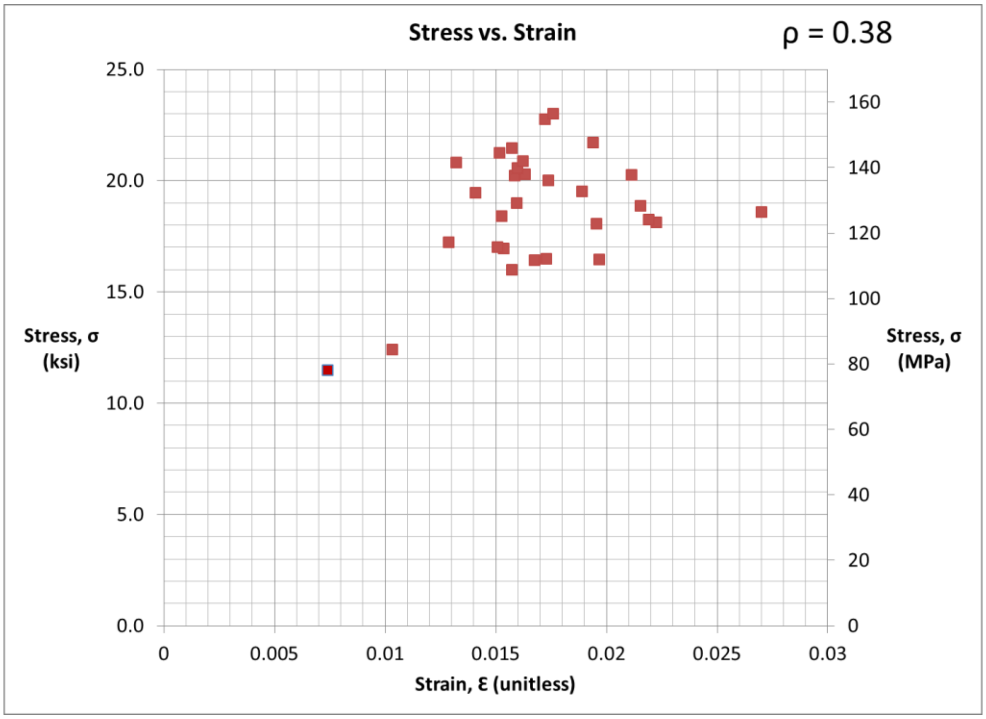

Figure 5.14 Correlation between stress and strain

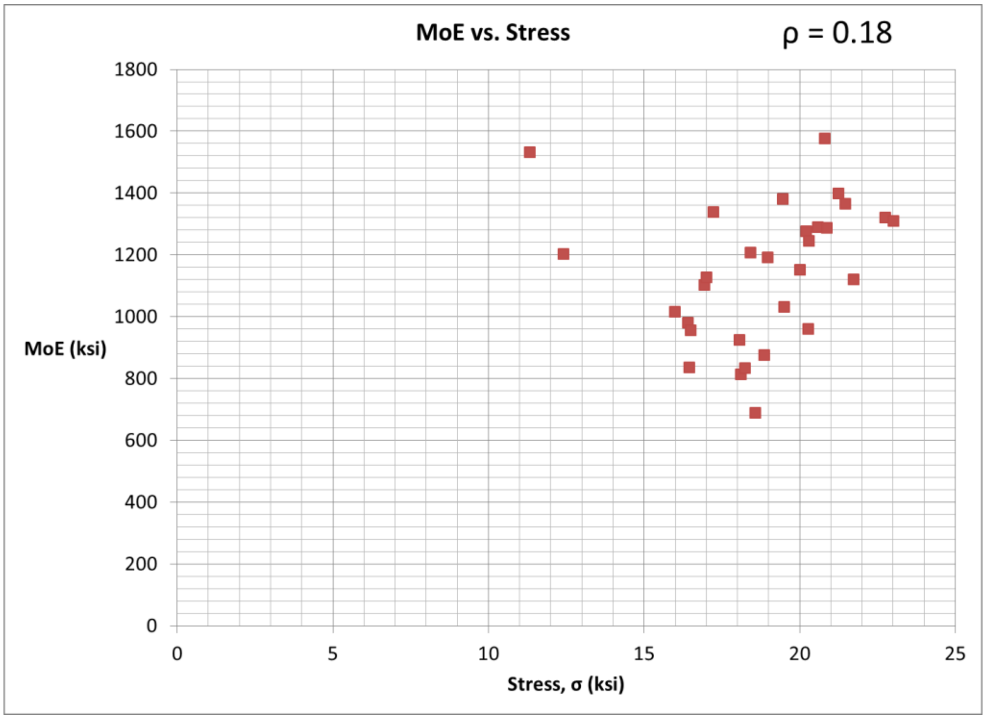

Figure 5.15 Correlation between modulus of elasticity and stress

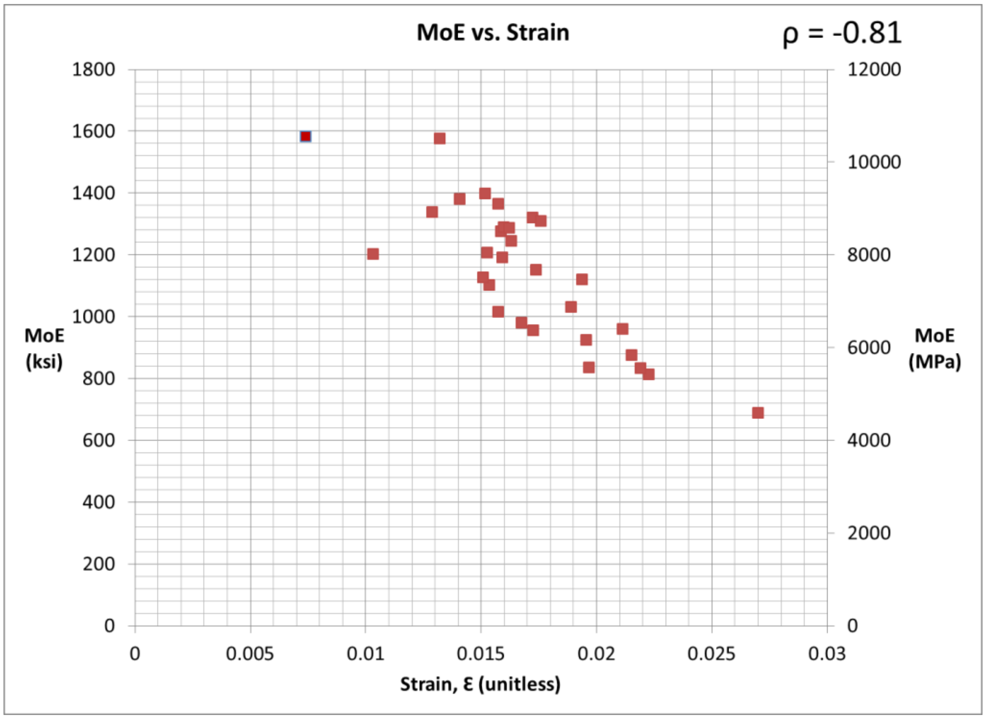

Figure 5.16 Correlation between modulus of elasticity and strain

The modulus of elasticity of the Moso bamboo was determined through the tensile test results. The modulus of elasticity is a measure of the stiffness of a material and is calculated by dividing the stress by the strain as shown in Equation 5.3.

$$E = \frac{\sigma}{\varepsilon} \quad 5.3$$

The modulus of elasticity of concrete can be approximated using Equation 5.4 from ACI 318-11 Section 8.5.1, assuming a normal weight concrete. The modulus of elasticity for a 3,000 psi (21 MPa) concrete would be approximately 3,100 ksi (21,000 MPa).

$$E = 57{,}000\sqrt{f'_c} \quad 5.4$$

The calculated modulus of elasticity (MoE) averaged 1,145 ksi (7,894 MPa) which is less than the value of 2,500 ksi (17,237 MPa) suggested by Janssen (2000) and Brink and Rush (1966, 2000). The MoE of bamboo in this study ranged from 688 – 1,577 ksi (4,744 – 10,873 MPa) with a standard deviation of 223 ksi (1,538 MPa). The results for the bamboo's MoE values can be found in Table 5.1 in Section 5.1.3. In Figure 5.7 the stiffness of each bamboo tensile specimen can be compared relative to each other. Steeper stress-strain curves indicate a higher MoE and therefore a higher stiffness.

The modulus of elasticity of the Moso bamboo tested is much lower than steel and even lower than concrete. Bamboo's low MoE is expected to have negative impacts on its performance as a reinforcement and ability to decrease crack width in the concrete, but this will be further examined during the beam tests.

5.2 Bond strength tests

A good bond between the reinforcement and the concrete is necessary to ensure an effective transfer of tensile stresses from the concrete to the reinforcement. There have been multiple studies on the bond strength between concrete and bamboo. The bond strength results tested by others as well as the test results by the author of this study are presented and briefly discussed next. The bond strength tests were not meant to be a comprehensive study, but rather to give an idea of how the bamboo would bond with the concrete.

The bond strength tests consisted of two phases. In the first phase, bond strength tests were conducted on specimens with varying embedment lengths and the results were used to determine the appropriate lap-splice lengths to examine during the full-scale beam tests. Very little literature is available on lap-splicing bamboo. Glenn (1950) suggests to lap splice bamboo a minimum distance of 25 inches (635-mm) and to securely tie the splices. Based on the results of the pullout tests, lap-splice lengths of 12 inches (305-mm), 18 inches (457-mm) and 24 inches (610-mm) were chosen for the lap-spliced beam tests.

In the second phase, a new set of bond strength specimens were cast using the same concrete mix as used for the large-scale beam tests. The bamboo in these bond strength samples was designed to be comparable to the longitudinal bamboo reinforcement used in the beams – being waterproofed and having the same dimensions. Since waterproofing was already known to improve bond strength, the effect of waterproofing was not studied in depth. The purpose of the second phase of bond strength tests was to quantify the bond strength of the waterproofed bamboo cast within the beams.

5.2.1 Specimen preparation

The pullout specimens were similar to the tensile specimens described in Section 5.1.1. However, the bond strength bamboo specimens differed from the tensile specimens in that only one end had a "dog-bone shape." The other end was straight and was embedded into a 4-inch (102-mm) diameter concrete cylinder. Four different embedment lengths were tested and can be seen in Figure 5.17. The embedment depths included 9 inches (229-mm), 12 inches (305-mm), 15 inches (381-mm), and 18 inches (457-mm).

The embedded length of the bamboo specimens were not sanded down or modified in any way to ensure that they were similar to the bamboo strips used for the beam design. The bamboo specimens for phase two testing were waterproofed with Thompson's WaterSeal, model number TH.024101-16 which was also used to waterproof the bamboo reinforcement in the beams. The bamboo pullout specimens were all roughly 1 inch (25.4-mm) wide and 0.3 inch (7.6-mm) thick. The dimensions of the embedded bamboo specimens were chosen based on the dimensions of the longitudinal bamboo strips used in the beam design since the purpose of the pullout specimens was to determine how the longitudinal bamboo would bond to the concrete.

Similarly to the tensile specimens, the grip ends of the bamboo specimens had to be sanded level to remove the natural curvature of the bamboo so the aluminum tabs could be epoxied to the ends and a tight hold could be achieved in the grips. The grip end of the pullout specimen without the aluminum tabs can be seen in Figure 5.18.

Figure 5.17 Pullout specimen embedment depths

Figure 5.18 Pullout specimen grip end

The pullout specimens proved difficult to test. The reduced cross-sectional area of the bamboo at the grip, combined with the concentrated pressure of the grips cutting into the bamboo, facilitated failure at the grip end of the specimens. The reduced cross-sectional area of the pullout specimen can be seen in Figure 5.18.

Aluminum is a very soft metal, with a hardness number of 2.5 – 3.0 on the Mohs hardness scale. The softness of the aluminum tabs did not distribute the pressure of the UTM grips along the length of the tab, and the pressure of the grips cutting into the bamboo caused premature failure at the grips. The aluminum tabs which were used for the tensile test specimens did not cause grip failure because the tabs were thicker than the tabs used for the bond strength specimens. Also, the middle portion of the tensile specimens had a small cross-sectional area and the ultimate stress was achieved at this location, preventing grip failure. A solution to the grip failure problem is to use a harder metal for the end tabs and use thicker tabs. Steel would be a good option since it has a hardness value of between 4.0 – 4.5 on Mohs hardness scale, but steel tabs were not used since all the samples had already been prepared with the aluminum tabs at the time of testing.

After the bamboo specimens were prepared, they were placed in the PVC pipe which was then filled with either QUIKRETE® Concrete Mix (No. 1101) or the concrete mixture used for the beam specimens. The QUIKRETE® Concrete Mix specimens were mixed in a portable concrete mixer as shown in Figure 5.19. About 9 pt. (4.3 L) of water was added for each 80 lb. bag of concrete as per the mixing instructions. Each specimen was filled in 4-inch (102-mm) layers – with each layer being compacted – before the next layer was added as per ASTM C31. Each specimen was air cured with a damp cloth covering the concrete during the initial curing phase. The phase two pullout specimens can be seen in Figure 5.20. A total of 23 bond strength samples were prepared and tested.

Figure 5.19 Portable concrete mixer

Figure 5.20 Phase two pullout specimens

5.2.2 Test setup

Similarly to the tensile strength tests, the bond strength tests were conducted in the 10-kip Tinius Olsen UTM. One V-grip was used to hold the bamboo end with the aluminum end tabs and the cylinder end of the specimen was held in a steel frame. The test setup can be seen in Figure 5.21 and a pullout test specimen can be seen in Figure 5.22.

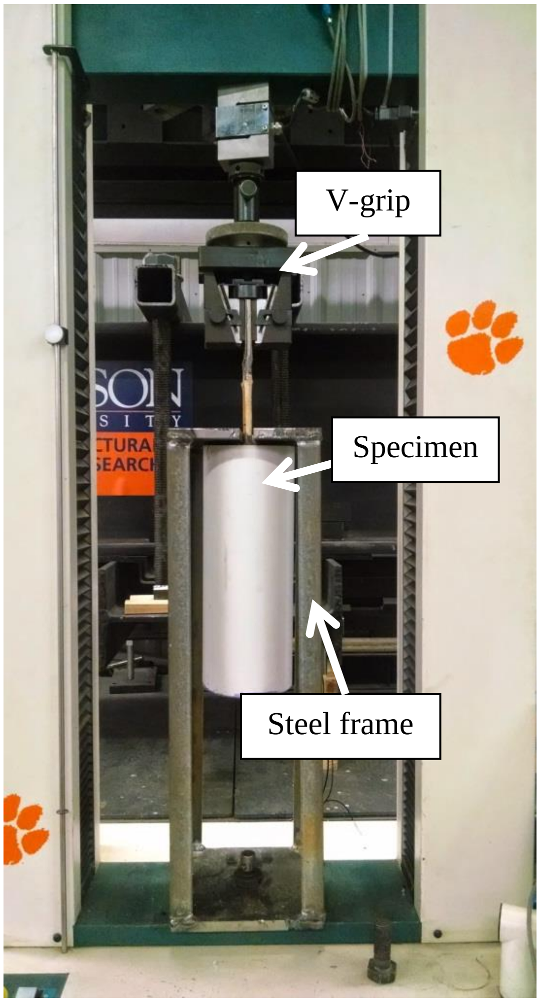

Figure 5.21 Pullout test setup

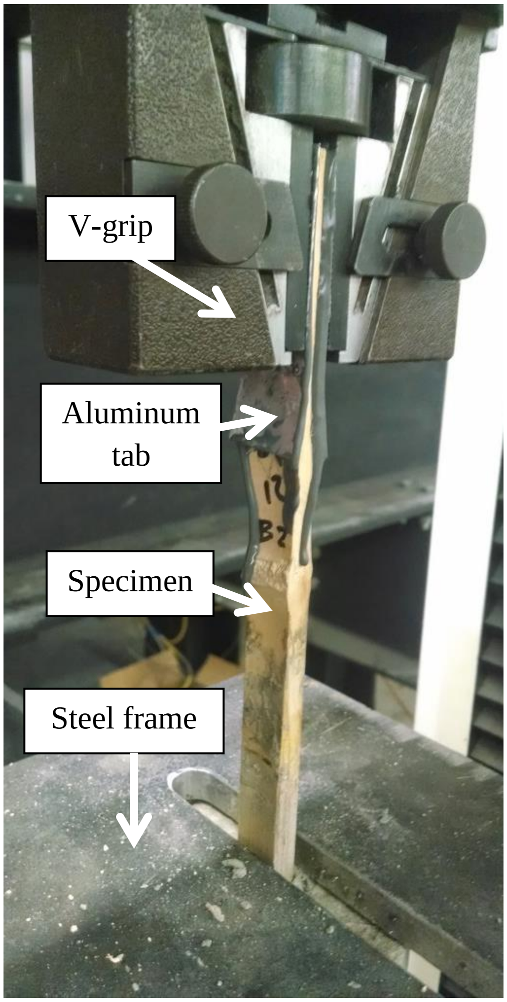

Figure 5.22 Pullout test specimen

The bond strength tests consisted of two phases. QUIKRETE® Concrete Mix was used in phase one to determine the range of embedment depths to test, since it cures to a high strength within a short period of 7 days. The QUIKRETE® Concrete Mix (No. 1101) used in the tests has a typical 7-day compressive strength of 2500 psi (17.2 MPa) according to ASTM C39. Once the range of embedment depths was determined with the QUIKRETE® Concrete Mix specimens, the rest of the phase two pullout specimens were cast using the same concrete mix as used for the full-scale beam specimens.

5.2.3 Results

In this study, four different embedment depths have been tested: 9 inches (229-mm), 12 inches (305-mm), 15 inches (381-mm), and 18 inches (457-mm) for bamboo specimens around 1 inch (25.4-mm) wide and 0.3 inch (8-mm) thick. The testing of the 9-inch (229-mm) sample resulted in pullout failure with a bond stress of 106 psi (0.73 MPa) indicating that the 9-inch (229-mm) embedment length is inadequate to develop sufficient bond between the bamboo and concrete even when a node and diaphragm are present.

Diaphragms increased the bonding strength between the bamboo and the concrete. The average bond stress for the pullout specimens can be seen in Table 5.3. The average bond stress for specimens with zero diaphragms was 72.0 psi (0.50 MPa) and the average bond stress for specimens with 1 diaphragm was 82.6 psi (0.57 MPa). Diaphragms increased the bond strength by 15%.

Table 5.3 Effect of diaphragms on bond stress

# of diaphragms	Bond stress	
	(psi)	(MPa)
zero	72.0	0.50
1	82.6	0.57

Results from the phase one tests showed pullout failures occurred for the 9-inch (229-mm), 12-inch (305-mm) embedment depths. Based on these results as well as the recommendation by Glenn (1950) to splice bamboo 25 inches (635-mm), lap-splice lengths of 12 inches (305-mm), 18 inches (457-mm) and 24 inches (610-mm) were chosen for the lap-spliced beam tests.

The purpose of the phase two bond strength tests was not to determine the effects of waterproofing and long term curing, but rather to quantify the bond strength of the waterproofed bamboo cast within the beams. The average bond stress of the waterproofed specimens after curing a period of 120 days was 76.4 psi (0.53 MPa).

Summary results from the phase one and phase two tests were compiled and the number of pullouts and percent pullout at each bonded length can be seen in Table 5.4 and the resulting graph can be seen in Figure 5.23. A second order polynomial trend line was fit to the data with Equation 5.5.

$$\%\, Pullout = 0.7496(d)^2 - 30.767(d) + 313.57 \qquad 5.5$$

Solving for the embedment depth, *d*, at which *% Pullout* equals zero results in an embedment depth of 18.8 inches (478-mm). From this analysis it can be concluded that a development length of about 20 inches (508-mm) is sufficient for waterproofed Moso bamboo strips 1 inch (25-mm) wide and 0.3 inch (8-mm) thick.

Table 5.4 *% Pullout* at different embedment depths

Embedment depth (in)	Total samples	# Pullouts	% Pullout
9	1	1	100
12	9	4	44
15	7	2	29
18	6	0	0

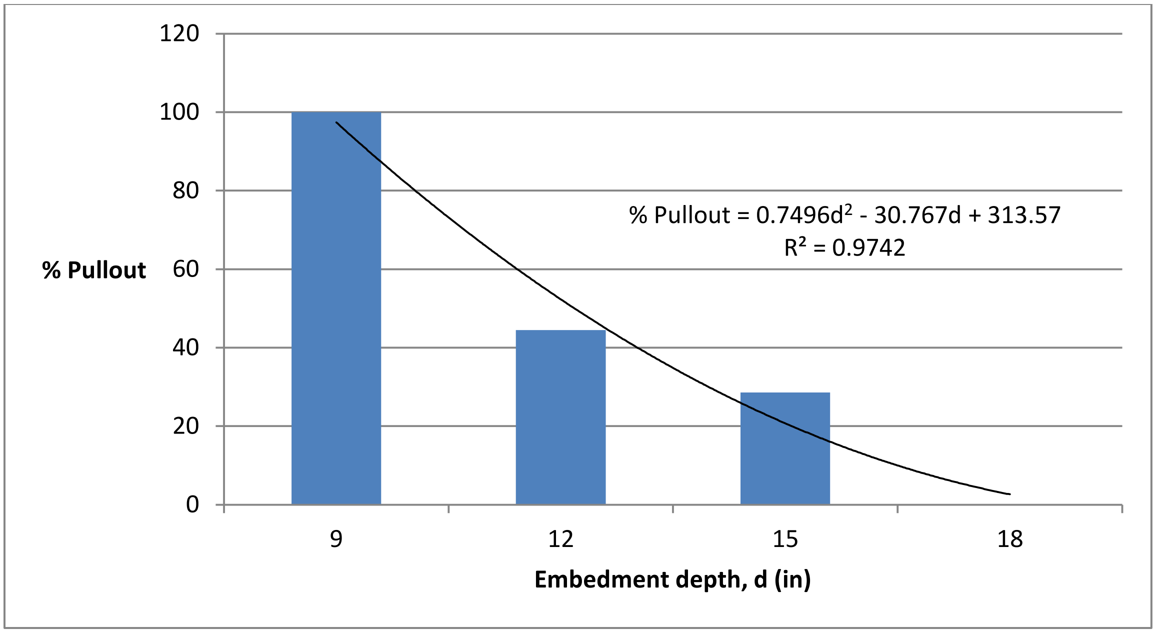

Figure 5.23 *% Pullout* at different embedment depths

Another analysis using detailed calculations for required embedment depth (development length) are found below. In these calculations, the results from the tensile strength tests are used in addition to the pullout tests to determine the range of development lengths required based on the distribution of bamboo tensile strengths. First, the ultimate force, *F*, is calculated based on the range of ultimate bamboo strengths (min, mean, and max), f_{ult}, and the average cross-sectional area, *A*, of the pullout specimens as shown in Equation 5.6.

$$F = f_{ult} \times A = \begin{pmatrix} 2.9 \\ 4.7 \\ 5.8 \end{pmatrix} kip \qquad 5.6$$

$$f_{ult} = \begin{pmatrix} 11.3 \\ 18.7 \\ 23.0 \end{pmatrix} ksi$$

$$A = 0.252 in^2$$

The bond stress is calculated by dividing the force required to achieve pullout failure by the contact surface area of the bamboo embedded in the concrete. The calculation for pullout bond stress can be seen in Equation 5.7, where *F* is the force required to achieve pullout failure, *P* is the perimeter around the bamboo specimen and *d* is the embedment depth. The average pullout bond stress determined from the bond strength tests is 78.1 psi (0.54 MPa).

$$f_{bond} = 78.1psi = \frac{F}{P \times d} \qquad 5.7$$

Re-arranging Equation 5.7 to solve for the embedment depth, *d*, and substituting the ultimate force, *F*, from Equation 5.6 and the average perimeter, *P*, of 2.377 inches (60.4-mm) results in Equation 5.8.

$$d = \frac{F}{P \times f_{bond}} = \begin{pmatrix} 15.3 \\ 25.4 \\ 31.2 \end{pmatrix} in \qquad 5.8$$

Therefore, the range of embedment depths for the previously specified bamboo dimensions is 15 – 31 inches (381 – 787-mm) based on the range of ultimate bamboo strengths and the average area, perimeter and pullout bond stress of the bond strength specimens. The average embedment depth required is 25 inches (635-mm). These results compare with the minimum required embedment depth, *d*, calculated from Equation 5.5 which is around 20 inches (508-mm).

Multiple bamboo pullout specimens failed at the grip after reaching tensile stresses ranging from 5.0 – 13.6 ksi (34.5 – 93.8 MPa). These stresses were less than the expected tensile strength of the bamboo, indicating a higher bond strength could be reached and making these test results inconclusive. Figure 5.24 shows the grip failure for specimen BW 14, while Figure 5.25 shows pullout failure for specimen BW 15.

Figure 5.26 shows the tested bond strength samples. The comprehensive data of the bond strength tests can be found in Appendix B.

Figure 5.24 Bond specimen grip failure

Figure 5.25 Bond specimen pullout failure

Figure 5.26 Tested bond strength samples

Chapter 6 – BEAM DESIGN, FABRICATION and TEST SETUP

6.1 Beam design

Once the mechanical properties of the bamboo were determined, 13 BRC beams were designed and constructed with varying configurations to test the application of bamboo as shear, flexural, and flexural lap-spliced reinforcement. The design of the BRC beams was done by following the Building Code Requirements for Structural Concrete (ACI 318-11) and replacing the properties of rebar with the properties of bamboo as determined from the component tests in Chapter 5.

A modification was made to the bamboo model since bamboo exhibits brittle characteristics and does not behave like steel. Instead of achieving the yield strength as steel would upon reaching its yielding strain, bamboo ruptures when it reaches its rupture strain of 0.016. Since the failure mechanism of the bamboo reinforcement when acting as a group is unknown, three different models were used to predict the capacity of each beam.

The first model assumes bamboo will yield like steel and retain all of its maximum stress upon reaching its strain limit. In the MathCAD calculations this model is labeled as *Yielding*. The second model assumes bamboo will only retain 2/3 of its maximum stress upon reaching its strain limit. This model is labeled as *Rupture 2/*3. The third model accounts for the loss of bond between the bamboo and the concrete by reducing modulus of elasticity of the flexural bamboo reinforcement. This model was first presented by Shimoda, Murakami, Takeda, Matsunaga and Kakuno (2010) and then confirmed by Yamaguchi et al. (2013). This model is labeled as *Yielding 0.6E.* The stress-strain diagrams for these models are shown in Figure 6.1 and compared to the results from the component tensile tests (average stress of combined bamboo). The three models were used to predict the capacities of each beam and the results were compared to the actual beam capacities after testing.

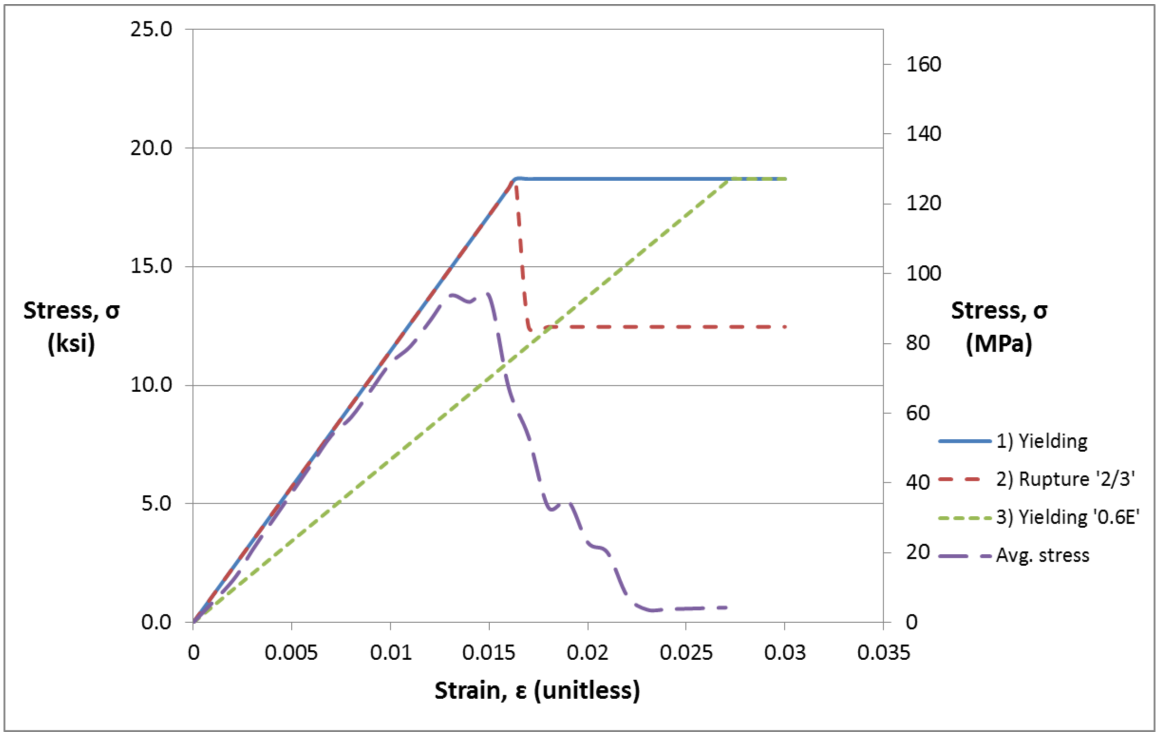

Figure 6.1 Stress-strain curves of three bamboo models

The beams were designed not based off of LRFD loads but rather from previous research and the component tests which gave target longitudinal reinforcement ratios and suggested stirrup spacing. The beam reinforcement configuration, including location, quantity and spacing was selected. Then the expected flexural and shear capacities for each beam were determined through multiple iterations, varying the depth of the neutral axis, *c*, which would result in static equilibrium between the compressive and tensile forces acting on the concrete beam cross section. The compression force from the equivalent rectangular stress block was calculated using Equation 6.1 where f'_c is the concrete design compressive strength, a is the depth of the compression block and b is the beam width.

$$C = 0.85f'_c(a)(b) \qquad 6.1$$

The tension force in the bamboo reinforcement, T, was calculated using Equation 6.2. A_{bi} is the area of bamboo in each layer and $Stress_i$ is the stress in each bamboo layer.

$$T = \sum_{i=1}^{N_{rows}}(Stress_i)(A_{b_i}) \qquad 6.2$$

The force in each bamboo layer, $Stress_i$, was determined based on the Bernoulli-Euler principle which state that the strains above and below the neutral axis are proportional to the distance from the neutral axis. The Bernoulli-Euler principle is shown visually in Figure 6.2. The strain in each bamboo layer is given as ε_{bi} and the depth to each layer is given as d_i. The depth to neutral axis is given as c. The equivalent triangles are based off the rupture strain limit of concrete, which is set at 0.003.

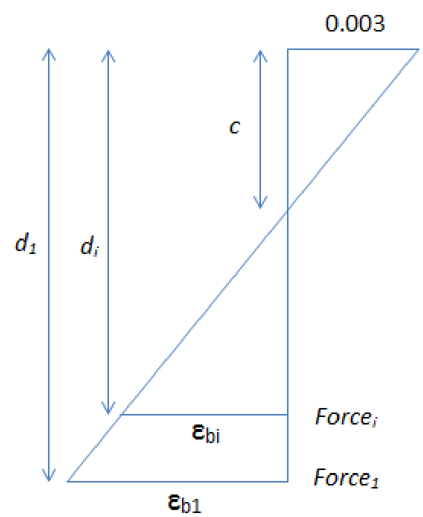

Figure 6.2 Bernoulli-Euler principle

Once the depth to the neutral axis resulting in static equilibrium between the compressive and tensile forces acting on the cross-section was determined, the nominal moment design M_n was calculated based off Equation 6.3. The depth of the equivalent compressive block a is calculated from Equation 6.4.

$$M_n = \sum_{i=1}^{N_{rows}} \left[(Force_i) \left(d - \frac{a}{2} \right)_i A_{b_i} \right] \qquad 6.3$$

$$a = \beta_1 c \qquad 6.4$$

The nominal shear capacity of the beams, V_n, were calculated using Equation 6.5. V_c is the shear capacity of the concrete and is given by Equation 6.6 where b is the width of the beam. For all concrete flexural members, the maximum shear applied V_u must be less than or equal to half the shear capacity of the concrete, V_c.

$$V_n = V_c + V_s \qquad 6.5$$

$$V_c = 2\sqrt{f'_c}(b)(d) \qquad 6.6$$

The shear capacity of the reinforcement, V_s, is given in Equation 6.7, where A_v is the area of the shear stirrups, and s is the stirrup spacing. Both f_y and d were previously defined.

$$V_s = \frac{A_v f_y d}{s} \qquad 6.7$$

After the nominal flexural and shear capacities of the beam was calculated, the minimum load, P, which would cause either flexural or shear failure was determined. Using P, the maximum applied moment M_a and max applied shear V_a were determined.

Each of the beams tested is technically classified as a deep beam according to ACI-318 Section 11.7.1. The provisions of Section 11.7 for deep beams are applied to members with the clear span-to-depth ratio not exceeding 4 (l_n/h<4) and when the shear span-to-depth ratio does not exceed 2 (a_v/h<2). Figure 6.3 provides a visual for the shear span limit. These dimensioning conditions allows for compression struts to be developed between the loads and supports as shown in Figure 6.4.

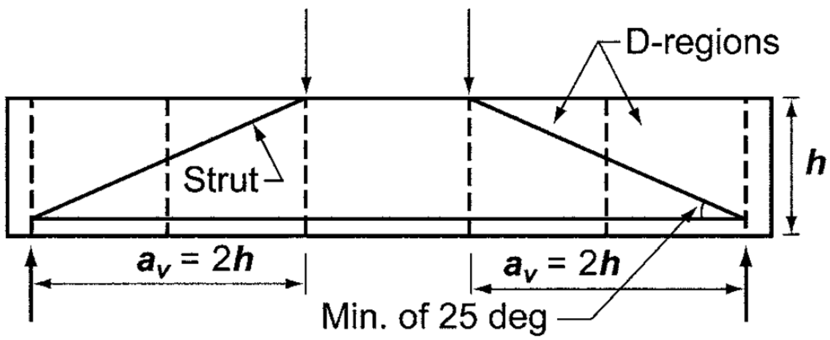

Figure 6.3 Shear span, a_v = 2h, limit for a deep beam
(ACI 318-11 Fig. RA.1.2)

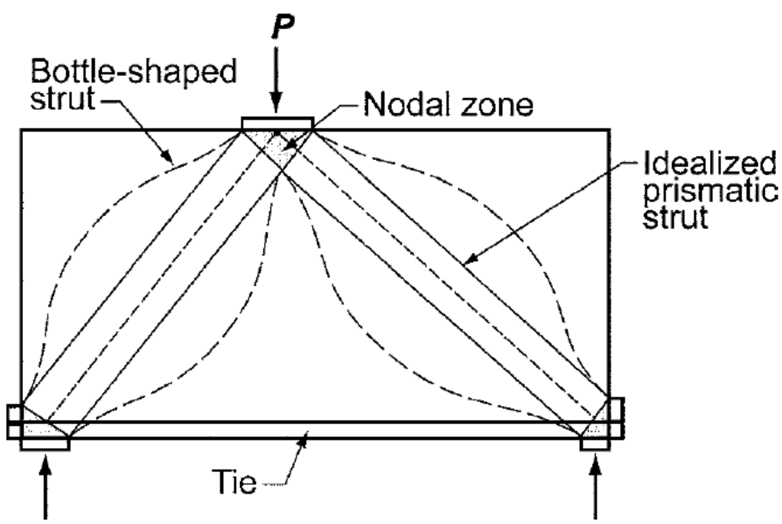

Figure 6.4 Description of strut and tie model
(ACI 318-11 Fig. RA.1.3)

Although the beams are classified as deep beams, beams were designed by the strength design method of the ACI 318-11 Code. According to Section R10.2, the strength of a member computed by the strength design method of the Code requires that two basic conditions be satisfied: (1) static equilibrium between the compressive and tensile forces acting on the cross section, and (2) compatibility between the stress and strain for the concrete and the reinforcement. Many tests have confirmed the linear distribution of strain across a reinforced concrete cross-section. The strain in the both the concrete and reinforcement is assumed to be directly proportional to the distance from the neutral axis. The flexure and shear beam designs were done in accordance with ACI 318-11 Chapter 10, Flexure and Axial Loads, and Chapter 11, Shear and Torsion.

6.1.1 Shear beams

The shear beams were designed to test varying stirrup spacing and check the design shear capacity versus the actual shear capacity of six bamboo reinforced concrete beams. From the results of the shear beam tests, the optimum stirrup spacing will be determined and the shear failure mechanism will be studied. The results of the shear beam tests can be found in Chapter 7. The following section provides background information on bamboo stirrups including previous research and then describes in greater detail the design of the shear beams.

6.1.1.1 Background

Stirrups are crucial in providing shear capacity to concrete members. In members with shear reinforcement, a portion of the shear strength is assumed to be provided by the concrete and the remainder is assumed to be provided by the shear reinforcement.

Shear failure, also known as “diagonal tension failure” is difficult to predict accurately. ACI 318-11 provides guidelines for shear reinforcement design based on the vertical shear force, V_u, that is present at any given cross-section of a member. The diagonal tensile forces are the real cause for shear failure, but these forces are not calculated. Traditionally, vertical shear force, V_u, has been taken to indicate the presence of diagonal tensile forces. Shear reinforcement is used to cross the diagonal tension cracks and keep them from opening.

Bamboo stirrups have been used by a number of researchers including Yamaguchi, Murakami and Takeda (2013), Mark and Russell (2011), Ghavami (2005), and Khare (2005). However, there is limited literature on the design of bamboo stirrups. Based on a study of the flexural performance of BRC beams, Yamaguchi et al. (2013) have stated that good load-carrying capacity, determined by when the main bamboo rebar ruptured, can be obtained if the number of bamboo stirrups is sufficient to prevent the shear failure of the beam.

Mark and Russell (2011) conducted a comparative study of BRC beams using different stirrup material for construction including bamboo, rattan cane, and steel. A beam performance index (BPI), developed by Mark and Russell, indicated the use of steel stirrups as the most economical.

Khare (2005) tested six BRC beams, three of which failed in shear. The bamboo stirrups used by Khare were 0.5 inch (12-mm) wide, open ‘U-shaped’ with no end hooks. Tonkin bamboo was used due to its flexible behavior. The stirrup spacing is not indicated, but Khare concluded that the stirrup design provided small resistance to shear forces.

Brink and Rush (1966, 2000) recommend vertical stirrups be made from wire or packing ties when available, but that they can also be improvised from split bamboo sections bent into U-shape and tied securely to the longitudinal reinforcement. Brink and Rush also suggest that stirrup spacing should not exceed 6 inches (152-mm).

Yamaguchi et al. (2013) utilized sliced Moso bamboo stirrups with a section size of 0.6 x 0.15 inches (15 x 3.8-mm) in their beam design. The bamboo stirrups were bent in hot water after the outer skin had been removed with a planer. A polymer cement mortar was sprayed on the reinforcement to waterproof it. Results from the beam tests showed that the fracture behavior of the BRC beams varied according to stirrup spacing. It was also concluded that good load-carrying capacity of a beam can be obtained if the number of stirrups is sufficient to prevent shear failure.

6.1.1.2 Design

To test the shear reinforcement capacity of the bamboo, six beams with varying stirrup spacing were designed. The beams were designed to fail in shear by specifying a very short clear span and loading the beam with a low *a/d* ratio. Stirrups were spaced at 4 inches (102-mm), 6 inches (152-mm) and 8 inches (204-mm) with two beams per stirrup spacing. Each shear beam had the same number of rows of flexural reinforcement (4 rows) with the only variable being the stirrup spacing. The shear controlled beam test matrix can be seen in Table 6.1. The values in the table were measure from the constructed bamboo cages which results in slightly varying *a/d* (shear span to depth) ratios and ρ_{bamboo} (ratio of A_{bamboo} to d) values. This was because the d values varied slightly between cages.

Table 6.1 Shear controlled beam test matrix

Beam ID	# Flexural Rows	Stirrup Spacing, s		Shear span, a		depth, d		a/d	A_{bamboo}		ρ_{bamboo}	Splice length	
		(in)	(mm)	(in)	(cm)	(in)	(cm)		(in^2)	(cm^2)		(in)	(mm)
S1	4	4.0	101.6	18.0	45.7	16.3	41.4	1.1	3.7	24.0	2.3%	n/a	n/a
S2	4	4.0	101.6	18.0	45.7	15.8	40.2	1.1	3.7	24.0	2.4%	n/a	n/a
S3	4	6.0	152.4	18.0	45.7	15.7	39.9	1.1	3.7	24.0	2.4%	n/a	n/a
S4	4	6.0	152.4	18.0	45.7	15.1	38.4	1.2	3.7	24.0	2.5%	n/a	n/a
S5	4	8.0	203.2	18.0	45.7	15.4	39.1	1.2	3.7	24.0	2.4%	n/a	n/a
S6	4	8.0	203.2	18.0	45.7	16.2	41.0	1.1	3.7	24.0	2.3%	n/a	n/a

The bamboo cages for the shear beam tests can be seen in Figure 6.5. The stirrups were made from the same Moso bamboo as the longitudinal reinforcement. The procedure for stirrup construction can be found in Section 6.2.1.2. The average cross sectional area of the stirrups was 0.20 in^2 (129-mm^2) which is the same cross sectional area as an imperial #4 rebar. The dimensions of the shear beams are 10 inches (25-cm) wide by 20 inches (51-cm) deep by 48 inches (122-cm) long. Each beam was tested under a single point load in the center of the beam with an *a/d* ratio between 1.1 and 1.2.

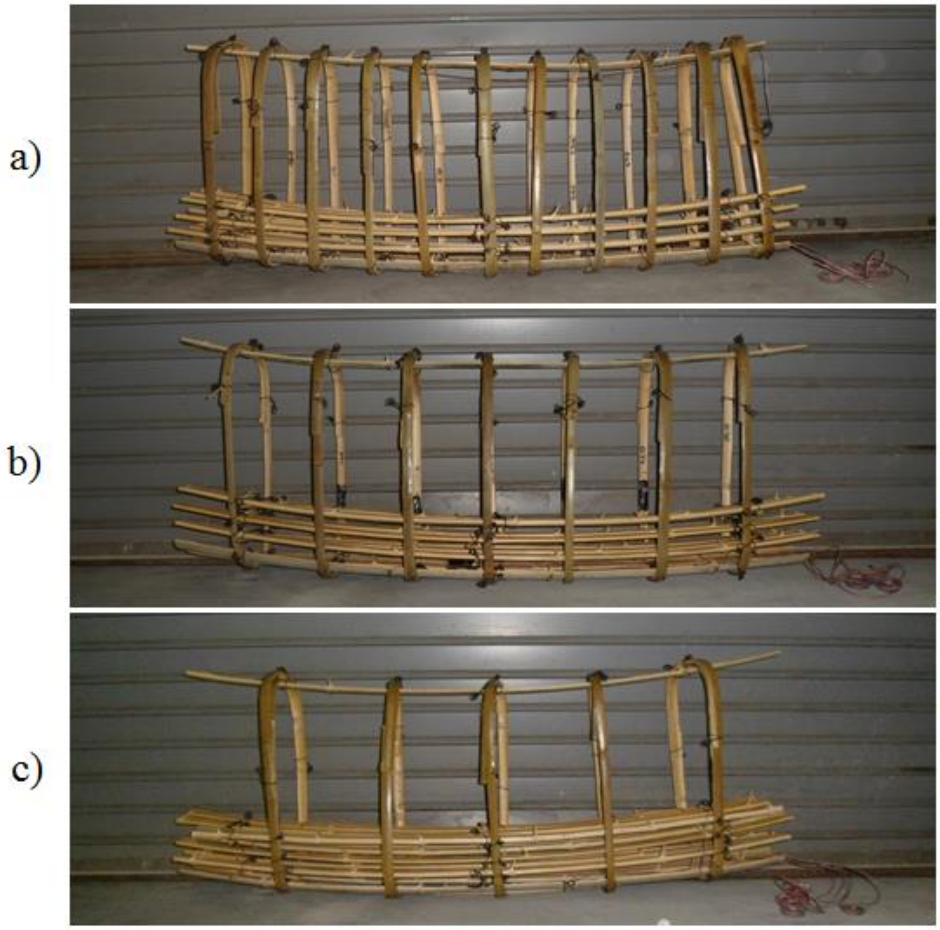

Figure 6.5 Varying stirrup spacing for shear beam tests
a) 4-inch (102-mm), b) 6-inch (152-mm), c) 8-inch (203-mm)

The *a/d* ratio is the shear span-to-depth ratio. In the *a/d* ratio, *a* is defined as the distance from the support to the point load and *d* is the distance from the top of the beam to the centroid of the longitudinal reinforcement. A higher *a/d* radio indicates a greater moment to be reached in the beam under the given loading while a lower *a/d* ratio means the flexure (moment) observed in the beam will be lower. This is explained visually in Figure 6.6.

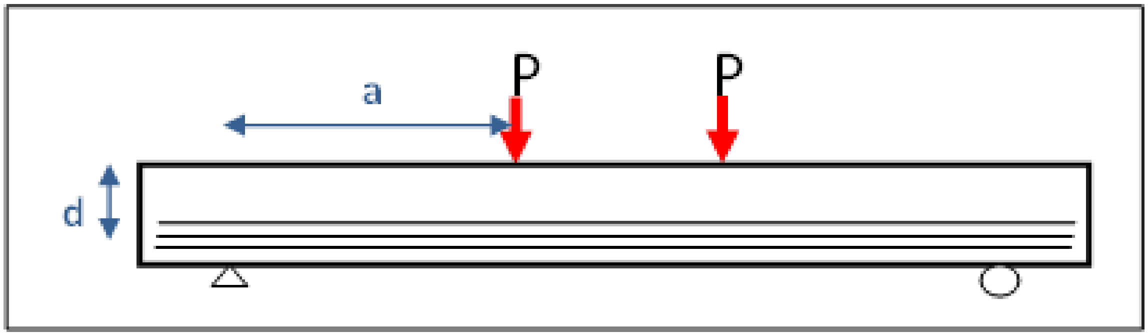

Figure 6.6 Definition of *a/d* ratio

The stirrup spacing, strain gauge placement and loading configuration for each beam can be seen in Figure 6.7. Strain gauges 1, 2, 4 and 5 were placed on stirrups located a distance *d* from the supports where the shear cracks were expected to form. A single strain gauge, 3, was placed on the bottom layer of longitudinal reinforcement to measure the strain in the reinforcement under the maximum moment. To measure the strain in the top and bottom faces of the concrete beam, reusable Bridge Diagnostics Inc. (BDI) strain transducers were installed near the mid span of the beam. Since the point load was applied directly at mid span, the BDIs were offset 4 inches (102-mm) from the beam's midpoint.

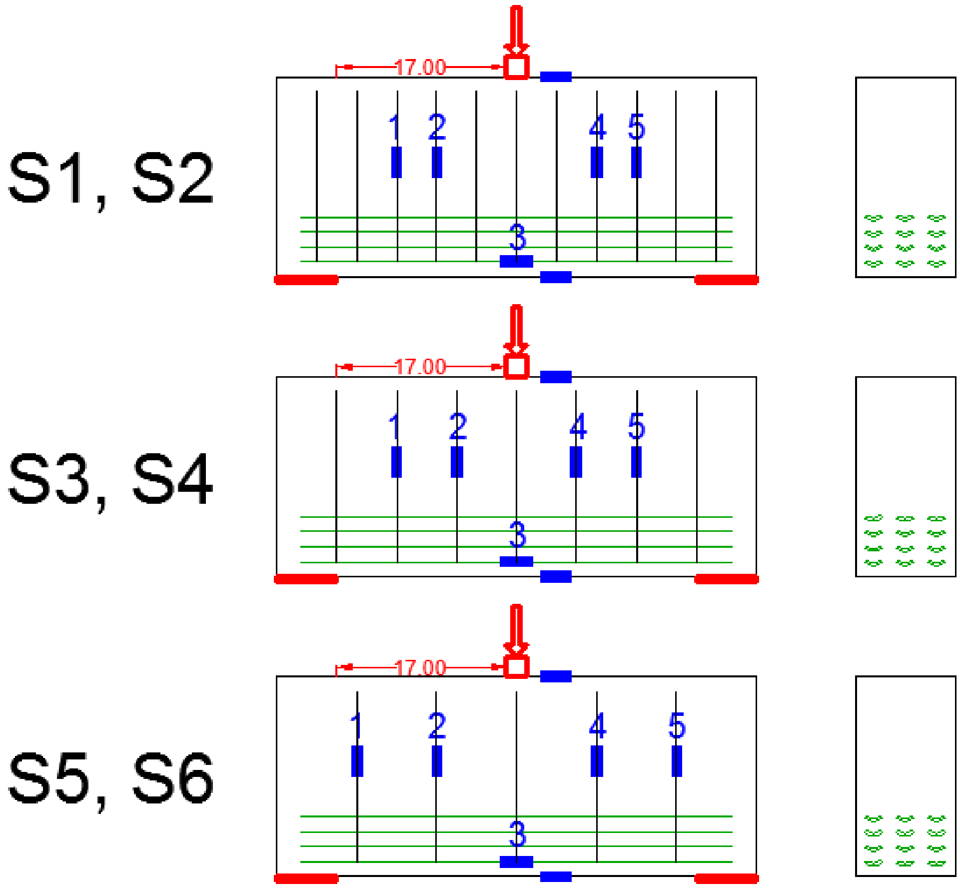

Figure 6.7 Loading conditions for shear beams
Note: distances are in inches

The major design parameters for the shear beams can be found in Table 6.2. Each model (*Yielding*, *Rupture 2/3*, *Yielding 0.6E*) has a different depth to the neutral axis, *c*, as shown in the table. Section 10.2.7 of the ACI 318 Code allows the use of an equivalent rectangular stress block to estimate the more exact concrete stress distribution. For the beam designs, all the design parameters were entered and the depth to neutral axis, *c*, was varied until the static equilibrium between the compressive and tensile forces acting on the cross-section was reached. For each failure model, the depth to the neutral axis is the same for each shear beams because each shear beam has the same number of rows of longitudinal reinforcement and therefore the same tensile forces acting on the beam's cross-section. The detailed MathCAD designs for the BRC beams can be found in Appendix C.

Table 6.2 Shear beam design parameters

Beam ID	Concrete strength, f'c (psi)	Concrete strength, f'c (MPa)	Bamboo strength, f_{bamboo} (ksi)	Bamboo strength, f_{bamboo} (MPa)	Depth to neutral axis, c (in): *Yielding* (in)	*Yielding* (mm)	*Rupture 2/3* (in)	*Rupture 2/3* (mm)	*Yielding 0.6E* (in)	*Yielding 0.6E* (mm)	# Flexural Rows	Stirrup spacing, s (in)	Stirrup spacing, s (mm)	Shear span, a (in)	Shear span, a (cm)
S1 S2	3000	20.7	18.7	129.1	2.836	72.0	2.543	64.6	2.245	57.0	4	4	101.6	17.0	43.2
S3 S4	3000	20.7	18.7	129.1							4	6	152.4	17.0	43.2
S5 S6	3000	20.7	18.7	129.1							4	8	203.2	17.0	43.2

The expected failure loads, P_{expect}, for the shear beams ranged from 67.0 – 97.9 kips (298.0 – 435.5 kN), and can be seen in Table 6.3. All of the shear beams were designed to fail in shear with the shear capacities ranging from 33.5 – 48.9 kips (149.0 – 217.5 kN). Since the beams were designed to fail in shear, the bamboo model had no impact on the expected beam capacities. This is because the model only affects the flexural capacities of the beam. Once the beams were tested, the expected shear beam capacities were compared to the actual beam capacities and the results can be found in Chapter 7.

Table 6.3 Expected shear beam capacities

Beam ID	M_{cr} (kip-ft)	V_n (kip)	V_u (kip)	M_n (kip-ft)		M_u (kip-ft)	P_{expect} (kip)	Expected Failure Mode
S1 S2	22.8	48.9	48.9	79.0	>	69.3	97.9	SHEAR
S3 S4	22.8	38.6	38.6	79.0	>	54.7	77.3	SHEAR
S5 S6	22.8	33.5	33.5	79.0	>	47.5	67.0	SHEAR

6.1.2 Flexure beams

The flexure beams were designed to test varying amounts of flexural reinforcement and check the design flexure capacity versus the actual flexural capacity. From the results of the flexure tests, the optimum level of flexural reinforcement will be determined and the flexural failure mechanism will be studied. The results of the flexure beam tests can be found in Chapter 7. The following sections provide background information on flexural bamboo reinforcement including previous BRC research and then describe the flexure beam design in greater detail.

6.1.2.1 Background

Flexural reinforcement, also known as longitudinal reinforcement, is necessary to increase the flexural capacity of a concrete beam. The tensile strength of concrete in flexure, also known as the modulus of rupture, is only about 10 to 15 percent of the compressive strength. A concrete beam in flexure will have zones in compression and tension, and reinforcement with high tensile strength is needed in the tension zones to carry the tensile stresses and prevent structural failure of the member.

A study by Clemson Agricultural College in 1950 concluded that the bamboo reinforcement in concrete beams increased the load capacity with increasing percentages of longitudinal bamboo reinforcement up to an optimum value of 3 to 4 percent (Glenn, 1950). This recommendation is also confirmed in a report prepared by the U.S. Naval Civil Engineering Laboratory (Brink & Rush, 1966). More recently, Khare (2005), at the University of Texas at Arlington, conducted a study on six beams with varying reinforcement ratios. This study showed that a beam with 4 percent reinforcement failed in crushing of the concrete in the top of the beam, indicating over-reinforcement.

Yamaguchi et al. (2013) utilized Moso bamboo as the main longitudinal reinforcement in their beams tests. Each section of reinforcement was made by tying two bamboo strips together with the inner sides facing each other. The bamboo strips had a section size of 1.2 x 0.4 inches (30 x 10-mm). The flexural capacity of the BRC beams was estimated using section analysis based on the Bernoulli-Euler assumptions. Two BRC beams were tested and the theoretical results were comparable to the experimental results when the bamboo's Young's modulus was reduced by 0.6 times.

6.1.2.2 Design

To test the flexural reinforcement capacity of the bamboo, four beams with varying bamboo reinforcement ratios have been prepared. Based on previous research, reinforcement ratios varying from 1.6 to 3.9 percent have been selected for the four flexure beams. Each beam had a different number of layers of longitudinal reinforcement, including 3, 4, 5 and 6 rows. The total area of longitudinal reinforcement for each beam was The stirrup spacing for the flexure beams remained constant at 6 inches (152-mm). The flexure controlled beam test matrix can be seen in Table 6.4. The values in the table were measure from the constructed bamboo cages which results in slightly varying *a/d* (shear span to depth) ratios and ρ_{bamboo} (ratio of A_{bamboo} to d) values. This was because the d values varied slightly between cages.

Table 6.4 Flexure controlled beam test matrix

Beam ID	# Flexural Rows	Stirrup Spacing, s		Shear span, a		depth, d		a/d	A_{bamboo}		ρ_{bamboo}	Splice length	
		(in)	(mm)	(in)	(cm)	(in)	(cm)		(in^2)	(cm^2)		(in)	(mm)
F1	3	6.0	152.4	25.0	63.5	17.0	43.2	1.5	2.8	18.0	1.6%	n/a	n/a
F2	4	6.0	152.4	25.0	63.5	15.8	40.1	1.6	3.7	24.0	2.4%	n/a	n/a
F3	5	6.0	152.4	30.0	76.2	15.5	39.4	1.9	4.7	30.0	3.0%	n/a	n/a
F4	6	6.0	152.4	38.0	96.5	14.5	36.7	2.6	5.6	36.0	3.9%	n/a	n/a

The longitudinal reinforcement was made from 1 inch (25-mm) wide bamboo strips. The average cross-sectional area of the longitudinal bamboo was 0.31 in^2 (200-mm^2) which is the same cross sectional area as an imperial #5 rebar. The bamboo reinforcement cage preparation is described in more detail in Section 6.2.1. The bamboo cage for flexure beam, F1, can be seen in Figure 6.8.

Figure 6.8 Bamboo cage for flexure beam, F1

The dimensions of the flexure beams are 10 inches (25-cm) wide by 20 inches (51-cm) deep by 90 inches (230-cm) long. These beams were tested under a monotonic loading of either one or two-point loads at a/d ratios of 1.5 to 2.6 to maximize the moment in each beam. Two point loads at a distance of 25 inches (635-mm) from the support were applied to the flexure beams F1, F2 and all the lap-spliced beams L1, L2 and L3. Since beam F2 was the control for the lap-spliced beams it had to be loaded under the same conditions. Beams F3 and F4 which had the highest quantities of flexural reinforcement were loaded under different conditions to prevent shear failure, since it was the goal of the tests to investigate the flexural failure mechanism. The flexure beam loading conditions can be seen in Figure 6.9.

Strain gauges were attached to the bottom layer of the flexural bamboo as well as to the stirrups at a distance ‘d’ from the support. For beams F1, F3 and F4, strain gauges 1, 2 and 3 were placed on each bamboo strip on the bottom row in the center of the beam. Strain gauges 4 and 5 were placed at a distance L/4 from the end of the beam and gauges 6 and 7 were placed on the stirrup a distance ‘d’ from the support. The strain gauge placement for beam F2 was similar to the lap-spliced beams since beam F2 was the control for the lap-spliced tests. The stirrup spacing, flexural reinforcement rows, strain gauge placement and loading configuration for each beam can be seen in Figure 6.9.

The major design parameters for the flexure beams can be found in Table 6.5. The detailed MathCAD designs for the BRC beams can be found in Appendix C.

Table 6.5 Flexure beam design parameters

Beam ID	Concrete strength, f'_c		Bamboo strength, f_{bamboo}		Depth to neutral axis, c (in)						# Flexural Rows	Stirrup spacing, s		Shear span, a	
					Yielding		*Rupture 2/3*		*Yielding 0.6E*						
	(psi)	(MPa)	(ksi)	(MPa)	(in)	(mm)	(in)	(mm)	(in)	(mm)		(in)	(mm)	(in)	(cm)
F1	3000	20.7	18.7	129.1	2.407	61.1	1.605	40.8	2.003	50.9	3	6	152.4	25	63.5
F2	3000	20.7	18.7	129.1	2.835	72.0	2.543	64.6	2.246	57.0	4	6	152.4	25	63.5
F3	3000	20.7	18.7	129.1	3.072	78.0	3.072	78.0	2.437	61.9	5	6	152.4	30	76.2
F4	3000	20.7	18.7	129.1	3.257	82.7	3.257	82.7	2.592	65.8	6	6	152.4	38	96.5

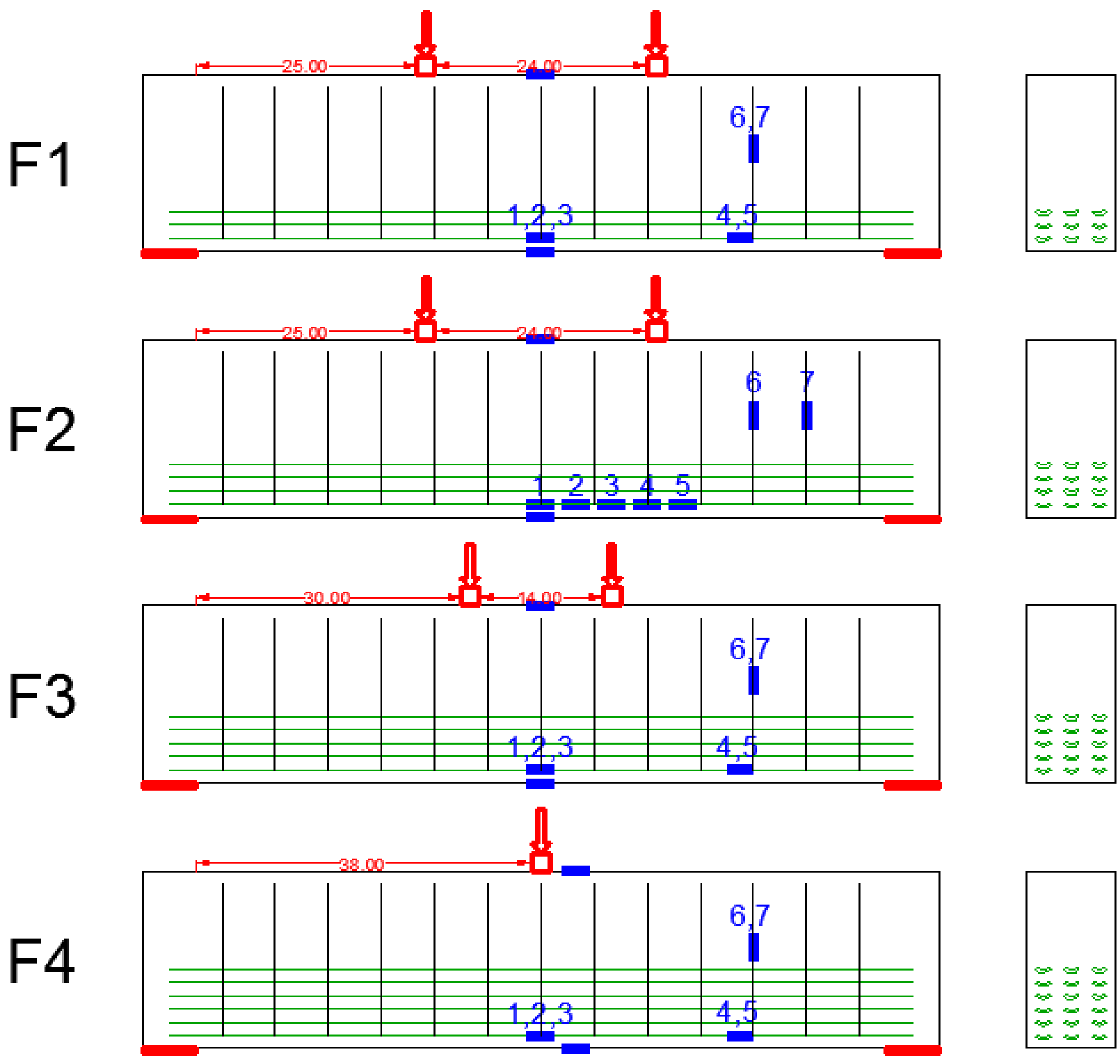

Figure 6.9 Loading conditions for flexure beams
Note: distances are in inches

The expected failure loads, P_{expect}, for the flexure beams ranged from 45.8 – 75.9 kips (203.7 – 337.6 kN), and can be seen in Table 6.7. All of the flexure beams were designed to fail in flexure, with the flexural capacities ranging from 47.7 – 83.8 kip-ft (64.7 – 113.6 kN-m). Beams F3 and F4 were expected to be compression controlled flexural failure for the *Yielding* and *Rupture 2/3* models. All the flexure beams were expected to be compression controlled flexural failure for the *Yielding 0.6E* model. The balanced depth to neutral axis, $c_{balanced}$, was determined for each of the models and is given in Table 6.6. The balanced depth to neutral axis is the depth of the neutral axis, c, at which the bottom layer of reinforcement will yield in a tension controlled flexural failure mode at the same moment as the concrete reaches its rupture strain limit of 0.003. Beams having c values less than $c_{balanced}$ will result in tension controlled flexural failure and beams with c values greater than $c_{balanced}$ will result in compression controlled flexural failure. Once the beams were tested, the expected flexural beam capacities were compared to the actual beam capacities and the results can be found in Chapter 7.

Table 6.6 Balanced depth to neutral axis ($c_{balanced}$) values

Model	$c_{balanced}$ (in)	$c_{balanced}$ (mm)
Yielding	2.852	72.4
Rupture '2/3'		
Yielding '0.6 E'	1.824	46.3

Table 6.7 Expected flexure controlled beam capacities

Model	Beam ID	M_{cr} (kip-ft)	V_n (kip)		V_u (kip)	M_n (kip-ft)	M_u (kip-ft)	P_{expect} (kip)	Expected Failure Mode
Yielding	F1	22.8	40.1	>	33.6	70.0	70.0	67.2	TENSION
	F2		38.6	>	38.0	79.1	79.1	75.9	TENSION
	F3		37.2	>	32.9	82.2	82.2	65.7	COMPRESSION
	F4		35.7	>	26.5	83.8	83.8	52.9	COMPRESSION
Rupture 2/3	F1	22.8	40.1	>	22.9	47.7	47.7	45.8	TENSION
	F2		38.6	>	33.6	70.0	70.0	67.2	TENSION
	F3		37.2	>	32.9	82.2	82.2	65.7	COMPRESSION
	F4		35.7	>	26.5	83.8	83.8	52.9	COMPRESSION
Yielding 0.6E	F1	22.8	40.1	>	28.4	59.1	59.1	56.8	COMPRESSION
	F2		38.6	>	30.5	63.6	63.6	61.0	COMPRESSION
	F3		37.2	>	26.5	66.4	66.4	53.1	COMPRESSION
	F4		35.7	>	21.4	67.9	67.9	42.9	COMPRESSION

6.1.3 Flexure lap-spliced beams

The flexure lap-spliced beams were designed to test varying lap-spliced lengths to determine the minimum lap-splice length required to achieve full capacity in the flexural reinforcement. From the results of the beam tests, the optimum lap-splice length will be determined and the lap-splice failure mechanism will be studied. The following section provides background information on bamboo lap-splicing and then describes in greater detail the design of the lap-spliced beams.

6.1.3.1 Background

A lap-splice is made when two pieces of reinforcement are overlapped to create a continuous piece. According to ACI 318-11 Chapter 12, the lap-splice length of rebar depends on many factors including concrete strength, rebar grade, size, spacing as well as the depth of concrete above the bars and the concrete cover.

Lap-splicing is a common practice in reinforced concrete construction, but currently there exits very little literature on the performance of lap-spliced bamboo. In the U.S. Navy report on Bamboo Reinforced Concrete Construction, Brink and Rush (1966, 2000) suggest that bamboo lap-splices should overlap at least 25 inches (635-mm) but gives no other details on the construction of the splices.

6.1.3.2 Design

In this research, three beams were tested using different lengths of lap-splicing for the bottom layer of reinforcement. The lap-splice lengths were chosen based on the results from the component bond strength tests described in Chapter 5. The pullout tests indicated the range of development lengths to be from 15 to 31 inches (38 – 79-cm) with an average of 25 inches (51-cm). Three different lap-splice lengths were chosen to be tested: 12-inch (30-cm), 18-inch (46-cm) and 24-inch (61-cm). Each lap-splice was tied with steel rebar ties spaced at about 5 inches (127-mm). Each lap-spliced beam had 4 layers of flexural reinforcement and had stirrups spaced at 6 inches (152-mm) with the exception of along the splice, where the stirrup spacing was 4 inches (102-mm). The lap-spliced beam testing matrix can be seen in Table 6.8 and the three varying splice lengths of the bamboo can be seen in Figure 6.10.

Table 6.8 Lap-splice beam test matrix

Beam ID	# Flexural Rows	Stirrup Spacing, s		Shear span, a		depth, d		a/d	A_{bamboo}		ρ_{bamboo}	Splice length	
		(in)	(mm)	(in)	(cm)	(in)	(cm)		(in^2)	(cm^2)		(in)	(mm)
L1	4	6.0*	152.4	25.0	63.5	14.7	37.3	1.7	3.7	24.0	2.5%	12	30
L2	4	6.0*	152.4	25.0	63.5	15.4	39.1	1.6	3.7	24.0	2.4%	18	46
L3	4	6.0*	152.4	25.0	63.5	15.5	39.4	1.6	3.7	24.0	2.4%	24	61

** Note: Lap-splice lengths are surrounded by stirrups with a spacing of 4.0 inches (102 mm)*

Figure 6.10 Lap-splicing of flexural reinforcement

The dimensions of the lap-spliced beams are the same as the flexure beams: 10 inches (25-cm) wide by 20 inches (51-cm) deep by 90 inches (230-cm) long. The lap-spliced beams were designed the same as the flexure beam, F2, apart from the bottom layer of longitudinal reinforcement being spliced. Strain gauges were spaced evenly at 4 inches (102-mm) on center along the lap-splice as well as on stirrups a distance ‘d’ from the supports.

These beams were tested under a monotonic loading of two-point loads at an *a/d* ratio of 1.5 to 2.6. Two point loads at a distance of 25 inches (64-cm) from the support and 24 inches (61-cm) between loads, were applied to the lap-spliced beams, L1, L2, and L3 as well as to the flexure beam, F2, which was the control for the lap-spliced beams. The two point loading condition was specified for the lap-spliced beams to ensure the entire lap-splice was in the region of constant moment.

According to ACI 318-11 Section 21.6.3, it is necessary to provide closer stirrup spacing around lap-spliced regions for earthquake resistant structures. The closer stirrup spacing is needed because of the uncertainty of moment distributions along the height and to confine the concrete around the splice. For each lap-spliced beam, a stirrup spacing of 4 inches (102-mm) was provided around the splice. The stirrup spacing, lap-splice length, strain gauge placement and loading configuration for each beam can be seen in Figure 6.11.

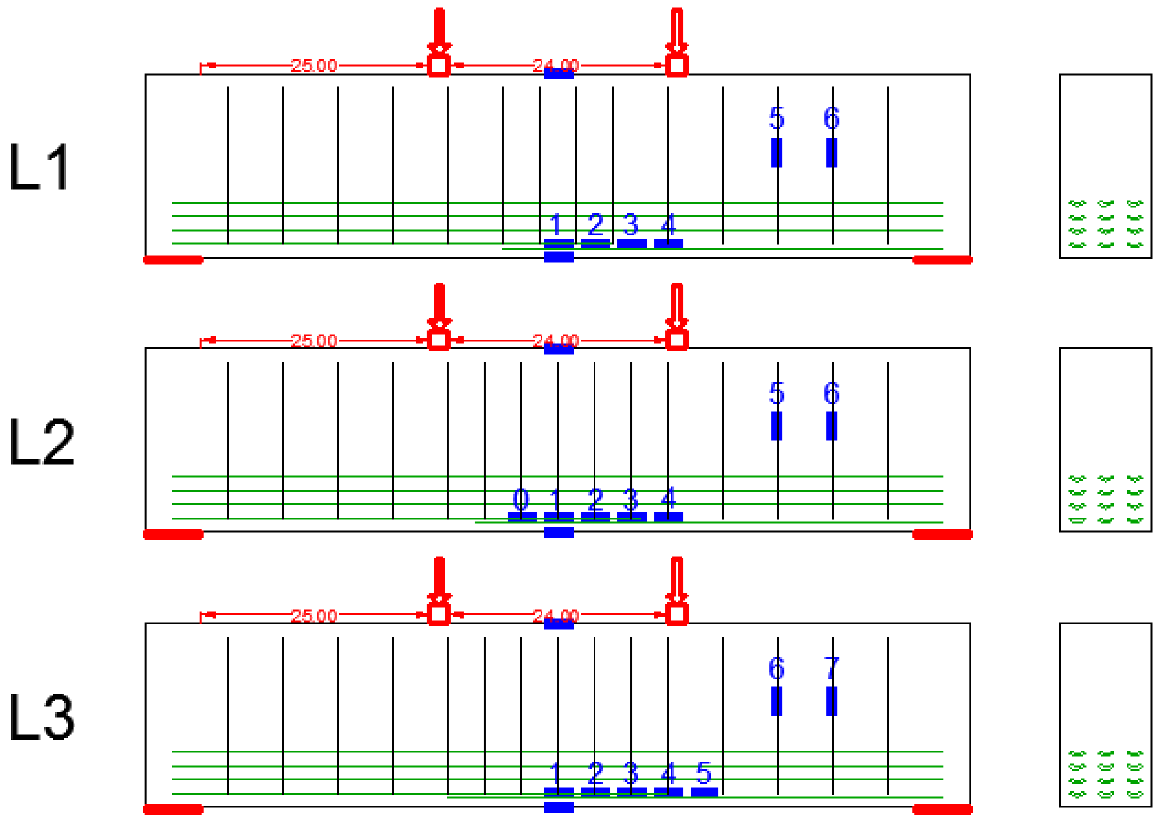

Figure 6.11 Loading conditions for lap-spliced beams
Note: distances are in inches

The design parameters for the lap-spliced beams are the same as for beam F2 and can be seen in Table 6.9. In the designs for the lap-spliced beams, it was assumed that the splices would be sufficient, so the design capacities of the lap spliced beams were the same as for beam F2. The lap-spliced beam expected capacities can be seen in Table 6.10. The MathCAD designs for the BRC beams can be found in Appendix C.

Table 6.9 Lap-spliced beam design parameters

Beam ID	Concrete strength, f'_c		Bamboo strength, f_{bamboo}		Depth to neutral axis, c (in)						# Flexural Rows	Stirrup spacing, s		Shear span, a	
					Yielding		*Rupture 2/3*		*Yielding 0.6E*						
	(psi)	(MPa)	(ksi)	(MPa)	(in)	(mm)	(in)	(mm)	(in)	(mm)		(in)	(mm)	(in)	(cm)
L1 L2 L3	3000	20.7	18.7	129.1	2.835	72.0	2.543	64.6	2.246	57.0	4	6	152.4	25	63.5

Table 6.10 Lap-spliced beam expected capacities

Model	Beam ID	M_{cr} (kip-ft)	V_n (kip)		V_u (kip)	M_n (kip-ft)	M_u (kip-ft)	P_{expect} (kip)	Expected Failure Mode
Yielding	L1, L2, L3	22.8	38.6	>	38.0	79.1	79.1	75.9	TENSION
Rupture 2/3	L1, L2, L3	22.8	38.6	>	33.6	70.0	70.0	67.2	TENSION
Yielding 0.6E	L1, L2, L3	22.8	38.6	>	30.5	63.6	63.6	61.0	COMPRESSION

6.2 Beam fabrication

Fabrication of the BRC beams included bamboo cage preparation, strain gauge installation, and casting of the beams. The bamboo cage preparation was by far the most time intensive process as it involved harvesting, splitting the culms into the proper widths, bending the bamboo and constructing the stirrups, assembling the cage and finally waterproofing.

6.2.1 Cage preparation

The bamboo used for the reinforcement cages was harvested and split in a similar manner as the bamboo used for the component tests. The harvesting and splitting of the bamboo is described in Chapter 4.

6.2.1.1 Bending

The purpose of bending the bamboo was to form the rectangular closed stirrups. There are various methods used to bend bamboo including heating with a propane torch, soaking in water and steaming. The steaming method was selected to bend the bamboo into the required stirrup shape. For this method, it is necessary to heat the bamboo above boiling (225^{o}F) to soften the lignin and pectin between the fibers. This temperature should be maintained for 30 minutes per 0.5 inch (13-mm) for sufficient steaming. To accomplish the bending, both a steam box and bending forms had to be constructed.

The steam box was constructed from a 6-inch (152-mm) PVC pipe with two end caps. Dowel rods with a diameter of 0.25 inch (6-mm) were used to support the bamboo strips within the PVC pipe in four layers. To secure the dowel rods, holes were drilled into the side of the pipe and the rods were inserted into the holes. An Earlex SS77USG Steam Generator was used to produce the steam and was connected to the side of the PVC pipe via the hose provided with the generator. Highland Woodworking[5] provides a datasheet with the steamer specs as well as a steamer manual. Detailed instructions to build a steam box are also included on their website. The supports for the steam box were constructed from 2x4 lumber. The steam box set up can be seen in Figure 6.12 and a close-up of the stirrup strips can be seen in Figure 6.13.

[5] http://www.highlandwoodworking.com/earlex-steam-generator.aspx

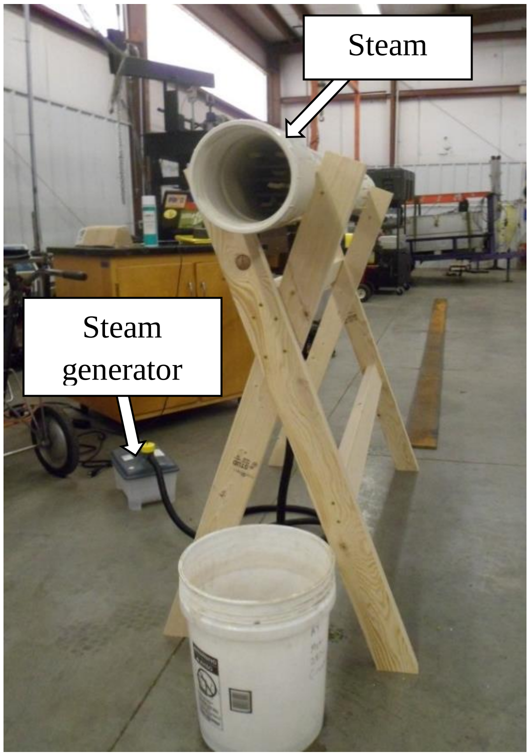

Figure 6.12 Steam boxe set-up

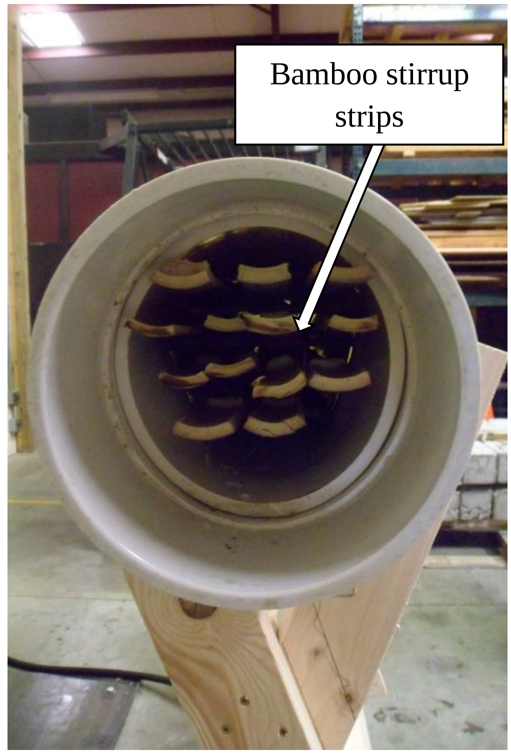

Figure 6.13 Bamboo strips in steam box

The bending forms were constructed from a sheet of OSB sheathing with 2x4 lumber screwed to it outlining six stirrups. A 1 inch (25-mm) gap was provided between the 2x4 lumber where the steamed bamboo pieces were bent into shape and left to cool. The bending forms can be seen in Figure 6.14 and the bending process can be seen in Figure 6.15. Once the bamboo stirrups cooled they were removed from the bending forms and secured into shape with rebar ties as shown in Figure 6.16.

Figure 6.14 Bending forms

Figure 6.15 Bending the bamboo stirrups into the forms

Figure 6.16 Securing stirrups with rebar ties

6.2.1.2 Stirrup construction

Bamboo stirrups have been made with 0.5 inch (13-mm) wide strips. The outside dimensions of each stirrup are roughly 7 inches (178-mm) wide by 17 inches (432-mm) tall to provide 1.5-inch (38-mm) cover to the surface of the concrete beams. The stirrup shape was made by heating strips of bamboo in a steam box and then bending them into wooden forms. This is a new stirrup design and was used for all the beams. The closed stirrup shape has an advantage over open stirrups when torsion is expected. The closed stirrups were also expected to confine the core of concrete better than open stirrups. However, under the monotonic loads of this research, no advantage was. A typical stirrup can be seen in Figure 6.17.

Figure 6.17 Bamboo closed stirrup

6.2.1.3 Cage assembly

Bamboo cages, similar to the traditional rebar cages, were used to reinforce the beams and increase their shear and moment capacities. The constructed bamboo cages can be seen in Figure 6.18. The cages were constructed from longitudinal bamboo strips 1 inch (25-mm) wide and 87 inches (220-cm) long and bamboo stirrups which are described in Section 6.2.1.2. All the connections were made with 6-inch (152-mm) rebar ties.

Figure 6.18 Constructed bamboo cage

The first step of the cage assembly was to gather the required number of stirrups for the cage as well as the total number of longitudinal bamboo strips. Next, the longitudinal bamboo strips were tied together in separate layers of three strips per layer. The longitudinal bamboo strips were tied together to bamboo splints spaced evenly about 25 inches (64-cm) apart for the flexural and lap-splice beams and about 40 inches (102-cm) for the shear beams. For best results, the rebar ties connecting the two outside strips to the bamboo splint should not be oriented parallel to each other. A horizontal spacing of about 1 inch (25-mm) was provided between each strip of longitudinal bamboo.

Once all the required layers of longitudinal reinforcement were tied together, a single layer was tied to the bottom of the stirrups and a single bamboo strip was tied to the top of the stirrups to provide stability to the cage and help maintain its shape. For the bottom layer of longitudinal reinforcement it is necessary to securely tie each stirrup to the two outside strips of bamboo. A single longitudinal layer, as well as a partially constructed cage can be seen in Figure 6.19.

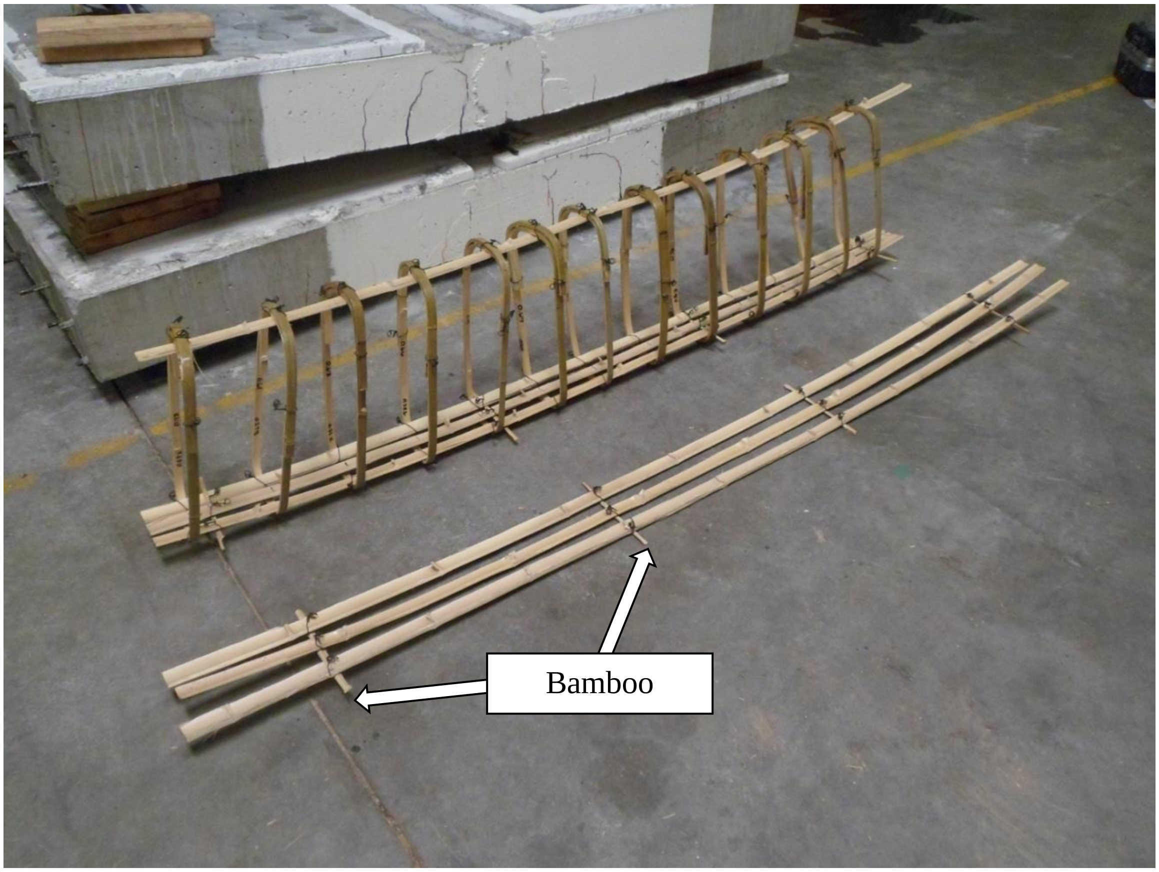

Figure 6.19 Partially constructed bamboo cage

Once the bottom layer of longitudinal reinforcement was secured, the strain gauges were installed. It was necessary to install the strain gauges to the bottom layer prior to installing the remaining layers because the bottom layer would not be accessible once the remaining layers of longitudinal reinforcement were installed. The strain gauge installation is described in further detail in Section 6.2.2.

After the strain gauges were installed, the remaining layers of longitudinal reinforcement were tied in place. A vertical spacing of about 1 inch (25-mm) was provided between layers of longitudinal reinforcement. It is not necessary to tie each stirrup to the longitudinal layers. It is sufficient to tie the longitudinal layers to the stirrups on each side at 25-inch (64-cm) spacing.

The steps of the cage assembly are outlined below:

1. Gather the required number of stirrups and longitudinal bamboo strips
2. Tie longitudinal strips in separate layers of three strips per layer
3. Tie stirrups at their designed spacing distance to a single layer of longitudinal reinforcement
4. Tie a single longitudinal strip to the under face of the top side of the stirrups to give stability to the cage and help it maintain its shape
5. Install strain gauges where needed
6. Install remaining layers of longitudinal reinforcement
7. Waterproof cages

6.2.1.4 Waterproofing

After the cages were fully constructed and allowed to dry in a controlled environment for over one month, the bamboo was waterproofed with Thompson's WaterSeal, model number TH.024101-16. Waterproofing the bamboo is necessary to mitigate the problem of poor bonding caused by the swelling and subsequent shrinkage of the bamboo after absorbing moisture from the fresh concrete. The sealant was applied to the bamboo cages with a paint sprayer and although care was taken to cover all surfaces, it was difficult to ensure complete coverage. The performance of the waterproofing will be assessed during the beam testing.

6.2.2 Strain gauge installation

Strain gauges were installed prior to full assembly of the bamboo cages. The strain gauges were placed at location of highest expected shear and moment within the beams. Strain gauges will be attached to the middle of the bottom layer of the flexural bamboo as well as to the stirrups at a distance ‘d’ from the support. The placement of the strain gauges can be seen for each beam in Section 6.1.

The strain gauges were purchased from Vishay Micro Measurements and were classified as general purpose strain gauges with a linear pattern. The type of strain gauges used were CEA-06-250UW-120/P2. The gauges were 0.250 inches (6-mm) in length, with a resistance of 120 ohms and a strain limit of approximately ±5%. The label P2 indicates that lead wires are attached to the gauges and no soldering is required. More detailed specifications can be found from the Vishay Micro Measurements website[6].

The gauges were installed on the bamboo using M-Bond AE-10 adhesive which was also purchased from Vishay Micro Measurements and allowed to cure for 8 hours. It is very important to install the strain gauges correctly to increase the chances of obtaining useful data. The harsh environment of wet concrete can cause the strain gauges to malfunction, and they must be properly protected with a polysulfide coating (M-Coat J). The strain gauge installation on the lap-spliced reinforcement can be seen in Figure 6.20.

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[6] http://www.vishaypg.com/micro-measurements/

Figure 6.20 Strain gauge installation on lap-spliced reinforcement

The procedure for the strain gauge installation is outlined below:

1. Sand down location with 60 grit sandpaper.
2. Blow off dust with compressed air.
3. Clean area with a shop towel dipped in acetone.
4. Allow surface to dry completely.
5. Cut tape and fold over one edge.
6. Lay tape with sticky side up on a clean plastic plate.
7. Attach gauge to tape with shiny side (the face with the solder connections) to the sticky surface of the tape.

8. Place a small amount of adhesive (M-Bond AE-10) on the bamboo surface and then press the gauge into place. The adhesive will harden beyond use at around 20 minutes.
9. Place a small piece of silicone rubber over the gauge and then clamp in place with spring clamp.
10. Allow to cure for at least 8 hours at 70F.
11. Once epoxy has cured, remove clamps and silicone rubber. The tape can be carefully removed or left in place.
12. Cover solder connections with Teflon tape. It is important that the solder connections are protected from the M-Coat J polysulfide coating.
13. Coat the strain gauges completely with M-Coat J, extending at least 0.5 inch (12-mm) beyond the edge of the strain gauge and sealing the gauge off completely where the lead wires attach.

6.2.3 Formwork and casting

6.2.3.1 Formwork design

The formwork for the beams was made from 2x4 lumber and 7/16 inch (11-mm) OSB sheathing as shown in Figure 6.21. A gap under the beam was provided for the forklift. The inside joints were sealed with caulk to prevent concrete from flowing out through the gaps. After the joints were sealed the inside of the forms were coated with WD-40 as a de-bonding agent.

To provide the 1.5-inch (38-mm) cover, small wood blocks were screwed into the forms as shown in Figure 6.22. The blocks also helped to straighten out the curvature in bamboo cages which occurred as the bamboo dried out. The completed forms can be seen in Figure 6.23.

Figure 6.21 Beam formwork preparation

Figure 6.22 Installation of block spacers

Figure 6.23 Completed forms with bamboo cages

6.2.3.2 Concrete casting

The concrete used for the beams was a normal strength mix of 3000 psi (21 MPa), 28-day compressive strength made using Portland Cement Type I/II. A small aggregate size (#89) was ordered to ensure that aggregate did not get stuck in between the bamboo and create voids in the concrete. Also, to prevent voids, the beams were cast on their sides so that the longitudinal bamboo strips were oriented on their edges which allowed for concrete to flow between them easily. The concrete had a high slump of 8 inches (20-cm) which was useful to provide good consolidation but may have caused lower bond strength between the bamboo and concrete.

The concrete was cast around noon on January 14, 2014 in Clemson, SC. During casting, the air temperature on day of pour was 66 ºF (19 ºC). The concrete was cast inside since temperatures were expected to drop to below freezing outside during the night. A front discharge mixer truck delivered the concrete as shown in Figure 6.24. The beams were finished smooth with a hand trowel as shown in Figure 6.25.

Figure 6.24 Front discharge mixer truck

Figure 6.25 Finishing concrete with a steel finishing trowel

Compressive strength cylinders were prepared from the same concrete mix as used for the beams and left to cure next to the beams as to be in the same environment. The 7, 14 and 28-day compressive strength was tested with 3 samples at each time period according to ASTM C39. The ends of the samples were first cut straight as shown in Figure 6.27 then they were tested in the compression testing machine (CTM) in Figure 6.28. The average 28-day compressive strength was 3027 psi and the compressive strength of the concrete at different ages is shown in Table 6.11 and Figure 6.26.

Table 6.11 Compressive strength of concrete

Age	1	2	3	Average Stress (psi)
7 day	1871	1672	1722	1755
14 day	2086	2006	2119	2070
28 day	2823	3295	2963	3027

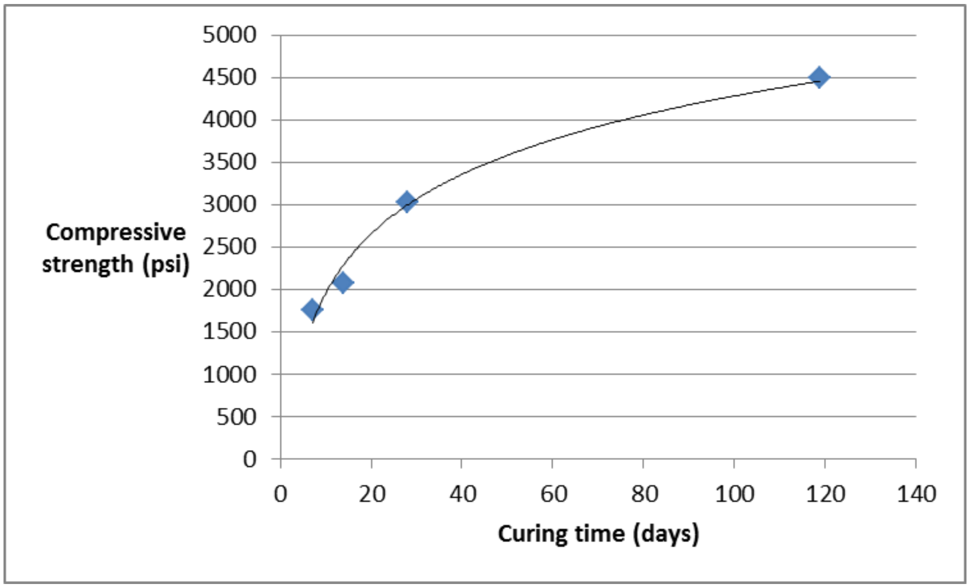


Figure 6.26 Compressive strength of concrete mix

Figure 6.27 Preparing sample ends

Figure 6.28 Compression Testing Machine

6.3 Beam test setup

The beam test setup consisted of positioning the beam in the UTM, fabrication of the Wheatstone bridge circuit box, and connecting and calibrating of all instrumentation. All the beams were tested in the universal testing machine (UTM) with a hydraulic actuator with a capacity of 150-kip (667 kN). The testing setup varied depending on the type of beam being tested and details for each beam design and setup can be found in Section 6.1. Before each test, the beams were positioned in the UTM and painted white so that the cracks could easily be identified and marked with different colors depending on the loading level. The instrumentation setup will be described in the next sections.

6.3.1 Wheatstone bridge circuit

To measure strain of a bonded resistance strain gauge, it must be connected to an electric circuit that is capable of measuring very small changes in resistance which correspond to strain. A Wheatstone bridge circuit was built to measure the resistance change across the gauges. The Wheatstone bridge consists of four resistive arms with an excitation voltage, V_{EX}, that is applied across the bridge as shown in Figure 6.29.

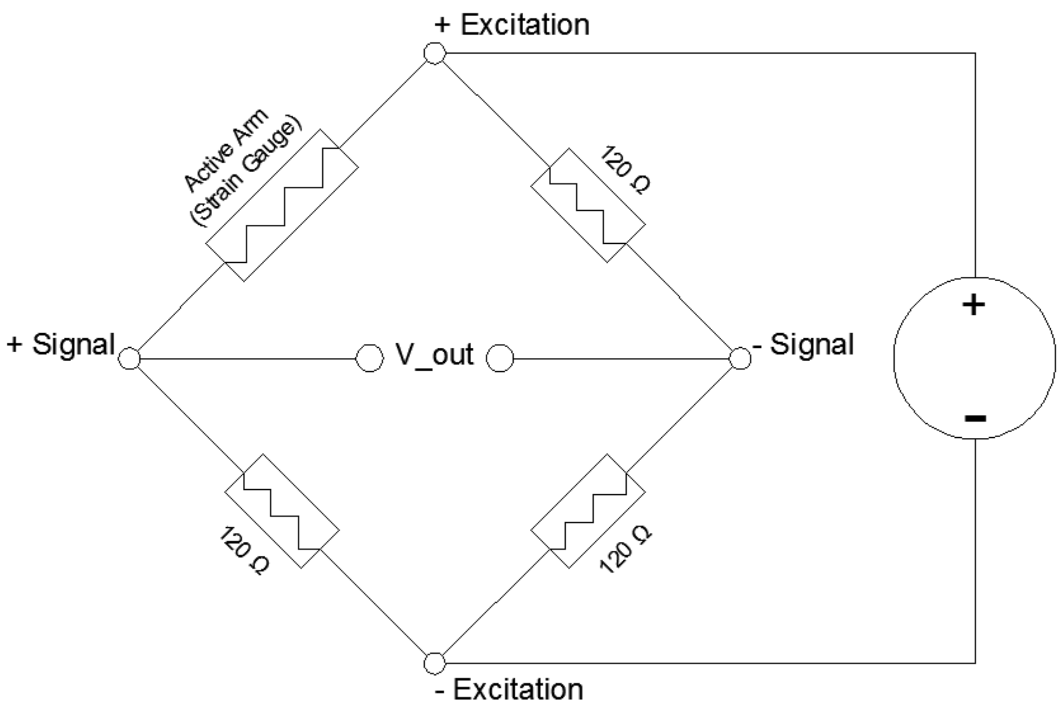


Figure 6.29 Wheatstone bridge circuit

A total of eight Wheatstone bridge circuits were soldered to a circuit board and a 2-pole PC mount terminal block was used to connect the lead wires from each strain gauges to the active arm of the Wheatstone Bridges. The Wheatstone bridge circuits can be seen in Figure 6.30 and the completed circuit box with the connected strain gauge lead wires can be seen in Figure 6.31.

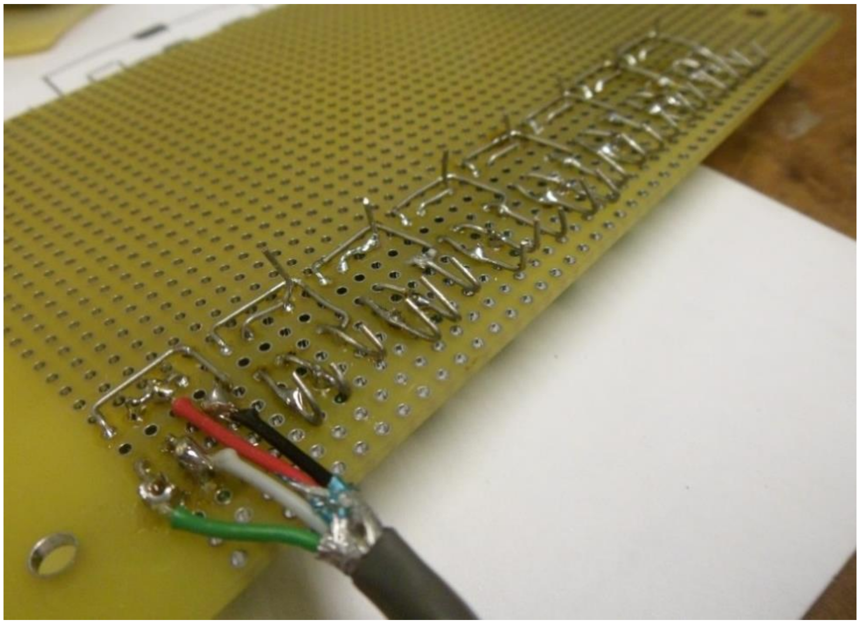


Figure 6.30 Wheatstone bridge circuits

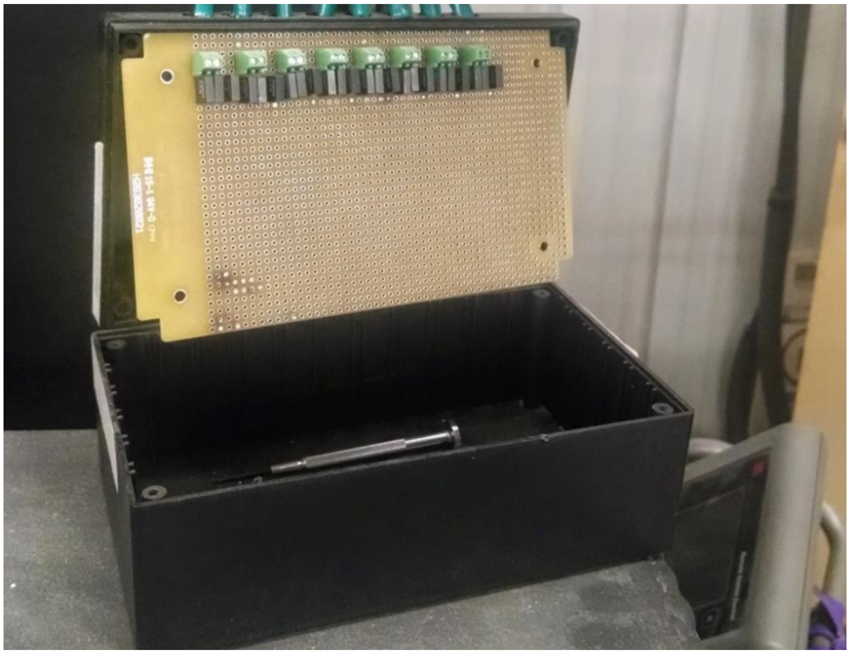


Figure 6.31 Wheatstone bridge circuit box

6.3.2 Instrumentation setup

Before every test, all the strain gauges, BDIs and LVDT string pots were calibrated. Calibration calculations were done using a Sensors Channel spreadsheet developed in a separate experimental test program which can be found in Appendix E. Figure 6.32 shows the instrumentation setup including the strain gauge lead wires, LVDT string pot and BDI.

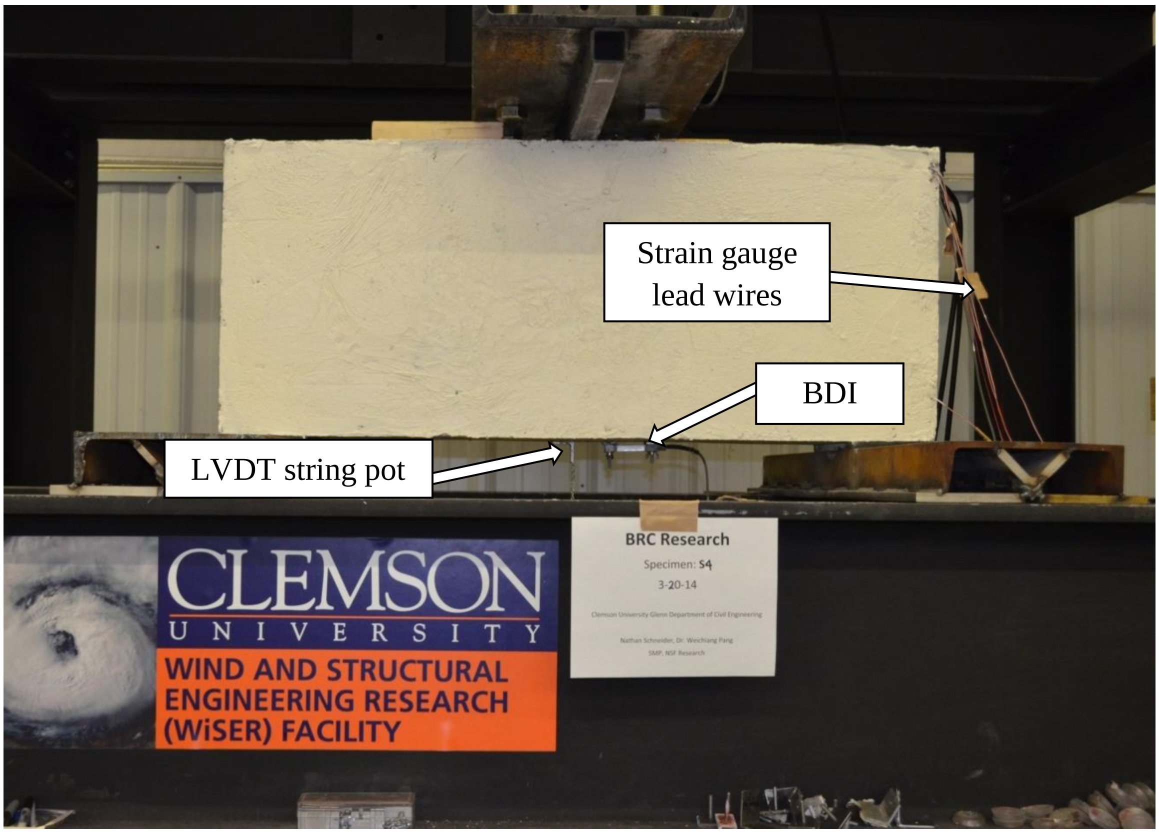


Figure 6.32 Instrumentation setup

6.3.3 Loading protocol

All beams were tested under displacement controlled loading. The loading protocol for the beams was targeted at approximately 20, 40, 60% of the expected failure load and then the beams were loaded until failure. However, due to the uncertainty of how the bamboo reinforcement would perform, the loading protocol varied slightly from the 20, 40, 60% loading levels. The actual loading levels can be seen in Table 6.12 through Table 6.14.

Table 6.12 Loading protocol for shear beams (kip)

	S1	S2	S3	S4	S5	S6
20%	n/a	29	20	20	20	20
40%	n/a	50	35	35	30	30
60%	n/a	75	50	50	45	45
Expected	95.4	89.8	77.3	77.3	67	67
Actual	110	78.2	115.7	103.1	94.9	75.6

Table 6.13 Loading protocol for flexure beams (kip)

	F1	F2	F3	F4
20%	12	15	15	12
40%	24	30	30	20
60%	36	40	45	30
Expected	56.8	61	53.1	42.9
Actual	51.3	64.8	62.4	43.9

Table 6.14 Loading protocol for flexure lap-spliced beams (kip)

	L1	L2	L3
20%	15	15	15
40%	30	30	30
60%	45	45	45
Expected	61	61	61
Actual	61.3	61.2	57.2

Chapter 7 – BEAM TEST RESULTS

The testing took place in the Wind and Structural Engineering Research (WiSER) Facility at Clemson University with a universal testing machine (UTM) incorporating a 150-kip actuator. A total of 13 BRC beams were tested and the strain in the concrete and in the bamboo reinforcement at strategic locations was measured. Each beam was loaded until failure and the failure mechanism of each beam was investigated. After testing, each beam was broken apart with a sledge hammer and chisel to expose the bamboo reinforcement to further explain the failure mechanism. The beam examination notes can be found in Appendix D. The results of the beam tests can be found in the following sections.

7.1 Shear beams

To test the shear reinforcement capacity of the bamboo, six beams with varying stirrup spacing were designed. The beams were designed to fail in shear by specifying a very short clear span and loading the beam with a low *a/d* ratio. Stirrups were spaced at 4 inches (102-mm), 6 inches (152-mm) and 8 inches (204-mm) with two beams per stirrup spacing. Each shear beam had the same number of rows of flexural reinforcement (4 rows) with the only variable being the stirrup spacing. The results from the shear controlled beam tests including the force-displacement curves, bamboo model comparisons, bamboo and steel comparisons, beam failure analysis and the strain analysis are presented in the following sections.

7.1.1 Force-displacement curves

Each of the force-displacement curves for the shear controlled beams can be seen from Figure 7.1 through Figure 7.6. The loading levels were targeted at approximately 20, 40, 60% of the expected failure load and then the beams were loaded until failure. Beam S1 was accidentally loaded prior to testing, so it was not loaded in stages, but rather was loaded directly to failure as shown in Figure 7.1.

The damage in the beams is evident after each loading cycle by the widening of the force-displacement curves. After each loading cycle, the beam experiences a greater deflection under a given force.

A few of the shear controlled beams exhibited a gradual failure. Apart from beam S1, the beams S2, S3 and S4 having a stirrup spacing of 4 and 6 inches (102 and 152-mm) showed a somewhat ductile failure. This indicates that a closer stirrup spacing allows stresses to transfer to the remaining stirrups and allow the beam to retain most of its capacity even after some of the stirrups have failed. This type of failure can be seen in Figure 7.2.

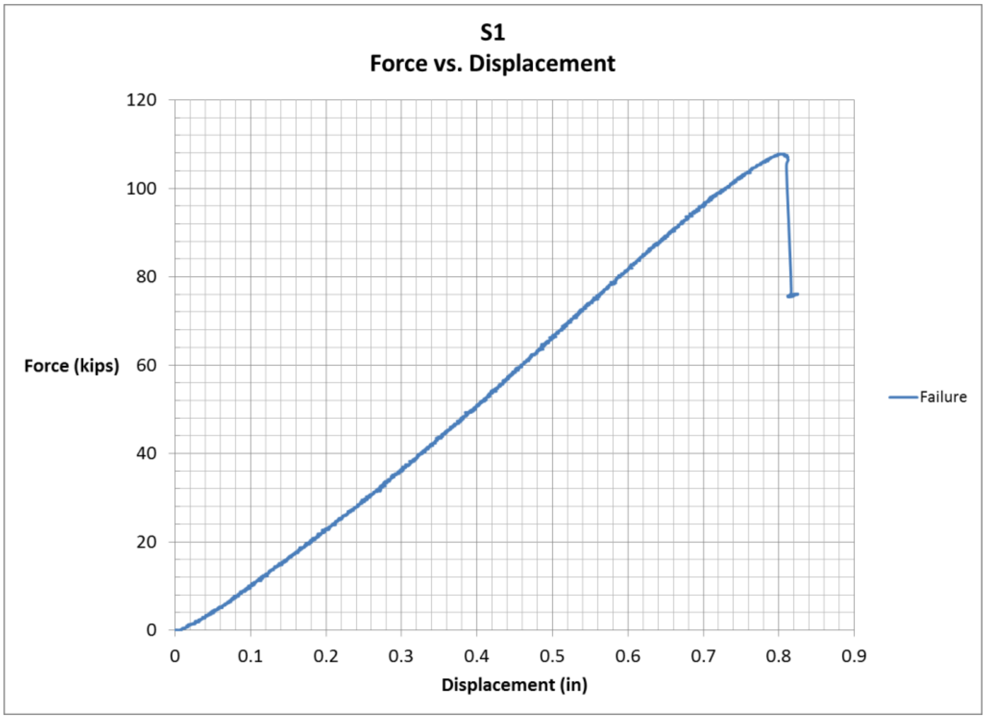


Figure 7.1 Force-displacement curve for S1

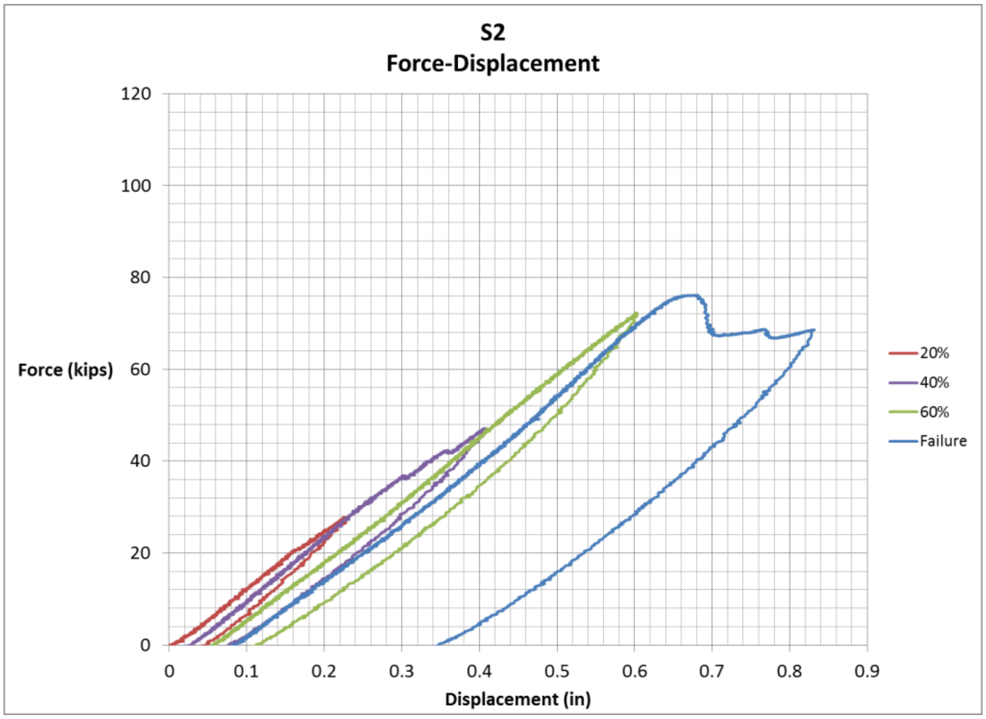


Figure 7.2 Force-displacement curve for S2

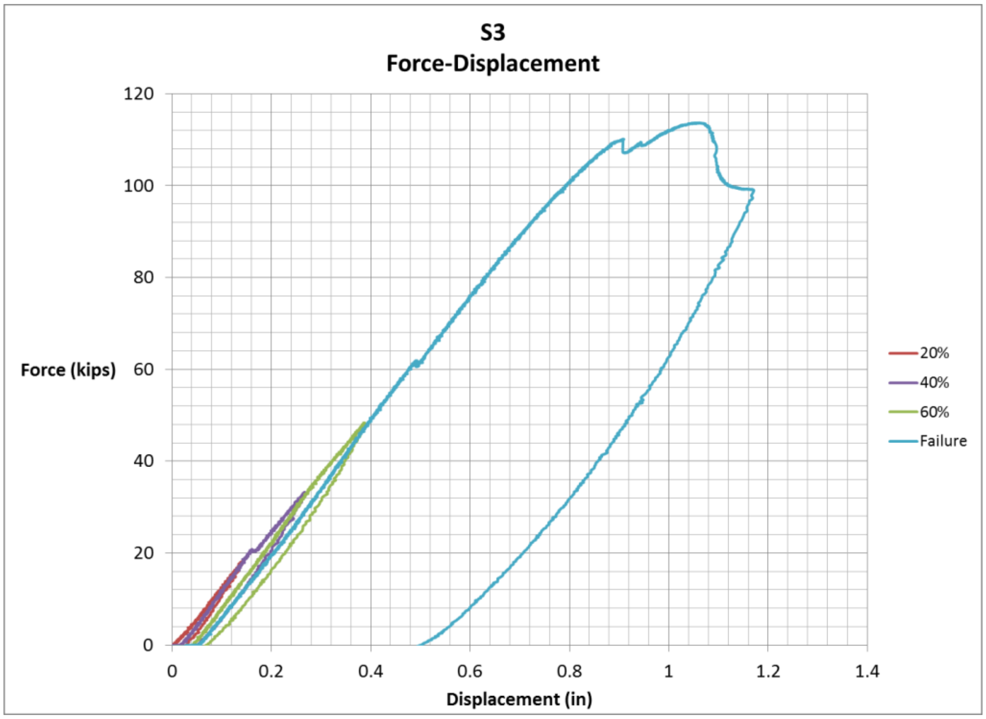


Figure 7.3 Force-displacement curve for S3

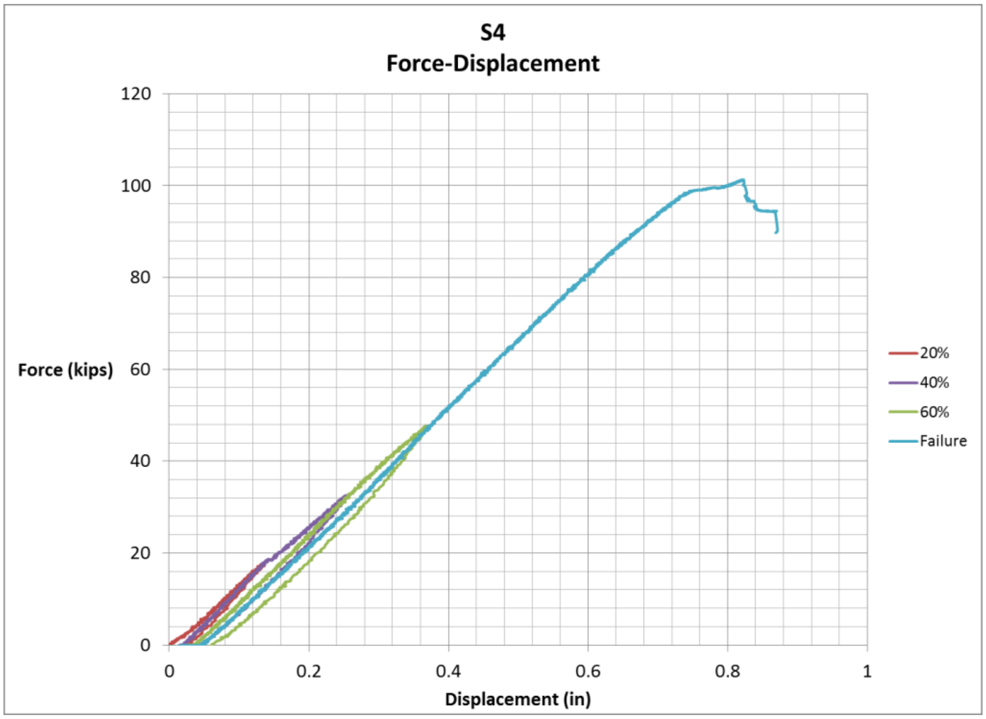


Figure 7.4 Force-displacement curve for S4

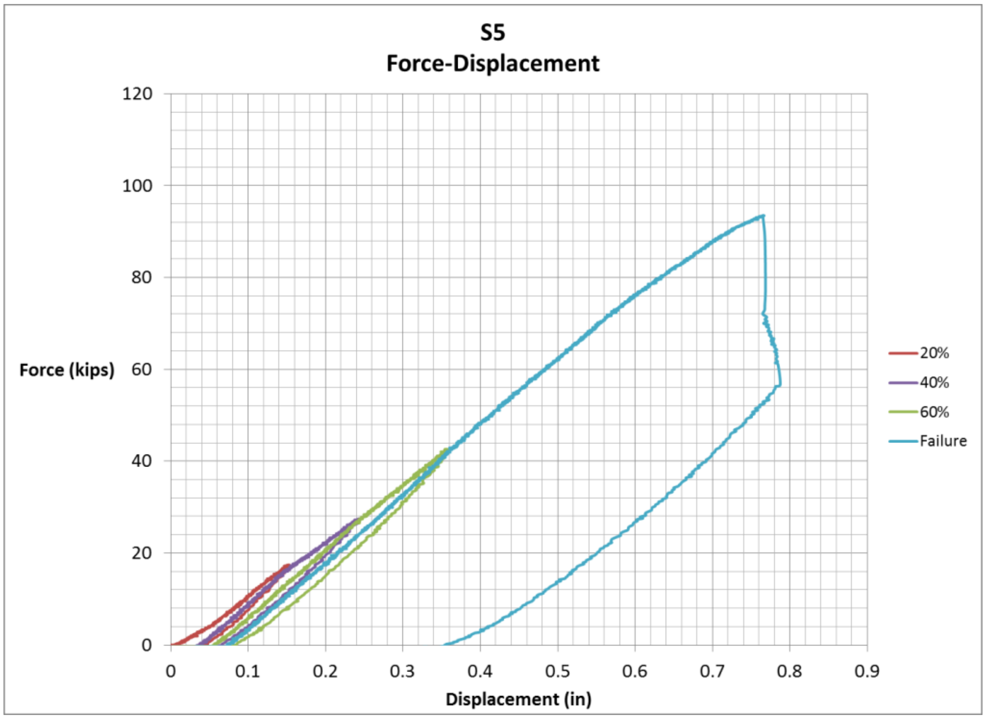


Figure 7.5 Force-displacement curve for S5

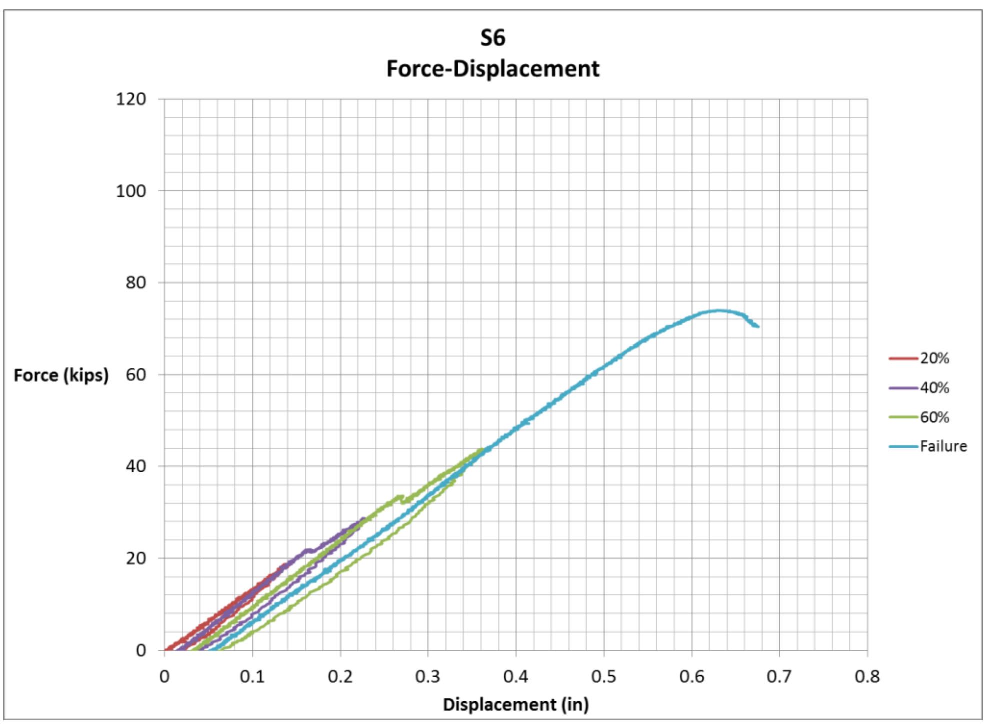


Figure 7.6 Force-displacement curve for S6

7.1.2 Bamboo model comparison

Once the actual capacities of the beams were determined from the test results, the expected failure forces for each bamboo model were compared to the actual forces. The expected and actual failure forces and the resulting percent errors are shown in Table 7.1. Since the three bamboo models do not affect the shear capacity of the beams, and each of the shear controlled beams was expected to fail in shear for both the *Yielding* and *Rupture 2/3* models, there is no difference in the expected forces between these two models. The *Yielding 0.6E* model indicated an expected compression controlled flexural failure which slightly decreased its expected failure forces.

Apart from beam S2, all the shear controlled beams had higher capacities than expected. Beam S2 performed unexpectedly, failing at 78.2 kips (348 kN), which was over 30 kips (133 kN) less than its counterpart, beam S1. It is unclear why beam S2 failed at a much lower capacity than beams S1, S3 and S4. The model predictions had relatively high percent errors, between -11 to -33% of the actual beam capacities, apart from beam S2. This indicates that the shear design for BRC beams is not very accurate and that higher shear capacities can be expected.

Table 7.1 Shear controlled beam force comparison

Beam ID	Actual Failure (kip)	*Yielding*		*Rupture 2/3*		*Yielding 0.6E*	
		Expected Failure (kip)	% Error	Expected Failure (kip)	% Error	Expected Failure (kip)	% Error
S1	110.0	97.9	-11%	97.9	-11%	95.4	-13%
S2	78.2		25%		25%		22%
S3	115.7	77.3	-33%	77.3	-33%	77.3	-33%
S4	103.1		-25%		-25%		-25%
S5	94.9	67.0	-29%	67.0	-29%	67.0	-29%
S6	75.6		-11%		-11%		-11%

7.1.3 BRC & steel comparison

The performance of bamboo and steel as shear reinforcement for concrete beams was compared. The six shear controlled beams were designed with an equivalent area of steel as the provided bamboo reinforcement. The steel reinforced concrete (SRC) beams were designed according to ACI 318-11. The capacities of the BRC shear controlled beams using the *Yielding 0.6E* model are compared to the equivalent SRC shear controlled beams in Table 7.2. The BRC beam capacities are shaded in green while the SRC beam capacities are shaded in purple. The BRC shear controlled beams had capacities ranging from 33 to 70% of the SRC shear controlled beams of equal dimensions and are shaded in light orange.

Table 7.2 Bamboo steel comparison for shear controlled beams

Beam ID	Stirrup spacing, s		BRC *Yielding 0.6E*		Equivalent SRC		BRC:SRC
	(in)	(mm)	(kip)	(kN)	(kip)	(kN)	Capacity
S1	4	102	110.0	489.3	234.1	1041.3	47%
S2	4	102	78.2	347.9	234.1	1041.3	33%
S3	6	152	115.7	514.7	168.1	747.7	69%
S4	6	152	103.1	458.6	168.1	747.7	61%
S5	8	203	94.9	422.1	135.1	601.0	70%
S6	8	203	75.6	336.3	135.1	601.0	56%

7.1.4 BRC & unreinforced beam comparison

The capacity of each shear controlled BRC beam was compared to the capacity of an unreinforced concrete beam of the same dimensions. The capacity of the unreinforced beams was calculated as the load required to cause M_{cr}, the cracking moment. The bamboo stirrups increased the capacity of the unreinforced shear beams by 135 up to 259%

as shown in Table 7.3. The beams with a 6-inch (152-mm) stirrup spacing provided the highest shear capacity and greatest *% strength* increase.

Table 7.3 *% strength* increase between unreinforced and BRC shear beams

Beam ID	Stirrup spacing, s		BRC *Yielding 0.6E*		Unreinforced		BRC vs. Unreinforced
	(in)	(mm)	(kip)	(kN)	(kip)	(kN)	% Strength Increase
S1	4	102	110.0	489.3	34.2	152.3	221%
S2	4	102	78.2	347.9	32.2	143.3	143%
S3	6	152	115.7	514.7	32.2	143.3	259%
S4	6	152	103.1	458.6	32.2	143.3	220%
S5	8	203	94.9	422.1	32.2	143.3	195%
S6	8	203	75.6	336.3	32.2	143.3	135%

7.1.5 Failure analysis

As expected, each of the shear controlled beams failed in shear. Under loading, the first cracks to form were flexure cracks near the mid-span of the beam. These flexure cracks often coincided with a stirrup location. As the beam was loaded further, the flexure cracks continued to widen to large widths of even greater than 0.25 inch (6-mm). The formation of the shear cracks could be seen shortly before they expanded and the beam failed. The front crack patterns for the shear controlled beams can be seen in Figure 7.7.

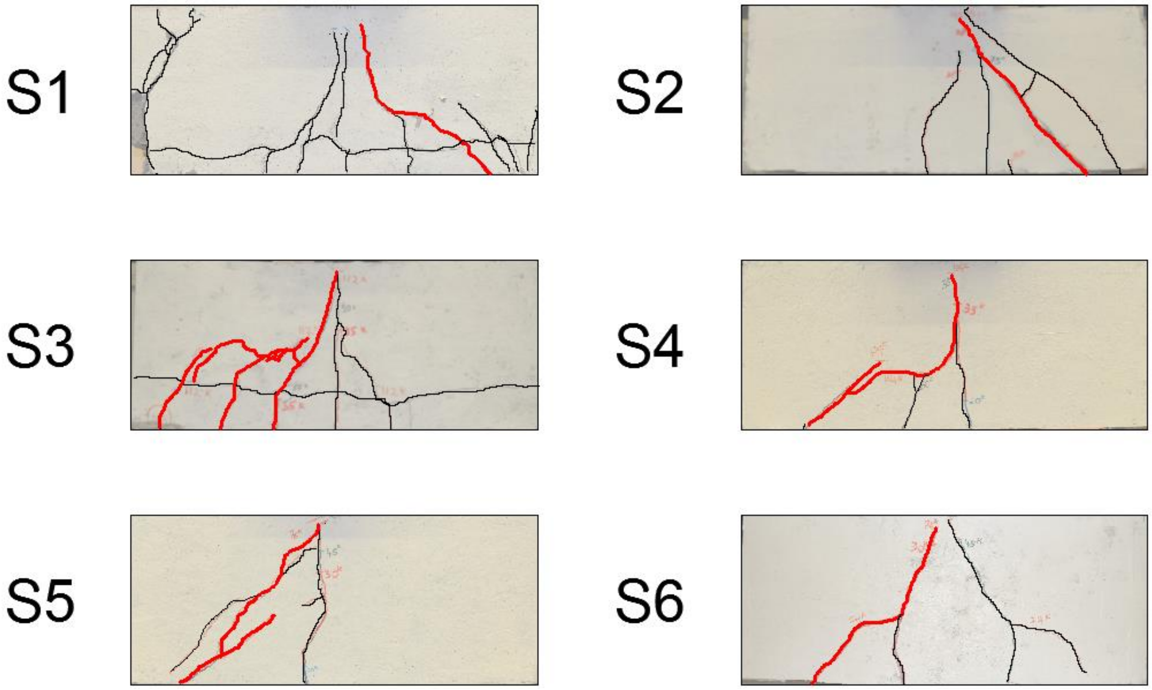


Figure 7.7 Shear beam front crack pattern

After the beams were tested, each one was broken open with a sledge hammer and chisel to reveal the bamboo reinforcement and further investigate the failure modes. Care was taken during this process to ensure that the bamboo was not damaged by the chisel and hammer.

Shear controlled beams S1, S2 and S3 had longitudinal ‘curing cracks’ which formed during the curing stage. Further investigation revealed that these ‘curing cracks’ formed in the beams only when the concrete cover was less than about 1.25 inches (32-mm). It is believed that these cracks formed due to expansion of the longitudinal bamboo reinforcement after absorbing moisture from the wet concrete during the curing phase of the concrete. Although the bamboo reinforcement was waterproofed as described in

Chapter 6, it is quite possible that not all the tightly spaced longitudinal bamboo strips were completely sealed.

The failure of the stirrups occurred both near the bent radius and also in the middle of the stirrup, indicating that there was not a significant strength loss upon bending the stirrups. Figure 7.8 shows a failure in the middle of a bamboo stirrup.

Figure 7.8 Failure in middle of bamboo stirrup of beam S3

Each of the shear controlled beams exhibited bond loss in the bottom layer of the longitudinal reinforcement. Figure 7.9 shows the slippage of the longitudinal reinforcement. After the stirrups failed, the concrete core and longitudinal bamboo reinforcement were no longer confined, and further loading from the actuator caused the slippage near the ends of the longitudinal reinforcement.

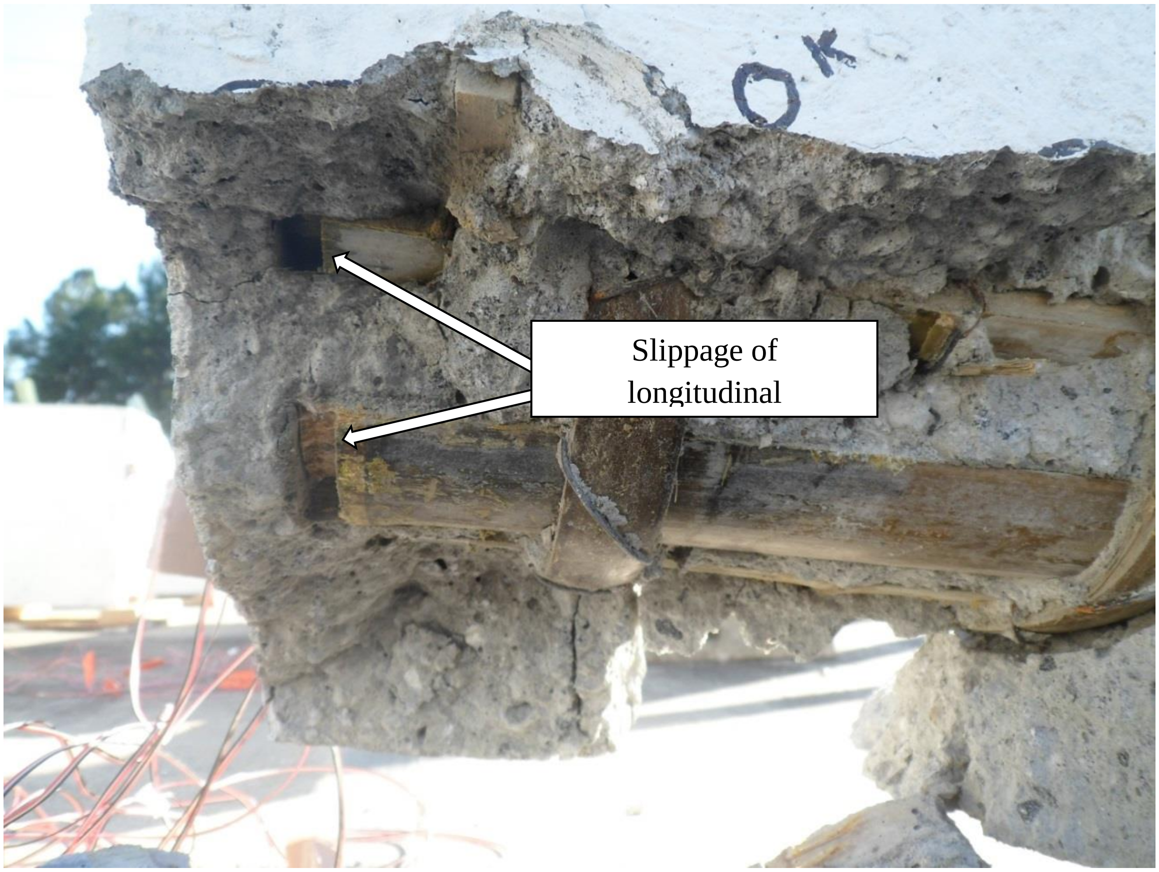


Figure 7.9 Longitudinal reinforcement slippage near beam end of beam S2

7.1.6 Strain analysis

The strain analysis of the shear controlled beams indicated the presence of compression struts running between the load and supports. Stirrups spanning the compression strut experienced compressive strain. Figure 7.10 shows the crack pattern for beam S2. The 75-kip (334 kN) shear crack runs from the right support to the point load at mid-span which is along the same location as the compression strut. The strain in gauge 2 of beam S2 is shown in Figure 7.11. Since no cracks span across gauge 2, there was no tension in the corresponding stirrup and gauge 2 showed a very small compressive strain reading. The maximum compressive strain is very small and only reached a value of about -0.00035. The negative value indicates compressive strain.

In comparison to the strain in gauge 2, the strain in gauge 4 is shown in Figure 7.12. The strain in gauge 4 remained as compressive strain until a load of about 45 kips (200 kN) was reached. During the 40% loading level, the strain changed from compressive to tensile strain as the shear cracks began to form. The tensile strain in gauge reached its limit indicating a possible stirrup failure. The stirrup failure was confirmed upon further investigation and can be seen in Figure 7.13.

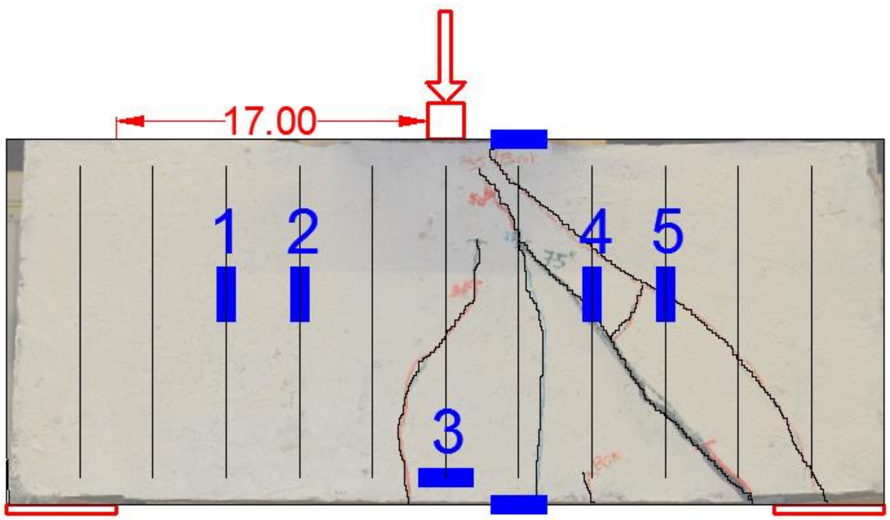


Figure 7.10 Crack pattern and strain gauge location for beam S2
Note: distances are in inches

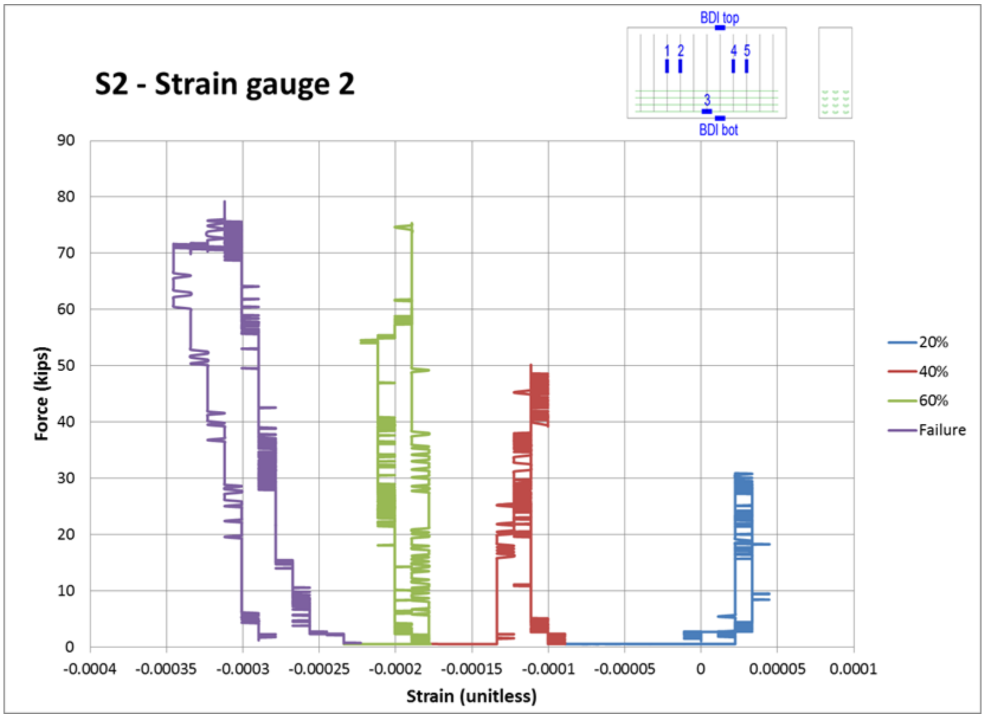


Figure 7.11 Strain gauge 2 on stirrup of beam S2

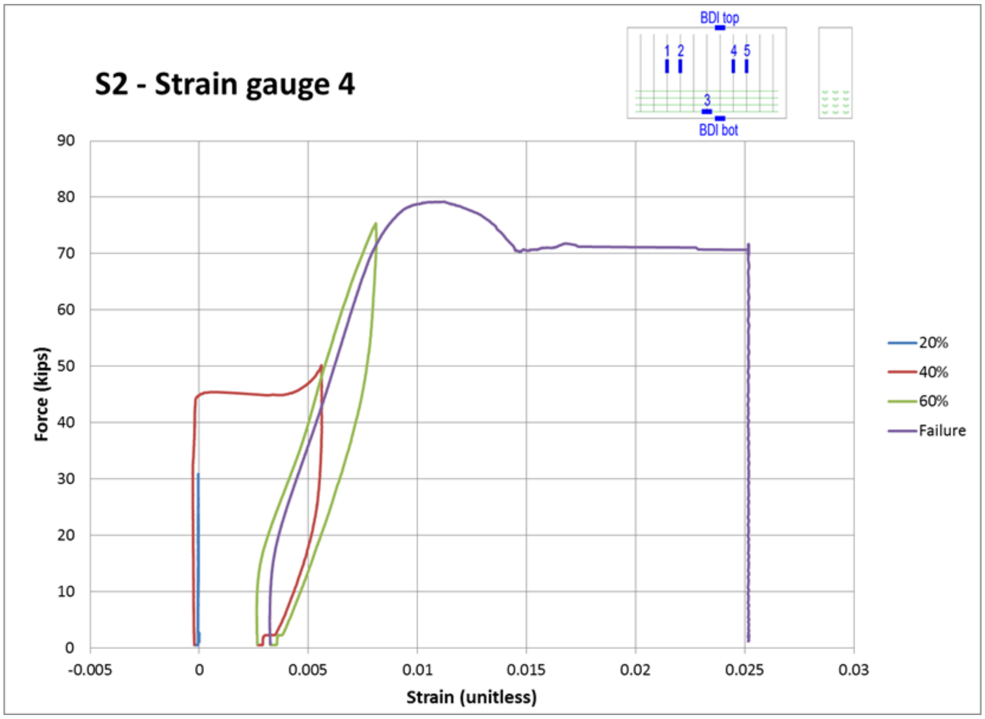


Figure 7.12 Strain gauge 4 on stirrup of beam S2

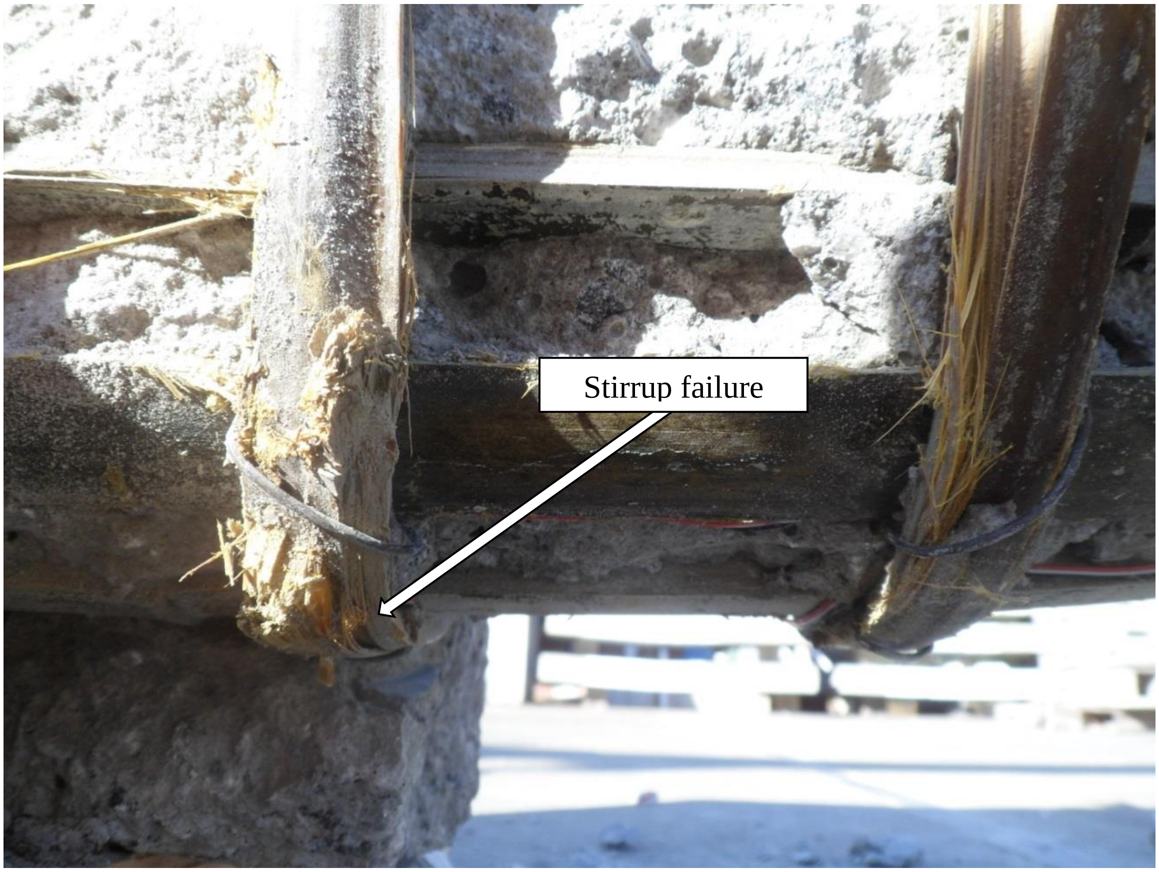


Figure 7.13 Stirrup failure in beam S2

Several of the strain gauges reached their limit during testing. This is indicated by a vertical line on the strain gauge graphs. The strain analysis steps can be found in Appendix F. All the strain graphs for each of the beams except S1 can be found in Appendix G.

7.2 Flexure beams

To test the flexural reinforcement capacity of the bamboo, four beams with varying bamboo reinforcement ratios have been prepared. Based on previous research, reinforcement ratios varying from 1.6 to 3.9 percent have been selected for the four flexure beams. Each beam had a different number of layers of longitudinal reinforcement, including 3, 4, 5 and 6 rows. The total area of longitudinal reinforcement for each beam was The stirrup spacing for the flexure beams remained constant at 6 inches (152-mm). The results from the flexure controlled beam tests including the force-displacement curves, bamboo model comparisons, bamboo and steel comparisons, beam failure analysis and the strain analysis are presented in the following sections.

7.2.1 Force-displacement curves

The force-displacement curves for the flexure controlled beams can be seen in Figure 7.14 through Figure 7.17. The loading levels were targeted at approximately 20, 40, 60% of the expected failure load and then the beams were loaded until failure. All the beams exhibited brittle failure except beam F4, which failed in an overall ductile manner. The failure force-displacement curve of F4, shown in Figure 7.17, reached its initial maximum at around 40 kips (178 kN) before dropping down to 35 kips (156 kN). After its initial drop, the force on beam F4 continued to rise and exceeded its previous peak, reaching about 42 kips (187 kN). Under continued loading, the force decreased in a step-like pattern until a displacement of 1.4 inches (36-mm) was reached. The beam was loaded 0.65 inches (17-mm) past its initial drop in force.

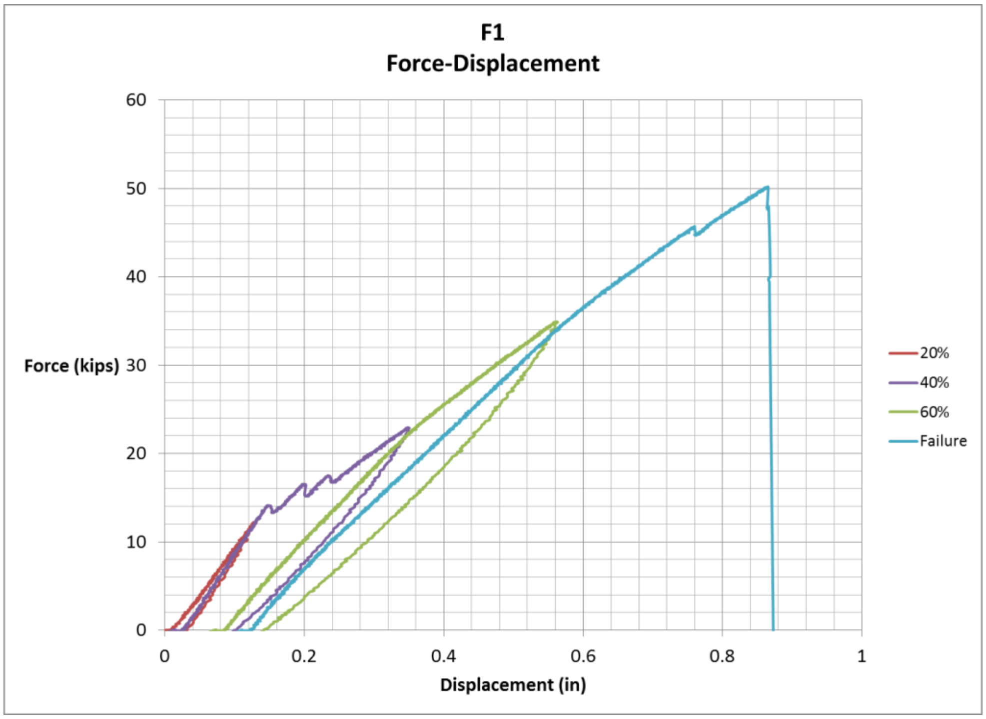


Figure 7.14 Force-displacement curve for F1

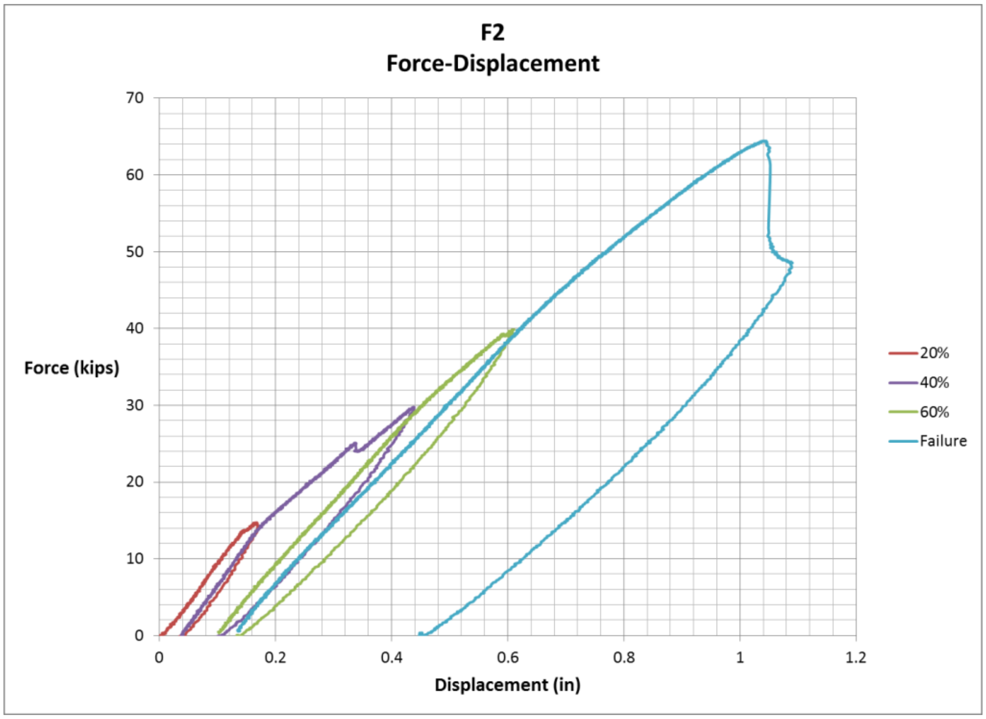


Figure 7.15 Force-displacement curve for F2

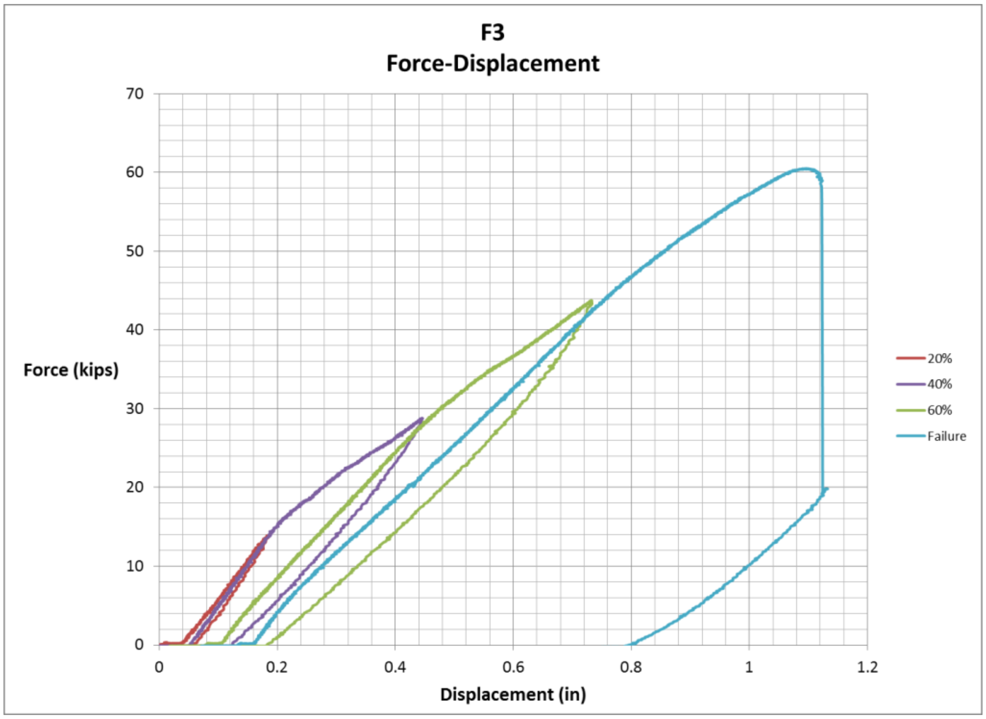


Figure 7.16 Force-displacement curve for F3

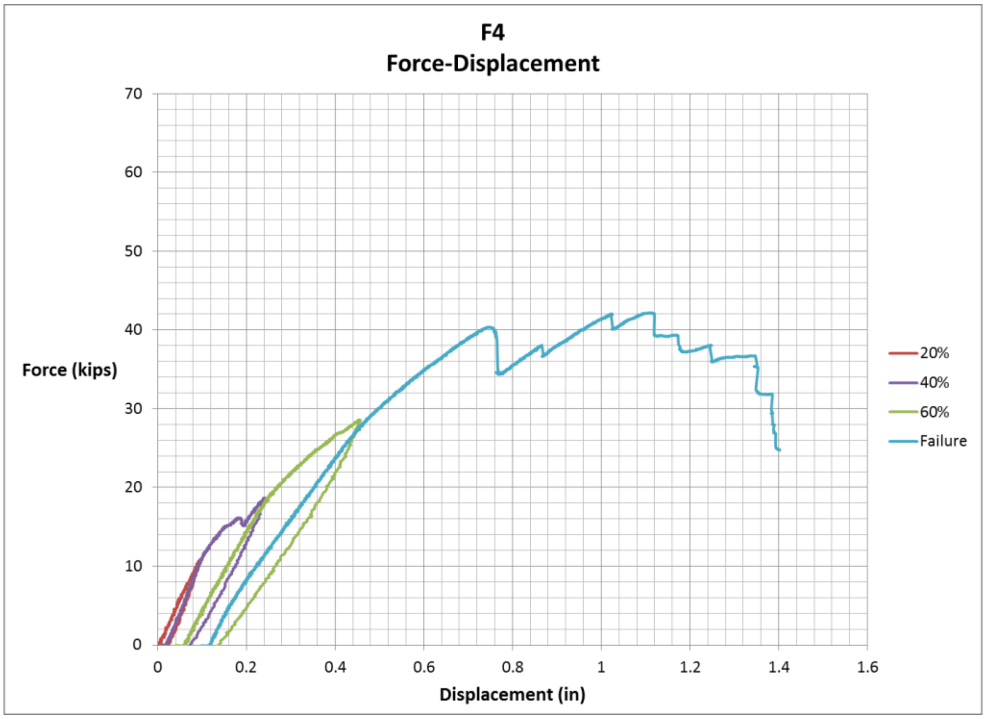


Figure 7.17 Force-displacement curve for F4

7.2.2 Bamboo model comparison

The actual capacities of each flexure controlled beam were compared to the expected capacities determined for each bamboo model. The expected and actual failure forces and the resulting percent errors are shown in Table 7.4. The *Yielding* model overestimated all the flexure controlled beam capacities, with percent errors ranging from 5 to 31%. The *Rupture 2/3* model had percent errors ranging from -11 to 21%. The *Yielding 0.6E* model underestimated all the flexure controlled beams except F1 and had percent errors ranging from -15 to 11%.

The sum of the absolute values of the percent errors for each beam were calculated and used to compare the bamboo models in Table 7.5. The lowest *% error* for each beam was highlighted in red. The sum of the *Yielding 0.6E* percent errors was 34% compared to 40% and 74% for the *Rupture 2/3* and *Yielding* models respectively. The *Yielding 0.6E* model most closely predicted the actual flexure controlled beam capacities since the sum of its percent errors was the lowest of the three models.

Table 7.4 Flexure controlled beam force comparison

Beam ID	Actual Failure (kip)	*Yielding*		*Rupture 2/3*		*Yielding 0.6E*	
		Expected Failure (kip)	% Error	Expected Failure (kip)	% Error	Expected Failure (kip)	% Error
F1	51.3	67.2	31%	45.8	-11%	56.8	11%
F2	64.8	75.9	17%	67.2	4%	61.0	-6%
F3	62.4	65.7	5%	65.7	5%	53.1	-15%
F4	43.9	52.9	21%	52.9	21%	42.9	-2%

Table 7.5 Flexure controlled beam % error comparison

Beam ID	Failure Mode	Bamboo Model		
		Yielding	*Rupture 2/3*	*Yielding 0.6E*
F1	51.3	31%	11%	11%
F2	64.8	17%	4%	6%
F3	62.4	5%	5%	15%
F4	43.9	21%	21%	2%
Sum		74%	40%	34%

7.2.3 BRC & steel comparison

The performance of bamboo and steel as flexural reinforcement for concrete beams was compared. The four flexure controlled beams were designed with an equivalent area of steel reinforcement as the provided bamboo reinforcement. The steel reinforced concrete (SRC) beams were designed according to ACI 318-11. The capacities of the BRC flexure controlled beams using the *Yielding 0.6E* model are compared to the equivalent SRC flexure controlled beams in Table 7.6. The BRC flexure controlled beams had capacities ranging from 29 to 39% of the SRC flexure controlled beams of equal dimensions and area of reinforcement.

Table 7.6 Bamboo steel comparison for flexure controlled beams

Beam ID	Reinforcement ratio	BRC *Yielding 0.6E*		Equivalent SRC		BRC:SRC
	ρ_{bamboo}	(kip)	(kN)	(kip)	(kN)	Capacity
F1	1.6%	51.3	228.2	174.5	776.2	29%
F2	2.4%	64.8	288.2	168.1	747.7	39%
F3	3.0%	62.4	277.6	161.8	719.7	39%
F4	3.9%	43.9	195.3	138.1	614.3	32%

7.2.4 BRC & unreinforced beam comparison

The bamboo reinforcement increased the capacity of the unreinforced flexure beams by 134 up to 242% as shown in Table 7.7. Beam F3 (3.0% reinforcement) had the highest increase in flexural capacity of 242% compared to an unreinforced beam of equal dimensions.

Table 7.7 % Strength increase between unreinforced and BRC flexure beams

Beam ID	Reinforcement ratio ρ_{bamboo}	BRC *Yielding 0.6E* (kip)	BRC *Yielding 0.6E* (kN)	Unreinforced (kip)	Unreinforced (kN)	BRC vs. Unreinforced % Strength Increase
F1	1.6%	51.3	228.2	21.9	97.5	134%
F2	2.4%	64.8	288.2	21.9	97.5	196%
F3	3.0%	62.4	277.6	18.3	81.2	242%
F4	3.9%	43.9	195.3	14.4	64.1	205%

7.2.5 Failure analysis

The flexure controlled beams did not all behave as expected. Beams F1 and F4 failed in flexure as designed, but beams F2 and F3 both failed in shear. This could be because the applied shear to beams F2 and F3 is close to their shear capacities (within 5 kips) and it is understood that shear failure is difficult to predict accurately. Also, the increased shear crack width decreases resistance mechanisms such as aggregate interlock. Stirrups are designed to control the crack width as well as provide doweling action but due to bamboo's low modulus of elasticity it cannot control the crack widths. Bamboo's low stiffness may contribute to the premature shear failures in the flexure controlled beams.

The cracking characteristics of the flexure controlled beams were similar to the shear controlled beams. Under loading, the first cracks to form were flexure cracks near the mid-span of the beam and in-between the point loads. These flexure cracks often coincided with a stirrup location. As the beam was loaded further, the flexure cracks continued to widen to large widths of even greater than 0.25 inch (6-mm). The formation of the shear cracks could be seen shortly before they expanded and the beam failed. The front crack patterns for the flexure controlled beams can be seen in Figure 7.18.

Beam F4 failed in tension controlled bending, but also had evidence of the concrete crushing on top of the beam, indicating slight compression failure. Each model predicted a compression controlled flexural failure for beam F4. The formation of some shear cracks can be seen for beam F4 but the ultimate failure mode was in bending.

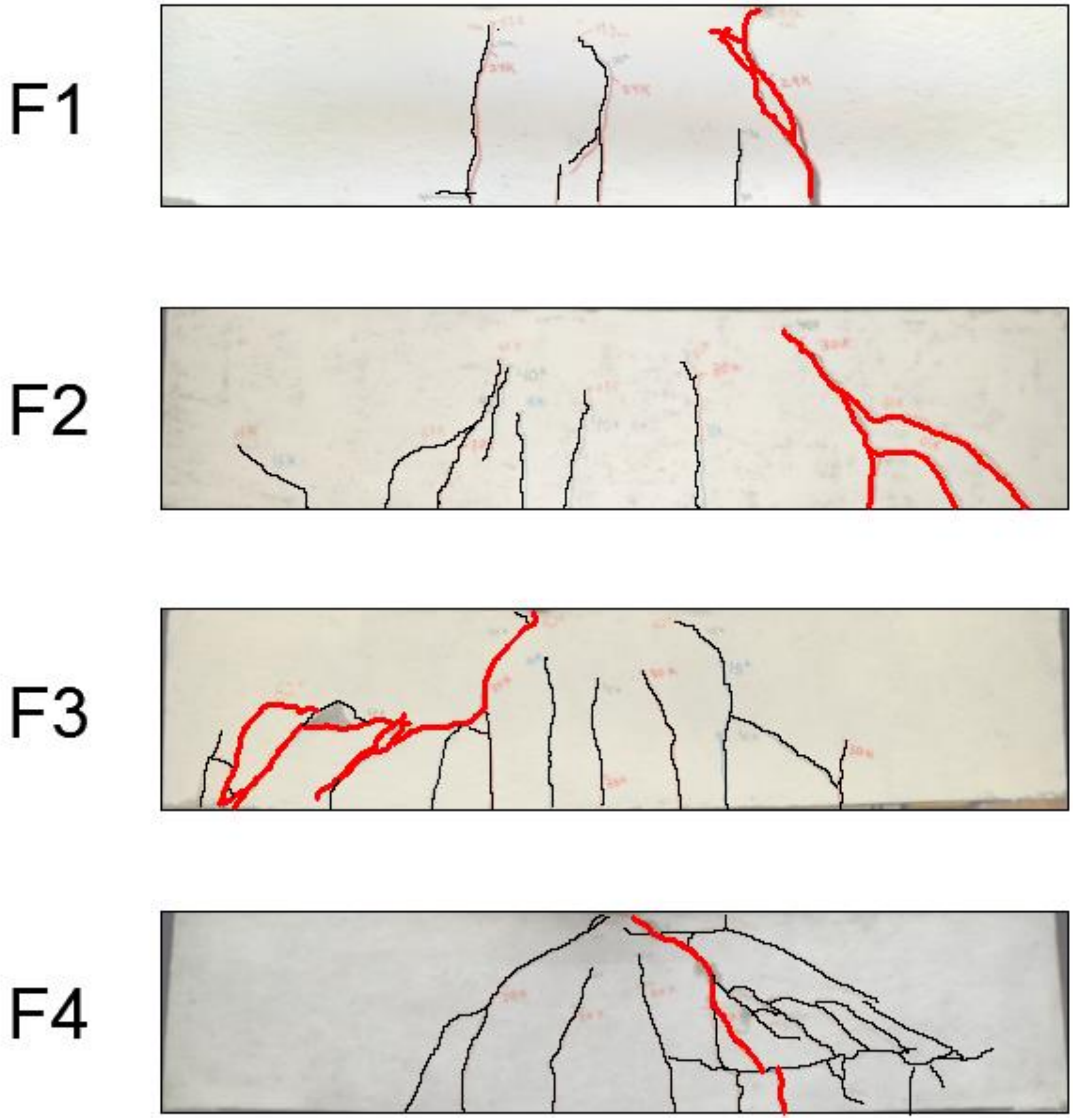


Figure 7.18 Front crack patterns for the flexure controlled beams

After the flexure controlled beams were tested, each one was broken open with a sledge hammer and chisel to reveal the bamboo reinforcement and further investigate the failure modes.

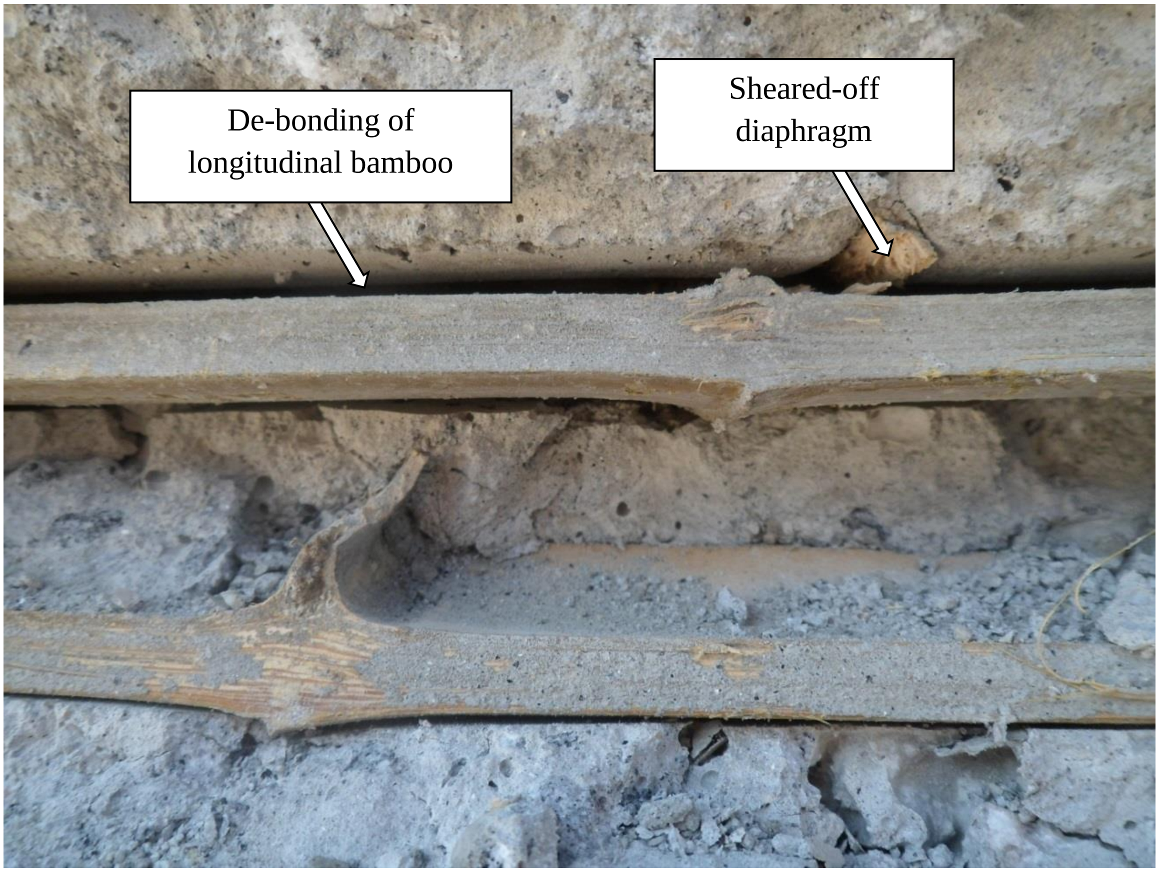


Figure 7.19 Longitudinal bamboo slippage in beam F3

Figure 7.19 shows a diaphragm failure and slippage of the longitudinal reinforcement in beam F3. Diaphragm failure shows that while diaphragms contribute to the mechanical bonding between the bamboo and concrete, they are weak and will shear off if sufficient force is applied. Beam F3 also experienced shear failure. After its stirrups failed, the concrete core and longitudinal bamboo reinforcement were no longer confined and the longitudinal bamboo de-bonded from the concrete, leaving a sizeable gap as shown in Figure 7.19.

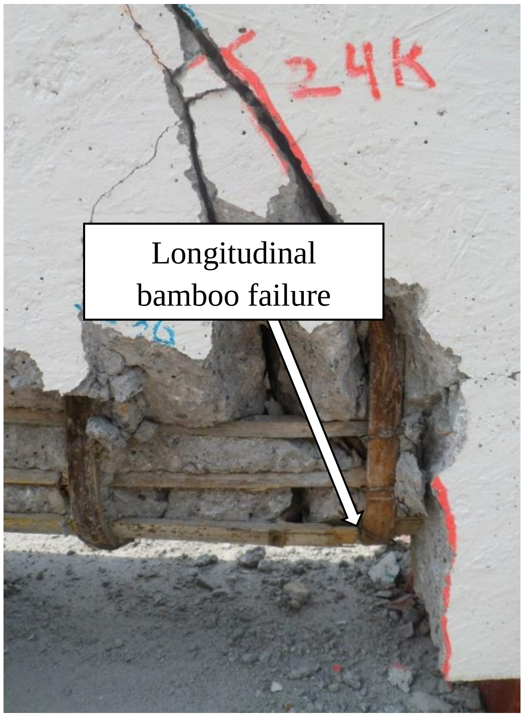


Figure 7.20 Longitudinal bamboo failure in beam F1

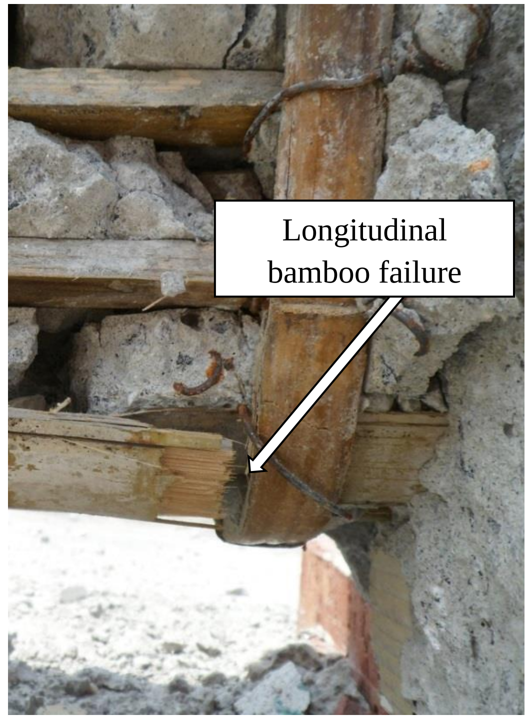


Figure 7.21 Longitudinal bamboo failure in beam F1 (close-up)

Figure 7.20 and Figure 7.21 show the longitudinal bamboo failure in beam F1. This failure occurred right near the large flexure gap which opened up in line with a stirrup. Many of the flexure cracks occurred at stirrup locations. This may be due to the weak plane in the concrete created by the stirrup. The flexure controlled beam F4 also displaced similar failure type in its longitudinal reinforcement. Figure 7.22 shows the three longitudinal bamboo strips in the bottom layer all fail at the same location directly in line with a stirrup.

Figure 7.22 Longitudinal failure at stirrup location in beam F4

7.2.6 Strain analysis

Analyzing the strain in the bottom layer of longitudinal reinforcement in beam F4 indicates that the strain started as tensile strain but turned into compressive strain after the beam failed. This phenomenon can be seen in Figure 7.23. It is believed that the strain turned into compressive strain after failure because the longitudinal bamboo snapped back into place after it failed. As the bamboo returned quickly into its original position, it went into compression.

The flexure controlled beams also contain compression zones running from the supports to the loads and passing through the stirrups. This is evident in Figure 7.24. The strain in the stirrup spanning the compression zone is in compression until the shear cracks begin to open up. Then the strain switches from compressive to tensile strain.

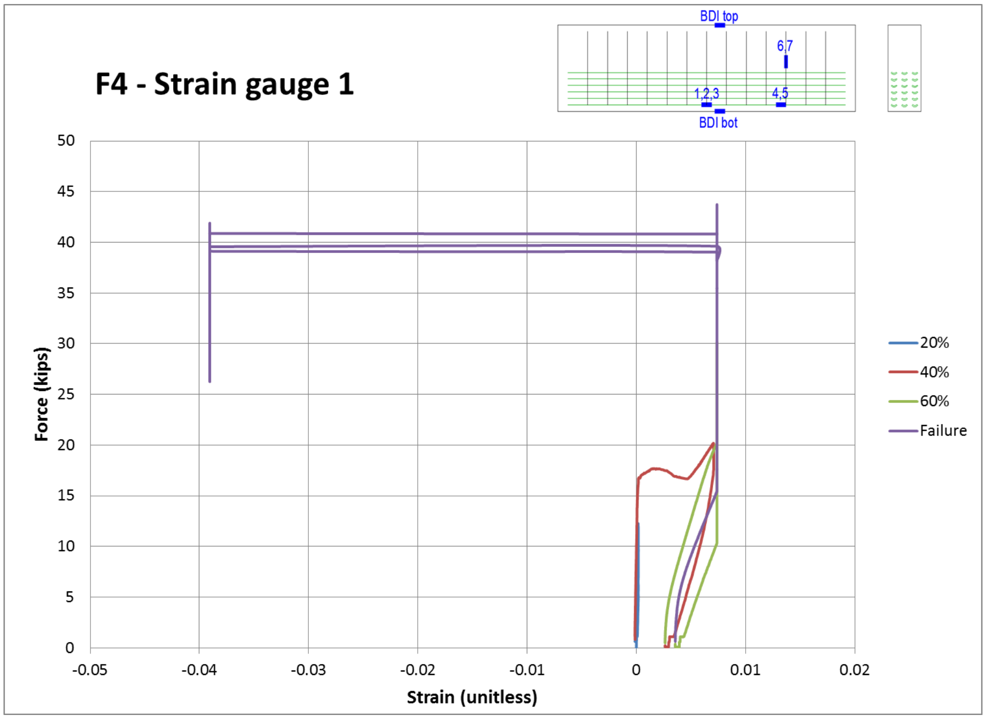


Figure 7.23 Longitudinal bamboo behavior in beam F4

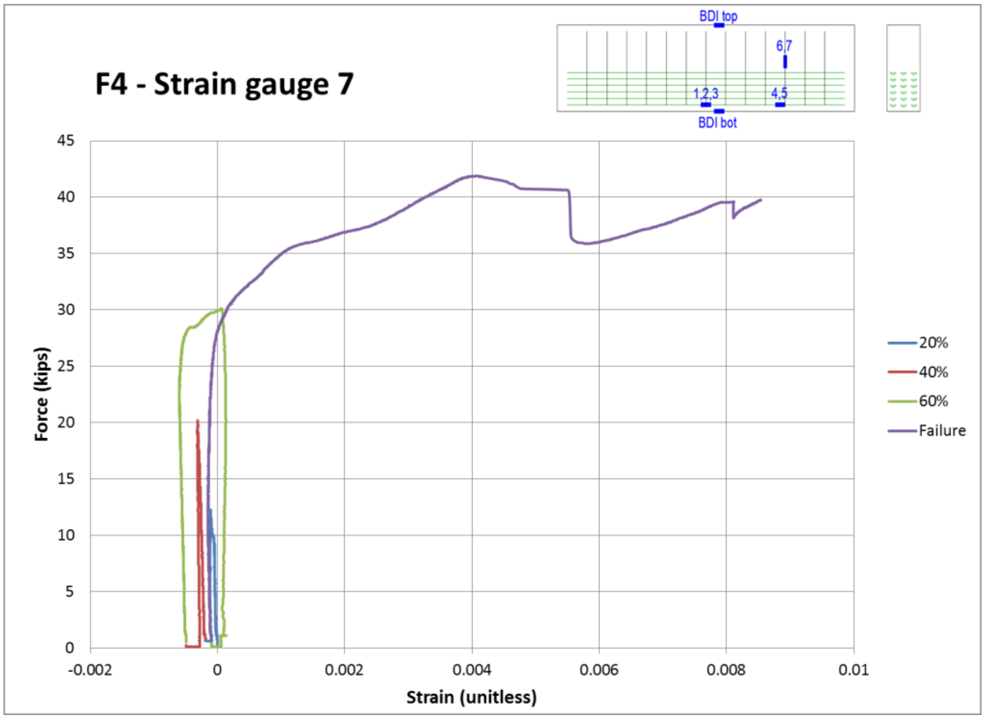


Figure 7.24 Shear stirrup behavior in beam F4

7.3 Flexure lap-spliced beams

To test the splicing capability of longitudinal bamboo reinforcement, three beams were tested using different lengths of lap-splicing for the bottom layer of reinforcement. The lap-splice lengths were chosen based on the results from the component bond strength tests described in Chapter 5. The pullout tests indicated the range of development lengths to be from 15 to 31 inches (38 – 79-cm) with an average of 25 inches (51-cm). Three different lap-splice lengths were chosen to be tested: 12-inch (30-cm), 18-inch (46-cm) and 24-inch (61-cm). Each lap-spliced beam had 4 layers of flexural reinforcement and had stirrups spaced at 6 inches (152-mm). The results from the flexure controlled lap-spliced beam tests including the force-displacement curves, bamboo model comparisons, bamboo and steel comparisons, beam failure analysis and the strain analysis are presented in the following sections.

7.3.1 Force-displacement curves

The force-displacement curves for the flexure controlled lap-splice beams can be seen from Figure 7.25 through Figure 7.27. The loading levels were targeted at approximately 20, 40, 60% of the expected failure load and then the beams were loaded until failure. Beam L1 failed in a very brittle manner while beams L2 and L3 behaved in a somewhat more ductile manner. Beam L2 was loaded about 0.4 inches (10-mm) past its initial drop in force and still retained about half of its max capacity.

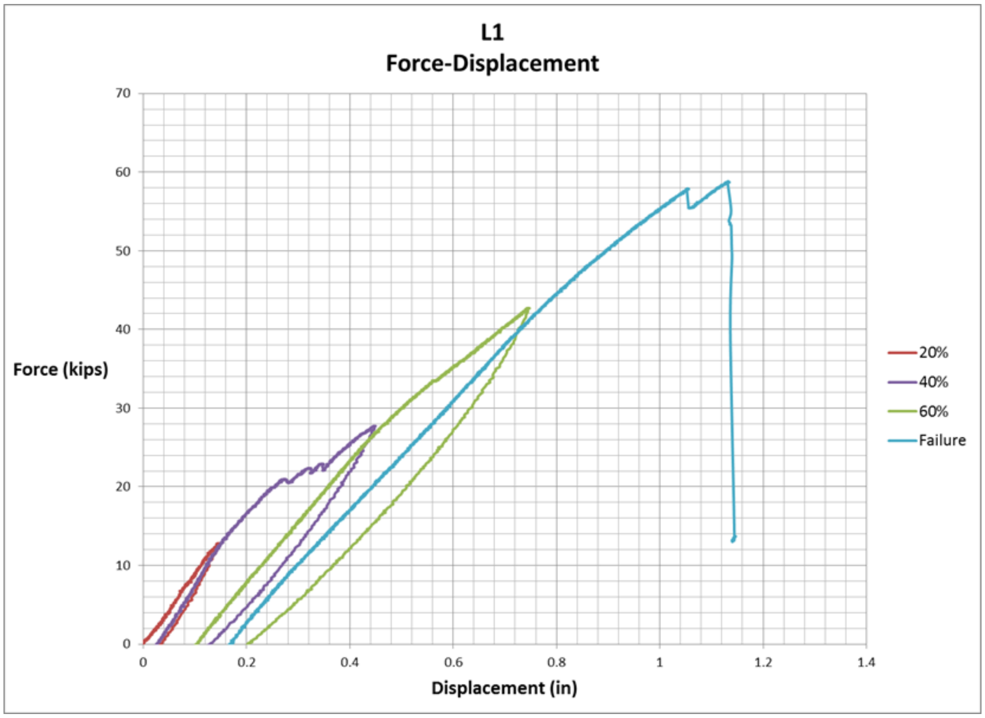


Figure 7.25 Force-displacement curve for L1

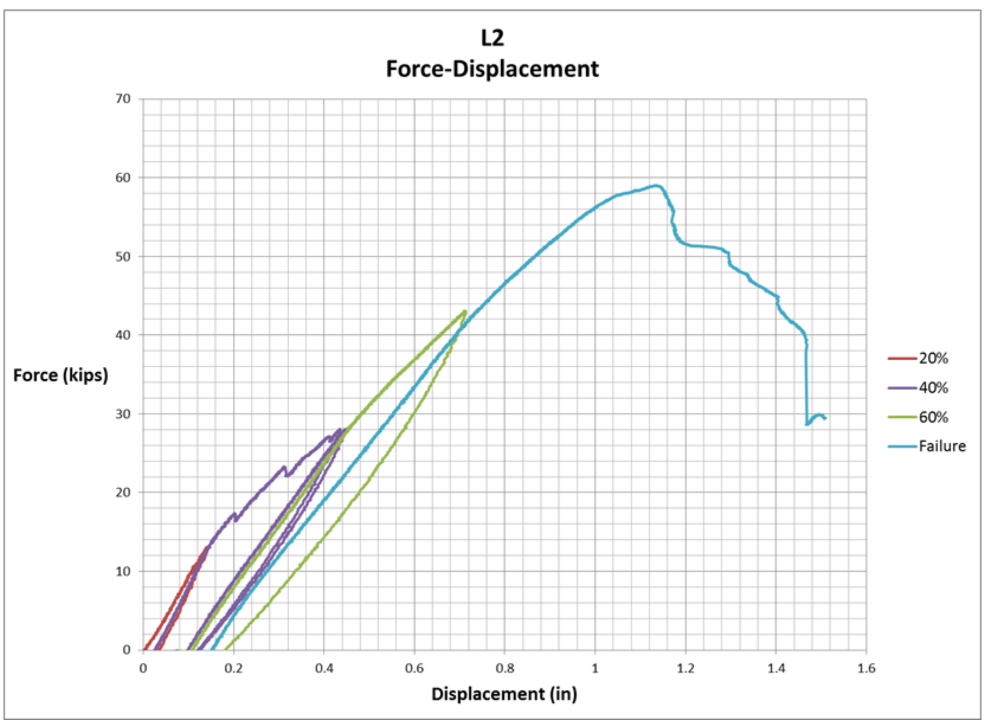


Figure 7.26 Force-displacement curve for L2

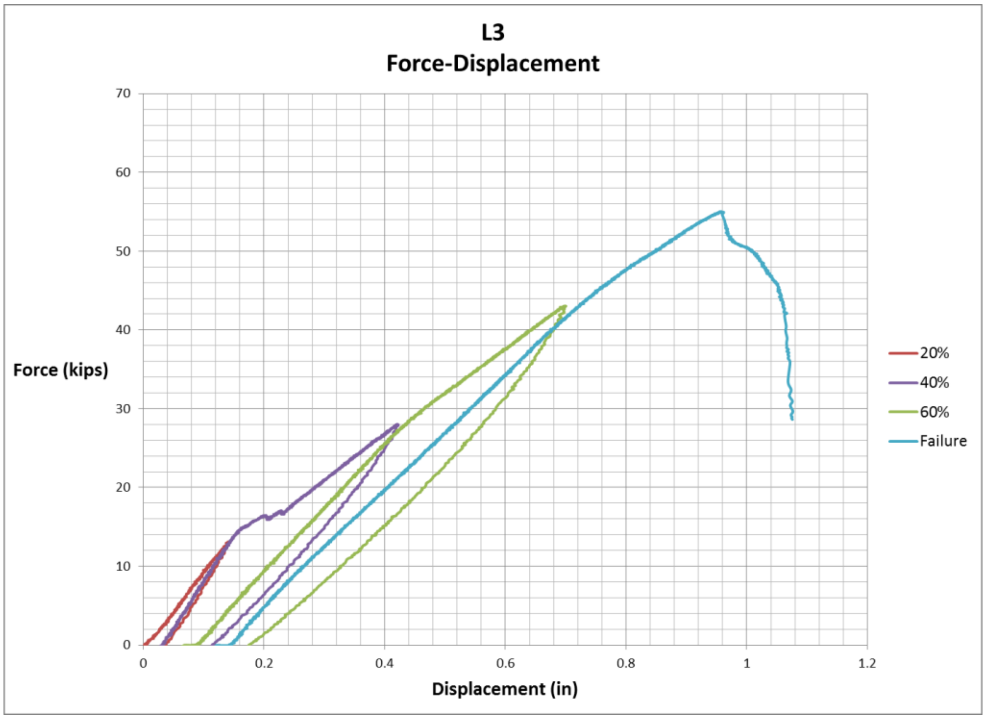


Figure 7.27 Force-displacement curve for L3

7.3.2 Bamboo model comparison

The actual capacities of each flexure controlled lap-splice beam were compared to the expected capacities determined for each bamboo model. The expected and actual failure forces and the resulting percent errors are shown in Table 7.8. Both the *Yielding* and *Rupture 2/3* models overestimated all the flexure controlled lap-spliced beam capacities. The *Yielding* model had percent errors ranging from 24 to 33%. The *Rupture 2/3* model had percent errors ranging from 10 to 17%. The *Yielding 0.6E* model closely estimated the flexure controlled lap-spliced beams and had percent errors ranging from 0 to 7%.

The sum of the absolute values of the percent errors for each beam were calculated and used to compare the bamboo models in Table 7.9. The sum of the *Yielding*

0.6E percent errors was 7% compared to 37% and 81% for the *Rupture 2/3* and *Yielding* models respectively. The *Yielding 0.6E* model most closely predicted the actual flexure controlled lap-spliced beam capacities since the sum of its percent errors was the lowest of the three models.

Table 7.8 Flexure controlled lap-splice beam force comparison

Beam ID	Actual Failure (kip)	*Yielding*		*Rupture 2/3*		*Yielding 0.6E*	
		Expected Failure (kip)	% Error	Expected Failure (kip)	% Error	Expected Failure (kip)	% Error
L1	61.3	75.9	24%	67.2	10%	61.0	0%
L2	61.2	75.9	24%	67.2	10%	61.0	0%
L3	57.2	75.9	33%	67.2	17%	61.0	7%

Table 7.9 Flexure controlled lap-spliced beam % error comparison

Beam ID	Failure Mode	Bamboo Model		
		Yielding	*Rupture 2/3*	*Yielding 0.6E*
L1	61.3	24%	10%	0%
L2	61.2	24%	10%	0%
L3	57.2	33%	17%	7%
Sum		81%	37%	7%

7.3.3 BRC & steel comparison

The performance of bamboo and steel as flexural reinforcement for concrete beams was compared. The three flexure controlled lap-splice beams were designed with an equivalent area of steel reinforcement as the provided bamboo reinforcement. The steel reinforced concrete (SRC) beams were designed according to ACI 318-11. The capacities of the BRC flexure controlled lap-splice beams using the *Yielding 0.6E* model are compared to the equivalent SRC flexure controlled beams in Table 7.10. The BRC flexure controlled beams had capacities ranging from 34 to 36% of the SRC flexure controlled beams of equal dimensions and area of reinforcement.

Table 7.10 Bamboo steel comparison for flexure controlled beams

Beam ID	Lap-splice length		BRC *Yielding 0.6E*		Equivalent SRC		BRC:SRC
	(in)	(mm)	(kip)	(kN)	(kip)	(kN)	Capacity
L1	12	305	61.3	272.7	168.1	747.7	36%
L2	18	457	61.2	272.2	168.1	747.7	36%
L3	24	610	57.2	254.4	168.1	747.7	34%

7.3.4 BRC & unreinforced beam comparison

The bamboo reinforcement increased the capacity of the unreinforced beams by 161 up to 180% as shown in Table 7.11.

Table 7.11 % Strength increase between unreinforced and BRC lap-spliced beams

Beam ID	Lap-splice length		BRC *Yielding 0.6E*		Unreinforced		BRC vs. Unreinforced
	(in)	(mm)	(kip)	(kN)	(kip)	(kN)	% Strength Increase
L1	12	305	61.3	272.7	21.9	97.5	180%
L2	18	457	61.2	272.2	21.9	97.5	179%
L3	24	610	57.2	254.4	21.9	97.5	161%

7.3.5 Failure analysis

The flexure controlled lap-splice beams exhibited both flexure failure and shear failure. Beam L1 (12-inch splice) failed in flexure at a load of 61-kip (271 kN), beam L2 (18-inch splice) failed in shear at a load of 61-kip (271 kN) and beam L3 (24-inch splice) failed in a combination of shear and flexure at a load of 57-kip (254 kN).

The cracking characteristics of the flexure controlled beams were similar to the shear controlled beams. Under loading, the first cracks to form were flexure cracks near the mid-span of the beam and in-between the point loads. These flexure cracks often coincided with a stirrup location. As the beam was loaded further, the flexure cracks continued to widen to large widths of even greater than 0.25 inch (6-mm). The formation of the shear cracks could be seen shortly before they expanded and the beam failed. The front crack patterns for the flexure controlled lap-spliced beams can be seen in Figure 7.28.

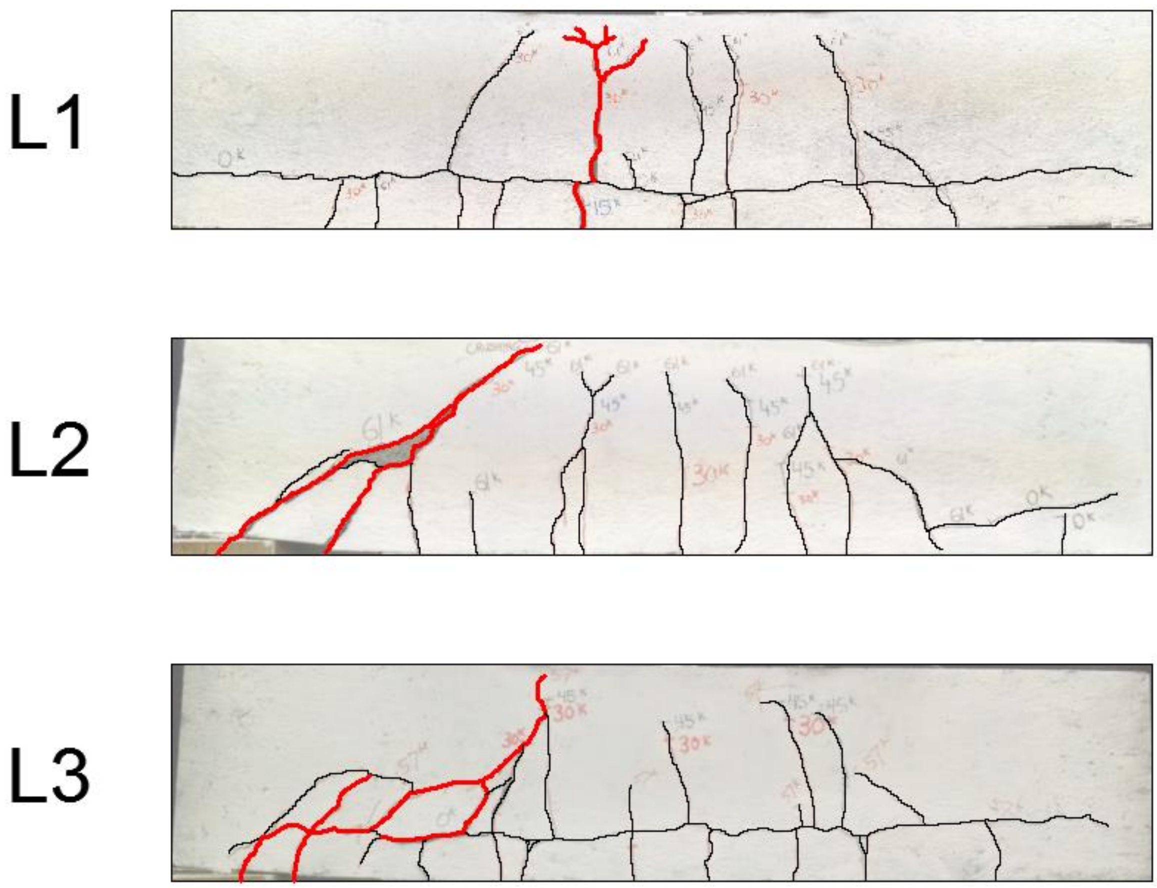


Figure 7.28 Front crack patterns for the lap-splice beams

Beam L1 (12-inch splice) failed in flexure at a force of 61-kip (271 kN). No stirrups failed but three strips of the longitudinal bamboo reinforcement failed. The longitudinal reinforcement failures occurred within the region of constant moment in the second and third layers of reinforcement. One longitudinal strip failed in the front face as shown in Figure 7.29 and two longitudinal strips failed in the back face as shown in Figure 7.30. No failure occurred within the lap-spliced reinforcement, indicating that the lap-spliced reinforcement slipped and did not carry the proportional amount of stress. A close examination of the L1 lap-splice showed that slippage occurred on either end of the splice and the new length of the splice was 11.125 inches (283-mm). This indicates that a 12-inch (305-mm) lap-splice length is insufficient to fully develop the strength in the longitudinal reinforcement.

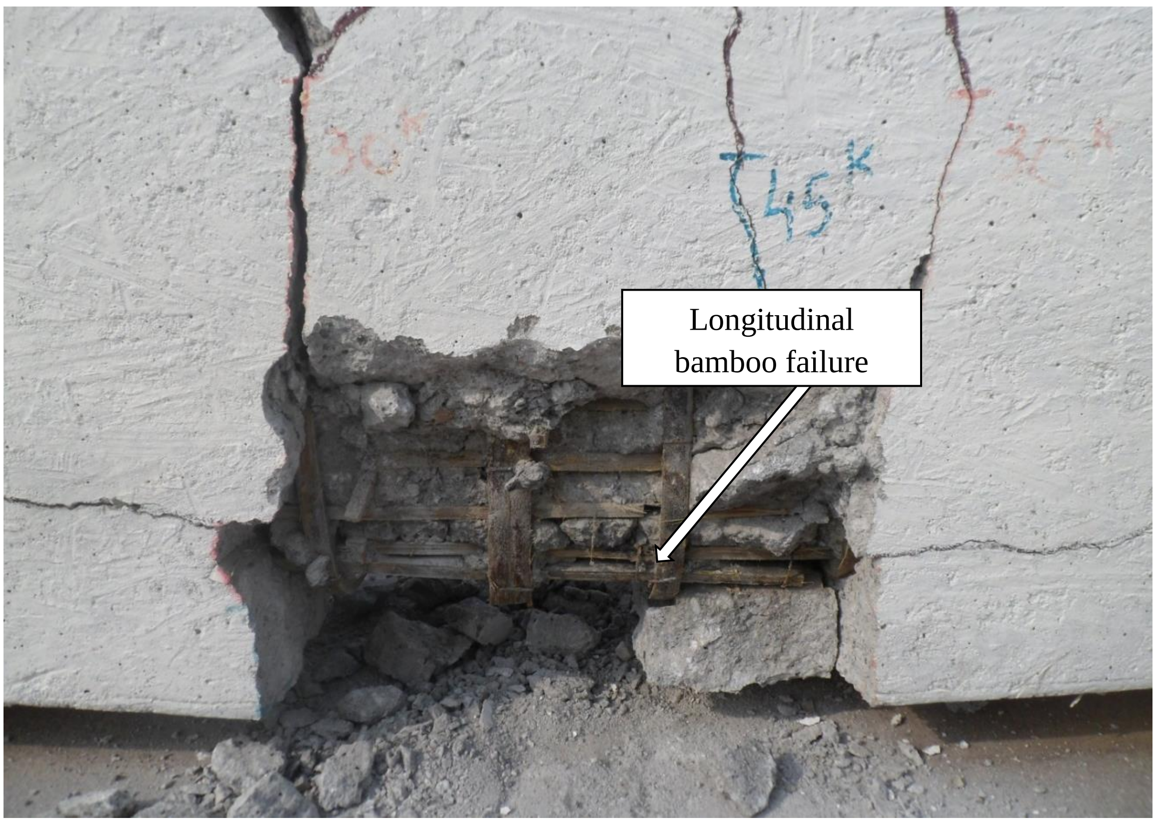


Figure 7.29 Longitudinal strip failure in front face of beam L1

The strain along the lap-splice is shown in Figure 7.31. When the longitudinal bamboo in the second and third layers failed the spliced layer slipped and there was a subsequent reduction in stress and strain within the bamboo.

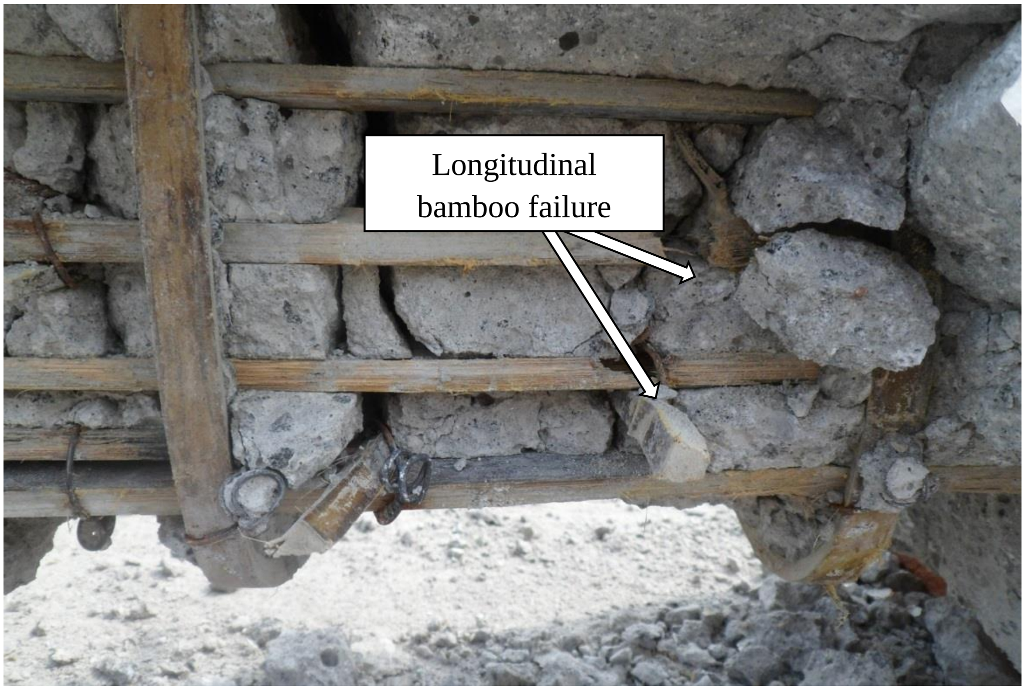


Figure 7.30 Longitudinal strip failures in back face of beam L1

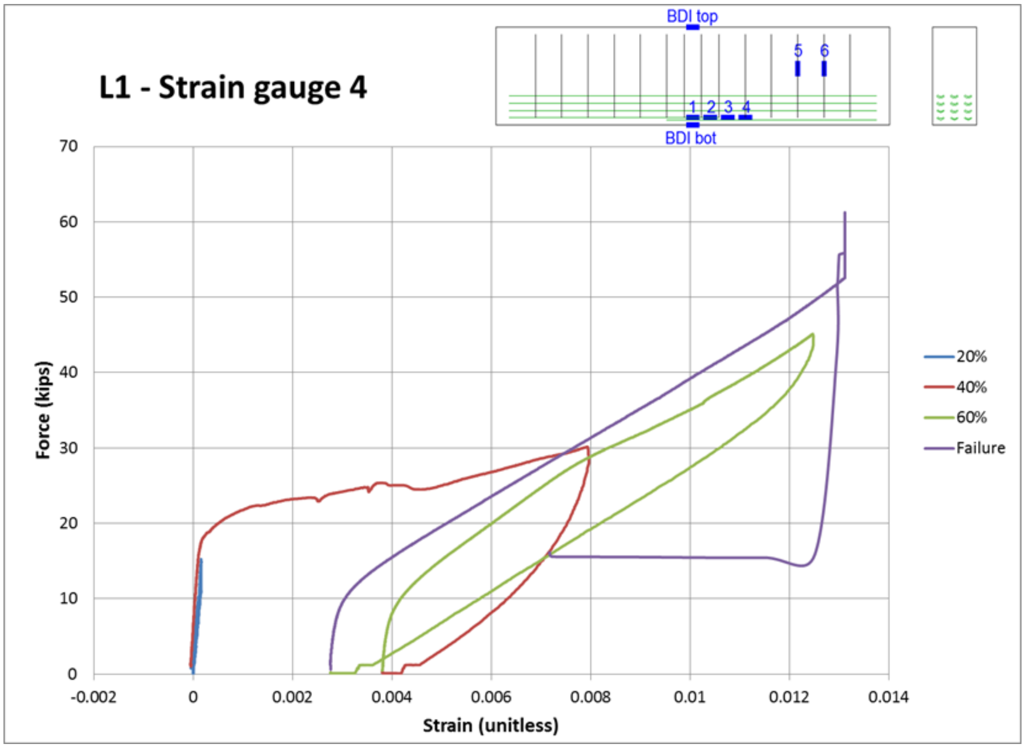


Figure 7.31 Longitudinal bamboo behavior in beam L1

Beam L2 (18-inch splice) failed in shear at a load of 61-kip (271 kN). The shear failure occurred well outside of the lap-spliced region. Two stirrup legs failed in the front face of the beam as shown in Figure 7.32 and the opposite two stirrup legs of the same stirrups failed in the back face of the beam as shown in Figure 7.33. The stirrup near the large shear crack is bent. The bend most likely occurred when the beam failed in shear and the large shear crack opened up, causing the stirrup to bend without snapping. In Figure 7.33, two failures are present in the same stirrup. The author believes that the bottom failure occurred first and then when the large shear crack opened up, the stirrup failed again at the point where it crossed the crack.

As evident in both Figure 7.32 and Figure 7.33, The top layer of longitudinal bamboo reinforcement is completely delaminated from the concrete. This is a result of the shear failure and subsequent loss of confinement in the concrete core. The shear failure in beam L2 indicates that the flexure capacity and corresponding lap-splice length of 18 inches (457-mm) is adequate.

Figure 7.32 Stirrup failure in front face of beam L2

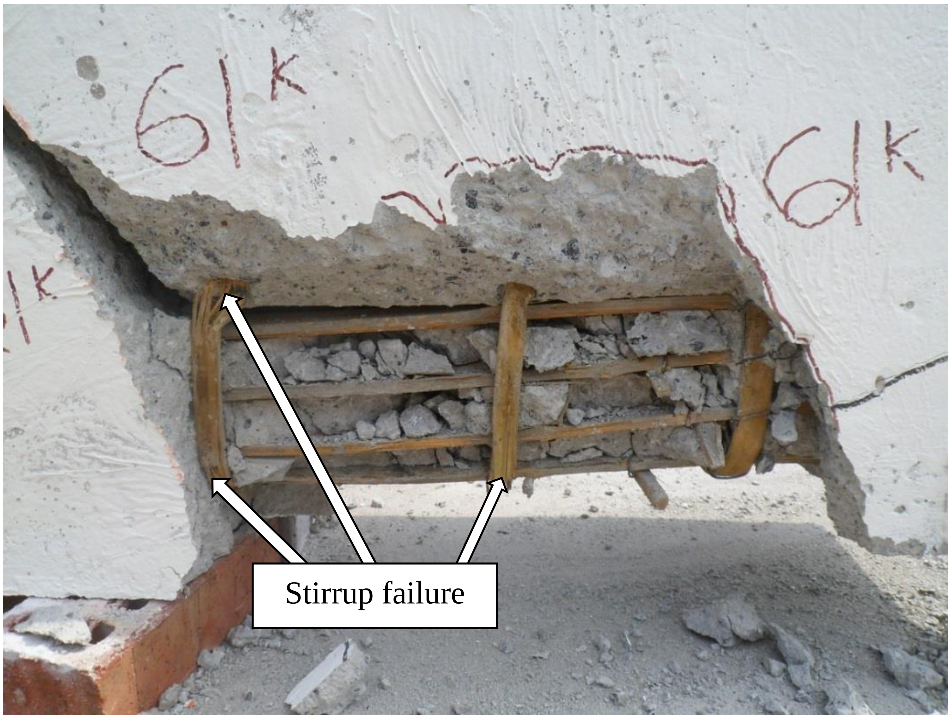


Figure 7.33 Stirrup failure in back face of beam L2

Beam L3 (24-inch splice) failed in shear at a load of 57-kip (254 kN). Beam L3 also had evidence of concrete crushing on the top face of the beam indicating slight compression failure. Three stirrup legs failed in the front face of beam L3 as shown in Figure 7.34 and two stirrup legs failed in the back face as shown in Figure 7.35. Also, one bamboo strip failed in the second layer of longitudinal reinforcement in between the two stirrup legs in the back face as evident in Figure 7.35.

No evidence of lap-splice slippage was evident. Although one of the longitudinal bamboo strips in the second layer failed, the overall beam failure was shear failure which indicates that the flexural capacity of the beam was sufficient and therefore the lap-splice length of 24 inches (610-mm) is adequate.

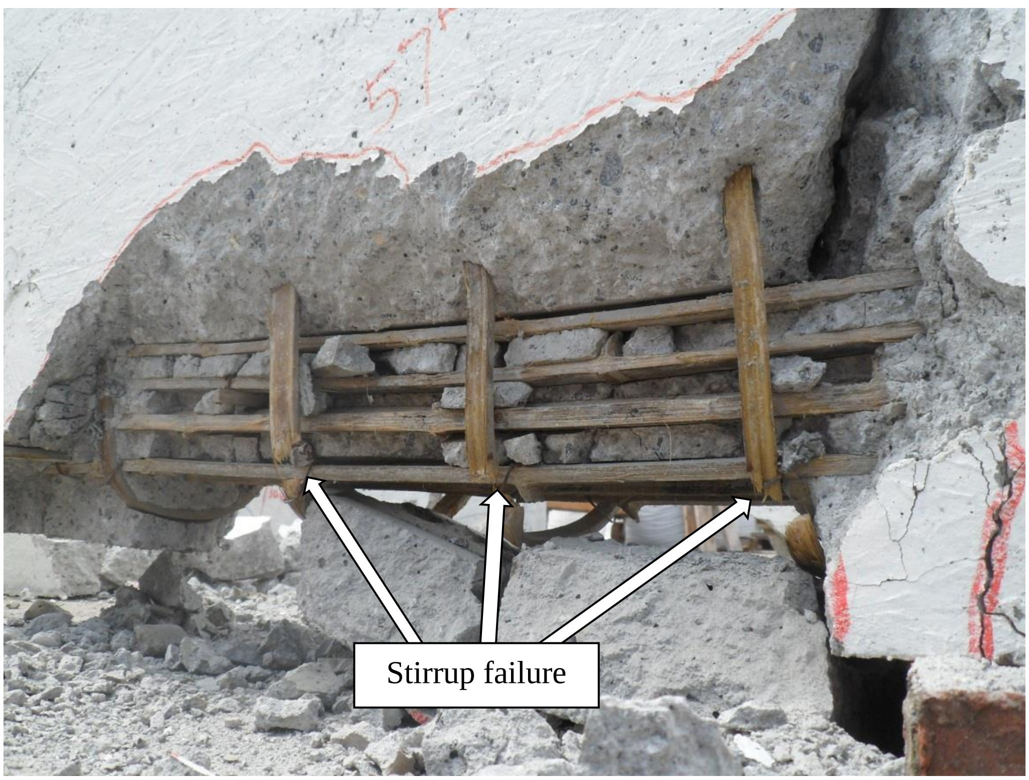


Figure 7.34 Stirrup failure in front face of beam L3

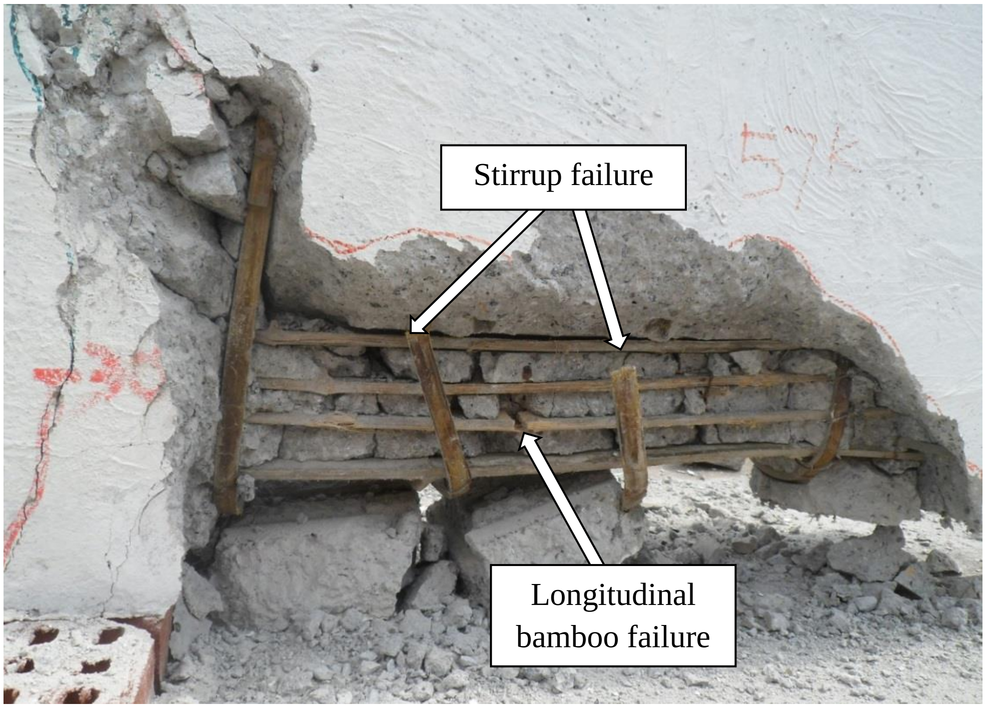


Figure 7.35 Stirrup and longitudinal failure in back face of beam L3

7.4 Beam test results summary

Longitudinal 'curing cracks' along the longitudinal bamboo reinforcement formed in the beams when the concrete cover was less than about 1.25 inches (32-mm). It is believed that these cracks formed due to expansion of the longitudinal bamboo reinforcement after absorbing moisture from the wet concrete during the curing phase of the concrete.

The cracking characteristics of all the beams were similar. Under loading, the first cracks to form were flexure cracks near the mid-span of the beam and in-between the point loads. The flexure cracks formed and expanded rapidly because bamboo is less stiff than the concrete. These flexure cracks often coincided with a stirrup location. As the beam was loaded further, the flexure cracks continued to widen to large widths of even greater than 0.25 inch (6-mm). The formation of the shear cracks could be seen shortly before they expanded and the beam failed.

All of the shear controlled beams failed in shear as expected, but a few of the flexure controlled beams unexpectedly failed in shear. Stirrups are designed to control the crack width as well as provide doweling action but due to bamboo's low modulus of elasticity it cannot control the crack widths. Bamboo's low stiffness may contribute to the premature shear failures in the flexure controlled beams. A closer stirrup spacing than 6 inches (152-mm) is required to prevent premature shear failure.

The lap-spliced beam tests indicated that the 12-inch (304-mm) splice length was insufficient but the beams with lap-splice lengths of 18 and 24 inches (457 and 610-mm) were adequate to develop the flexural reinforcement and prevent slippage.

Of the three models tested, the *Yielding 0.6E* model most closely predicted the BRC beam capacities, especially when slippage of the longitudinal reinforcement is expected. A better model needs to be developed for the shear bamboo reinforcement.

The failure of the stirrups occurred both near the bent radius and also in the middle of the stirrup, indicating that there was not a significant strength loss upon bending the stirrups. The stirrups failed at the location where the shear cracks opened up.

The shear controlled beams performed more closely to the equivalent steel reinforced beams than the flexure controlled beams. Compared to equivalent steel reinforced concrete beams of the same dimensions and area of reinforcement, the shear controlled beams had between 33 and 70% capacity and the flexure controlled beams had between 29 and 39% capacity. The BRC and steel reinforced concrete (SRC) beams are compared in Figure 7.36. The bamboo reinforcement increased the capacity of the unreinforced shear beams by up to 259%. The bamboo reinforcement increased the capacity of the unreinforced flexure beams by up to 242%. The BRC and unreinforced concrete beams are compared in Figure 7.37.

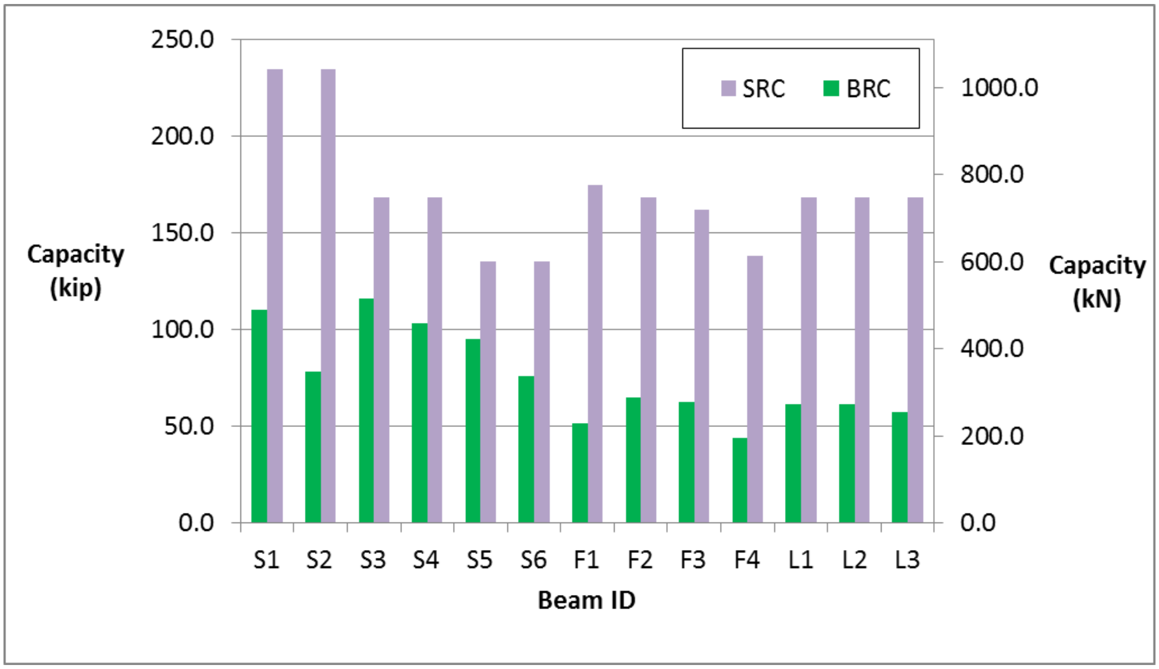


Figure 7.36 BRC and SRC beam capacity comparisons

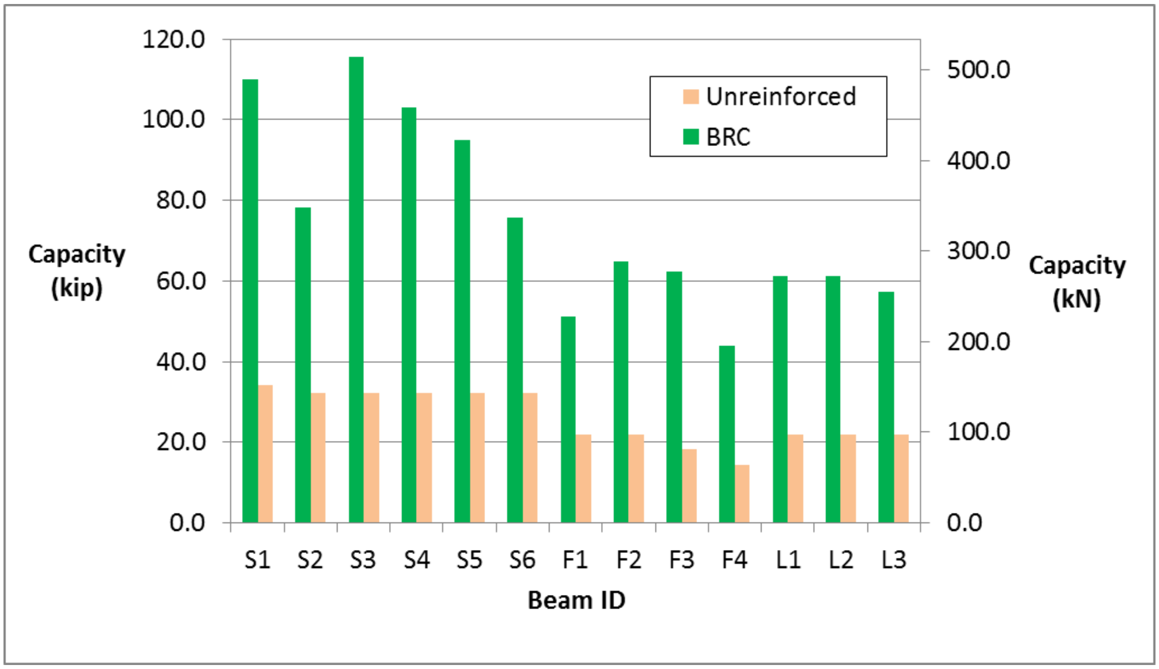


Figure 7.37 BRC and unreinforced beam capacity comparisons

Chapter 8 - CONCLUSION

8.1 Summary

This study evaluated the application of Moso bamboo (*Phyllostachys edulis*) as shear, flexural and flexural lap-spliced reinforcement for concrete through the construction and testing of 13 full-scale beams. To design the bamboo reinforced concrete (BRC) beams, the properties of the bamboo were investigated including the tensile strength, modulus of elasticity and bond strength with concrete.

Once the mechanical properties of the bamboo were determined, 13 full-scale BRC beams were designed and constructed with varying configurations to test the application of bamboo as shear, flexural, and flexural lap-spliced reinforcement. The design of the BRC beams was done by following the Building Code Requirements for Structural Concrete (ACI 318-11) and replacing the properties of rebar with the predetermined properties of bamboo.

Each of the beams were tested in the Wind and Structural Engineering Research (WiSER) Facility at Clemson University in a reaction frame incorporating a 150-kip actuator. A total of 13 BRC beams were tested in bending, and the strain in the concrete and in the bamboo reinforcement at strategic locations was measured. Each beam was loaded until failure and the failure mechanism of each beam was investigated. The test results indicated that bamboo stirrups increased unreinforced concrete beam shear capacities by up to 259%. The flexural bamboo increased beam capacities by up to 242% with the optimal reinforcement ratio of 3.9%. Limitations of the bamboo reinforcement included water absorption and swelling as well as poor bonding capability to the concrete.

The test results show that bamboo is a viable alternative to steel as tensile reinforcement for concrete as it increases the ultimate capacity of the concrete, allows for high deflections and cracks, and provides warning of impending structural failure.

8.2 Conclusion

Bases on the component test results and the full-scale beam test results, the following conclusions can be made.

8.2.1 Component tests

1. All the tensile specimens failed at the nodes in a brittle manner indicating that the nodes are the weakest part of the bamboo culm when loaded in tension. Therefore the nodes in the longitudinal reinforcement should be staggered to prevent planes of weakness.
2. Bamboo's variable properties makes it difficult to specify design provisions for construction with bamboo reinforcement and any designs must have appropriate factors of safety.
3. The presence of diaphragms increased the bond strength between the bamboo and concrete by 15%. However, the strength of the diaphragms was insufficient in many cases as they sheared off from the main bamboo culm as the beams were loaded. Once they sheared off they no longer provided any benefit.
4. Pullout tests indicated the required development length to achieve maximum tensile stress in 1 inch (25.4-mm) wide and 0.3-inch (8-mm) thick bamboo strips

ranges from 15 to 31 inches (38 to 79-cm) with an average length of 25 inches (64-cm). At least 25 inches (64-cm) is recommended for lap-splicing bamboo.

8.2.2 Beam tests

5. Longitudinal ‘curing cracks’ formed parallel to the longitudinal reinforcement in beams with concrete clear covers of less than 1.25 inches (32-mm). It is believed that these cracks formed due to expansion of the longitudinal bamboo reinforcement after absorbing moisture from the wet concrete during the curing phase of the concrete. A cover of at least 1.5 inches (38-mm) is recommended to prevent ‘curing cracks’ from forming.
6. Bamboo stirrups greatly increased the ultimate shear capacity of the concrete beams by up to 259% compared to an unreinforced concrete beam of equal dimensions.
7. Bamboo’s low stiffness may contribute to the premature shear failures in the flexure controlled beams. A stirrup spacing of less than 6 inches (152-mm) is recommended to prevent premature shear failure.
8. The optimum percentage of flexural reinforcement was found to be 3.9% as this produced a ductile failure in beam F4. Having many strips of longitudinal reinforcement allowed the stresses to be transferred to the remaining strips when the strips failed. The flexure beam with 3.0% of flexure reinforcement had the highest increase in capacity over an unreinforced beam of equal dimensions, but failed in shear. It is therefore necessary to have a closer stirrup spacing to fully develop the flexural capacity of each beam and prevent premature shear failure.

9. The longitudinal bamboo reinforcement increased the ultimate capacities of unreinforced beams of equal dimension by 134 to 242%.
10. The 18 and 24-inch (46 and 61-cm) lap-splice lengths were sufficient to develop the spliced flexural reinforcement and the beams subsequently failed in shear. The 12-inch (31-cm) lap-spliced beam failed in flexure after the lap-splice slipped and the stresses were transferred to the other layers of reinforcement. This indicates that the 12-inch (31-cm) lap-splice is insufficient.
11. The closed-stirrup design was easy to manufacture through the steaming procedure and made the bamboo reinforcement cages easy to assemble.
12. An efficient and practical method for constructing bamboo cages for reinforcing concrete in the field was determined and explained in Chapter 6. The production of bamboo reinforcement is a labor-intensive process with very low material costs. This is ideal in many developing regions of the world where labor is plentiful and inexpensive.

Bases on the these conclusions, the author believes that bamboo is a viable alternative to steel for reinforcing concrete as it increases the ultimate capacity of the concrete, allows for high deflections and cracks, and provides warning of impending structural failure. However, more research is required before bamboo reinforcement can be recommended to use in concrete construction in the field.

8.3 Recommendations

The following studies are recommended by the author for further investigation of bamboo reinforced concrete.

1. A closer stirrup spacing should be tested, perhaps as low as 3 inches (76-mm). A closer spacing than 3 inches (76-mm) may be impractical for construction of the bamboo cages.
2. Smaller beam dimensions should be tested since the concrete members found in structures in the developing world generally have smaller dimensions.
3. Higher *a/d* ratios should be tested to further investigate the flexural failure mode of the BRC beams.
4. Smaller bamboo cross-sectional areas should be tested as reinforcement which may help to solve the bonding issues. Bamboo fiber reinforced concrete should also be investigated.
5. Variations in cage construction should be investigated, including orienting the longitudinal bamboo strips on their sides, with the thin ends facing up and down, so more strips could be included in each layer.
6. Quality control measures, such as non-destructive visual grading and machine grading, should be established to limit the variability of the bamboo.
7. A beam-column joint should be tested under cyclic loading to investigate bamboo's ability to develop plastic hinges in the beam to determine if bamboo can be used to reinforce concrete under earthquake loads.

8. A full-scale BRC house could be tested on a shake table to investigate the performance of bamboo reinforcement in earthquake design.
9. Other methods to increase the bond strength between the bamboo and concrete should be investigated.
10. A long-term study further investigating the durability of bamboo in the high alkali environment of concrete should be performed.
11. Bamboo has a lower stiffness than concrete. Solutions to this problem should be investigated such as increasing the stiffness of bamboo or reducing the stiffness of concrete by using a rubber concrete which substitutes recycled rubber in place of some aggregate. This could also increase the sustainability of the BRC technology.
12. Bamboo fiber composite should be investigated as a reinforcement for concrete. Advantages of the engineered bamboo composite over natural bamboo include the elimination of node defects, controlled mechanical properties with lower variability, water resistance, and the ability to shape the composite to increase mechanical bonding to the concrete.

APPENDICES

Appendix A – Tensile strength test comprehensive data

Date	Sample ID	Testing Location	Location	Width (in)	Thickness (in)	Area (in^2)
8/9	T_1	WiSER	Node L	0.223	0.193	0.043
			Middle	0.245	0.170	0.042
			Node R	0.220	0.172	0.038
	T_2	WiSER	Node L	0.205	0.200	0.041
			Middle	0.180	0.225	0.041
			Node R	0.220	0.190	0.042
9/3	T_3	WiSER	Node L	0.320	0.245	0.078
			Middle	0.310	0.180	0.056
			Node R	0.290	0.265	0.077
9/16	T_4	WiSER	Node L	0.260	0.270	0.070
			Middle	0.245	0.195	0.048
			Node R	0.280	0.270	0.076
5/17	T_5	WiSER	Node L	0.251	0.279	0.070
			Middle	0.265	0.263	0.070
			Node R	0.266	0.260	0.069
9/16	T_6	WiSER	Node L	0.260	0.280	0.073
			Middle	0.230	0.220	0.051
			Node R	0.270	0.290	0.078
	T_7	WiSER	Node L	0.295	0.340	0.100
			Middle	0.275	0.250	0.069
			Node R	0.245	0.295	0.072
	T_8	WiSER	Node L	0.260	0.310	0.081
			Middle	0.275	0.245	0.067
			Node R	0.275	0.280	0.077
	T_9	WiSER	Node L	0.230	0.250	0.058
			Middle	0.240	0.230	0.055
			Node R	0.220	0.290	0.064
	T_10	WiSER	Node L	0.270	0.285	0.077
			Middle	0.250	0.230	0.058
			Node R	0.250	0.280	0.070
	T_11	WiSER	Node L	0.207	0.332	0.069
			Middle	0.220	0.235	0.052
			Node R	0.174	0.230	0.040
	T_12	WiSER	Node L	0.210	0.170	0.036
			Middle	0.200	0.185	0.037
			Node R	0.210	0.190	0.040

	T_13	WiSER	Node L	0.320	0.210	0.067
			Middle	0.350	0.210	0.074
			Node R	0.340	0.190	0.065
	T_14	WiSER	Node L	0.280	0.270	0.076
			Middle	0.290	0.230	0.067
			Node R	0.290	0.320	0.093
	T_15	WiSER	Node L	0.120	0.230	0.028
			Middle	0.180	0.295	0.053
			Node R	0.150	0.260	0.039
	T_16	WiSER	Node L	0.265	0.265	0.070
			Middle	0.325	0.220	0.072
			Node R	0.345	0.285	0.098
	T_17	WiSER	Node L	0.240	0.260	0.062
			Middle	0.265	0.235	0.062
			Node R	0.225	0.305	0.069
	T_18	WiSER	Node L	0.300	0.275	0.083
			Middle	0.250	0.230	0.058
			Node R	0.260	0.260	0.068
	T_19	WiSER	Node L	0.205	0.250	0.051
			Middle	0.220	0.230	0.051
			Node R	0.205	0.260	0.053
5/17	T_20	WiSER	Node L	0.257	0.277	0.071
			Middle	0.215	0.233	0.050
			Node R	0.233	0.300	0.070
	T_21	WiSER	Node L	0.241	0.263	0.063
			Middle	0.211	0.265	0.056
			Node R	0.253	0.268	0.068
	T_22	WiSER	Node L	0.282	0.244	0.069
			Middle	0.219	0.265	0.058
			Node R	0.264	0.284	0.075
	T_23	WiSER	Node L	0.225	0.210	0.047
			Middle	0.235	0.220	0.052
			Node R	0.200	0.220	0.044
	T_24	WiSER	Node L	0.283	0.258	0.073
			Middle	0.234	0.261	0.061
			Node R	0.295	0.269	0.079
	T_25	WiSER	Node L	0.242	0.266	0.064
			Middle	0.275	0.269	0.074
			Node R	0.310	0.275	0.085
	T_26	WiSER	Node L	0.255	0.293	0.075

		Middle	0.243	0.240	0.058
		Node R	0.225	0.275	0.062
T_27	WiSER	Node L	0.249	0.269	0.067
		Middle	0.272	0.220	0.060
		Node R	0.255	0.225	0.057
T_28	WiSER	Node L	0.246	0.250	0.062
		Middle	0.235	0.250	0.059
		Node R	0.268	0.250	0.067
T_29	WiSER	Node L	0.231	0.265	0.061
		Middle	0.254	0.260	0.066
		Node R	0.250	0.285	0.071
T_30	WiSER	Node L	0.271	0.247	0.067
		Middle	0.298	0.232	0.069
		Node R	0.265	0.270	0.072

Sample ID	Length between Grips (in)	Loading Rate (in/min)	Max Load (kip)	Displacement (in)
T_1	11.500	0.050	0.469	0.1187
T_2	11.625	0.050	0.873	0.1886
T_3	12.500	0.050	1.335	0.1991
T_4	14.375	0.050	1.444	0.2295
T_5	12.000	0.050	1.398	0.1902
T_6	14.500	0.050	1.418	0.3227
T_7	13.750	0.050	1.324	0.2707
T_8	14.000	0.050	1.392	0.2735
T_9	14.250	0.050	1.234	0.2242
T_10	12.375	0.050	1.430	0.3343
T_11	12.063	0.050	0.833	0.1592
T_12	11.625	0.050	0.679	0.1754
T_13	11.813	0.050	1.260	0.2233

T_14	11.625	0.050	1.479	0.2503
T_15	11.688	0.050	0.312	0.0864
T_16	14.125	0.050	1.209	0.1819
T_17	14.375	0.050	1.162	0.2206
T_18	12.688	0.050	1.468	0.2459
T_19	12.500	0.050	1.166	0.2153
T_20	14.125	0.050	1.445	0.230
T_21	14.063	0.050	1.319	0.1980
T_22	11.750	0.050	1.501	0.204
T_23	12.125	0.050	0.892	0.256
T_24	13.875	0.050	1.303	0.233
T_25	11.688	0.050	1.481	0.2055
T_26	13.813	0.050	1.232	0.239
T_27	12.250	0.050	1.071	0.1928

T_28	11.750	0.050	1.133	0.179
T_29	11.688	0.050	1.117	0.2561
T_30	11.750	0.050	1.422	0.178

Sample ID	Stress (ksi)	Strain (in/in)	MoE (ksi)	Failure Notes	Comments
T_1	12.407	0.0103	1202	Node R	Failure at node
T_2	20.876	0.0162	1287	Node R	Failure at node
T_3	18.981 8	0.0159	1192	Both	Split from left node diagonally to right node
T_4	20.576	0.0160	1289	Node L	Fairly clean break through node.
T_5	20.213 2	0.0159	1275	Node R	First node L snapped partially and split to longitudinally along specimen to midpoint. Then node R snapped clean through.
T_6	18.112	0.0223	814	Node R	Break through the right node; Split from bottom along the longitudinal axis to the middle
T_7	16.456 1	0.0197	836	Both	Crack a little at left node first and then fairly clean break through the right node
T_8	18.072	0.0195	925	Node R	Crack from the right node toward the bottom, but still connected. Press pause button on Matlab and the bamboo break itself "pa..pa..bang"
T_9	21.464 5	0.0157	1364	Node L	Fairly clean break through the node

T_10	18.581	0.0270	688	Node L	Crack at left node a little and then split along the longitudinal axis towards both direction
T_11	20.8126	0.0132	1577	Node R	Cracked at right node then split across the bamboo to the other face and met near the aluminum tab
T_12	17.016	0.0151	1128	Node R	Clean split through node R
T_13	19.5114	0.0189	1032	Node R	Clean split at right grip No skin
T_14	18.871	0.0215	877	Both	Failed longitudinally from node to node Area of failure should be average between two nodes or middle cross sectional area
T_15	11.3220	0.0074	1531	Node L	Clean break through node L No skin
T_16	17.218	0.0129	1337	Node L	Initial failure on inside of left node then longitudinal failure diagonally near to right node
T_17	16.9365	0.0153	1103	Node R	Initial failure on inside of right node then clean failure across node
T_18	21.722	0.0194	1121	Node R	Cracked at right node then split across the bamboo to the other face and met near the aluminum tab and also split towards the middle of the specimen

T_19	22.759	0.0172	1321	Node L	Cracked at left node then split across the bamboo to the other face and met near the aluminum tab
T_20	20.300	0.0163	1245	Node L	Split from node L longitudinally to center of specimen and to left tabs
T_21	19.451	0.0141	1382	Node R	Split from node to grip
T_22	20.014	0.0174	1152	Node R	Split from node to grip
T_23	20.267	0.021	959	Node R	Clean break through node
T_24	16.417	0.017	979	Node R	Split from node R to grip
T_25	23.015	0.018	1309	Node L	Split from node L to grip
T_26	16.494	0.017	955	Node L	Split from node L to grip
T_27	15.993	0.016	1016	Node L	Split from node L to grip
T_28	18.417	0.015	1207	Node L	Split from node L to node R
T_29	18.246	0.022	833	Node L	Split from node to grip and to center Skin separated
T_30	21.237	0.015	1399	Node L	Split from node to grip and to center Split down center of skin

Appendix B – Bond strength test comprehensive data

	Non-Waterproofed									
Bonded length (in)	ID	Casting Date	Testing Date	# Nodes	# Diaphragms	Results	Force at Failure (lb)	Cross-sectional Area (in^2)	Perimeter (in)	Stress at Failure (ksi)	Bond stress (psi)
9	BN_1	10/15	10/23	1	1	Pullout	2869	0.3808	3.02	7.53	105.6
12	BN_2	10/15	10/23	1	1	Failure at grip	3150	0.3472	2.82	9.07	93.1
	BN_3	10/28	11/4	0	0	Failure at grip	3101	0.22	2.44	14.10	105.9
	BN_4	10/28	11/4	0	0	Pullout	1977	0.26	2.8	7.60	58.8
15	BN_5	10/15	10/23	2	2	Failure at grip	4272	0.352	2.9	12.14	98.2

	Waterproofed										
Bonded length (in)	ID	Casting Date	Testing Date	# Nodes	# Diaphragms	Results	Force at Failure (lb)	Cross-sectional Area (in^2)	Perimeter (in)	Stress at Failure (ksi)	Bond stress (psi)
12	BW_19	12/9	5/17	2	2	Failure at grip	2163	0.16	1.87	13.4	77.3
	BW_14	12/9	5/17	1	0	Failure at grip	2189	0.17	1.97	12.6	74.0
	BW_15	1/14	5/17	1	0	Pullout	2911	0.24	2.25	12.0	86.1
	BW_16	1/14	5/17	1	0	Failure near grip where holes drilled	2977	0.20	2.18	14.6	91.0
	BW_17	1/14	5/17	1	1	Pullout	3648	0.27	2.35	13.4	103.6
	BW_18	1/14	5/17	1	1	Pullout	1273	0.20	2.15	6.4	39.5
15	BW_9	12/9	5/17	2	2	Failure at grip	2334	0.25	2.21	9.5	70.3
	BW_12	12/9	5/17	2	0	Pullout, concrete failure	2228	0.22	2.09	10.0	71.2
	BW_7	1/14	5/17	1	1	Pullout	2435	0.19	1.99	13.1	81.7
	BW_8	1/14	5/17	2	2	n/a	n/a	0.22	2.09	n/a	n/a
	BW_10	1/14	5/17	1	0	Failure at grip	3018	0.22	2.05	13.6	98.3
	BW_11	1/14	5/17	2	2	Failure at grip	1603	0.19	1.99	8.6	53.6
	BW_13	1/14	5/17	2	2	Failure in bamboo	3790	0.29	2.41	13.2	105.0
18	BW_6	12/9	1/4	3	0	Failure at grip	2777	0.25	2.13	11.1	87.1
	BW_3	12/9	5/17	1	1	Failure at grip	2300	0.18	1.80	12.8	85.2
	BW_1	1/14	5/17	2	0	Failure at grip	2028	0.25	2.27	8.1	59.6
	BW_2	1/14	5/17	1	0	Failure at grip	2194	0.18	2.11	12.0	69.4
	BW_4	1/14	5/17	3	0	Failure at grip	1664	0.34	2.53	5.0	43.9
	BW_5	1/14	5/17	2	0	Failure in bamboo	2680	0.28	2.39	9.6	74.9

Appendix C – MathCAD beam designs

Beam Design

Inputs

IMPORTANT: Only change red inputs

Beam Dimensions and Loading

Width	$b := 10in$
Height	$h := 20in$
Total length	$L := 48in$
Bearing pad length	$L_{pad} := 6in$
Distance from support to load	$a := 17in$

Concrete Properties

Concrete strength	$f_c := 3000psi$	
Concrete rupture strain	$\varepsilon_c := 0.003$	
Compression block	$c := 2.836in$	**Check**

Bamboo Properties

Bamboo strength	$f_y := 18.7ksi$
Bamboo modulus of elasticity	$E_b := 1145ksi$
Bamboo rupture strain	$\varepsilon_{b\_yield} := \frac{f_y}{E_b} = 0.016$

Reinforcement

Number of rows	$N_{rows} := 4$
Stirrup spacing	$s := 4in$

Flexure Design - Bamboo

Beam Dimensions

Width $b = 10 \cdot in$

Height $h = 20 \cdot in$ Height to width ratio $\frac{h}{b} = 2$

Total length $L = 48 \cdot in$

Effective Length $L_e := L - (L_{pad} \cdot 2) = 36 \cdot in$

Concrete compressive strength $f_c = 3 \cdot ksi$

$$\beta_1 := \begin{cases} 0.85 & \text{if } 2500psi \le f_c \le 4000psi \\ \left(1.05 - 0.05 \frac{f_c}{1000 psi}\right) & \text{if } 4000psi \le f_c \le 8000psi \\ 0.65 & \text{otherwise} \end{cases}$$

$\beta_1 = 0.85$ because $f_c < 4000psi$

$\lambda := 1.0$ normal weight concrete

Concrete rupture strain $\varepsilon_c = 3 \times 10^{-3}$

Bamboo tensile strength $f_y = 18.7 \cdot ksi$

Bamboo modulus of elasticity $E_b = 1.145 \times 10^3 \cdot ksi$

Bamboo rupture strain $\varepsilon_{b\_yield} = 0.016$

Guess c value

Guess $c = 2.836 in$

$\alpha := c \cdot \beta_1$

Area of one bamboo strip

$A_{bstrip} := 0.31 in^2$ (from experimental data)

Number of rows required

$N_{rows} = 4$ ORIGIN:= 1

$$N_r := \begin{pmatrix} \text{if}(N_{rows} \geq 1, 1, 0) \\ \text{if}(N_{rows} \geq 2, 1, 0) \\ \text{if}(N_{rows} \geq 3, 1, 0) \\ \text{if}(N_{rows} \geq 4, 1, 0) \\ \text{if}(N_{rows} \geq 5, 1, 0) \\ \text{if}(N_{rows} \geq 6, 1, 0) \end{pmatrix} = \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 0 \\ 0 \end{pmatrix}$$

Number of bamboo strips provided

$$N_b := \begin{pmatrix} \text{if}(N_{rows} \geq 1, 1, 0) \\ \text{if}(N_{rows} \geq 2, 1, 0) \\ \text{if}(N_{rows} \geq 3, 1, 0) \\ \text{if}(N_{rows} \geq 4, 1, 0) \\ \text{if}(N_{rows} \geq 5, 1, 0) \\ \text{if}(N_{rows} \geq 6, 1, 0) \end{pmatrix} \cdot 3 = \begin{pmatrix} 3 \\ 3 \\ 3 \\ 3 \\ 0 \\ 0 \end{pmatrix}$$

Area of bamboo provided

$$A_b := N_b \cdot A_{bstrip} = \begin{pmatrix} 0.93 \\ 0.93 \\ 0.93 \\ 0.93 \\ 0 \\ 0 \end{pmatrix} \cdot in^2$$

Detailing

Clear cover $cc := 1.5in$

Horizontal spacing $s_h := 1in$

Vertical spacing $s_v := 1in$

Stirrup thickness $t_{stirrup} := 0.25in$

Bamboo strip width $w_b := 1in$

$cc \cdot 2 + s_h \cdot 2 + t_{stirrup} \cdot 2 + w_b \cdot 3 = 8.5 \cdot in$ $\leq$ $b = 10 \cdot in$ Detailing works

Note: must also take into account radius of stirrup

$d_1 := h - cc - \frac{t_{stirrup}}{2}$ $d_2 := d_1 - s_v - t_{stirrup}$ $d_3 := d_2 - s_v - t_{stirrup}$

$d_4 := d_3 - s_v - t_{stirrup}$ $d_5 := d_4 - s_v - t_{stirrup}$ $d_6 := d_5 - s_v - t_{stirrup}$

$$x := \begin{pmatrix} d_1 \cdot N_{r_1} \\ d_2 \cdot N_{r_2} \\ d_3 \cdot N_{r_3} \\ d_4 \cdot N_{r_4} \\ d_5 \cdot N_{r_5} \\ d_6 \cdot N_{r_6} \end{pmatrix} = \begin{pmatrix} 18.375 \\ 17.125 \\ 15.875 \\ 14.625 \\ 0 \\ 0 \end{pmatrix} \cdot in \qquad d := \frac{\left[\sum_{i=1}^{N_{rows}} \left(x_i \cdot N_{b_i}\right)\right]}{\sum_{i=1}^{N_{rows}} \left(N_{b_i}\right)} \qquad \rho_b := \frac{\sum A_b}{(b \cdot d)}$$

Depths to centroid of bamboo layers

$d = 16.5in$

Reinforcement ratio

$\rho_b = 0.023$

Model of bamboo (Rupture or Yielding)

Choose either rupture or yielding model for bamboo by dragging the function into the box

Rupture $$f_b(\varepsilon_b, f_y) := \begin{cases} \mathrm{sign}(\varepsilon_b)\cdot(E_b\cdot\varepsilon_b) & \text{if } |\varepsilon_b| < \varepsilon_{b\_yield} \\ \mathrm{sign}(\varepsilon_b)\cdot\frac{2}{3}f_y & \text{otherwise} \end{cases}$$

Yielding $$f_b(\varepsilon_b, f_y) := \begin{cases} \mathrm{sign}(\varepsilon_b)\cdot(E_b\cdot\varepsilon_b) & \text{if } \varepsilon_b < \varepsilon_{b\_yield} \\ \mathrm{sign}(\varepsilon_b)\cdot f_y & \text{otherwise} \end{cases}$$

Strain in each layer

$$\varepsilon_b := \begin{pmatrix} \begin{cases} \frac{x_1 - c}{c}\cdot\varepsilon_c & \text{if } x_1 > 0 \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{x_2 - c}{c}\cdot\varepsilon_c & \text{if } x_2 > 0 \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{x_3 - c}{c}\cdot\varepsilon_c & \text{if } x_3 > 0 \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{x_4 - c}{c}\cdot\varepsilon_c & \text{if } x_4 > 0 \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{x_5 - c}{c}\cdot\varepsilon_c & \text{if } x_5 > 0 \\ 0 & \text{otherwise} \end{cases} \\ \begin{cases} \frac{x_6 - c}{c}\cdot\varepsilon_c & \text{if } x_6 > 0 \\ 0 & \text{otherwise} \end{cases} \end{pmatrix}$$

Stress in each layer

$$Stress := \begin{pmatrix} \mathrm{sign}(\varepsilon_{b_1})\cdot f_b(\varepsilon_{b_1}, f_y) \\ \mathrm{sign}(\varepsilon_{b_2})\cdot f_b(\varepsilon_{b_2}, f_y) \\ \mathrm{sign}(\varepsilon_{b_3})\cdot f_b(\varepsilon_{b_3}, f_y) \\ \mathrm{sign}(\varepsilon_{b_4})\cdot f_b(\varepsilon_{b_4}, f_y) \\ \mathrm{sign}(\varepsilon_{b_5})\cdot f_b(\varepsilon_{b_5}, f_y) \\ \mathrm{sign}(\varepsilon_{b_6})\cdot f_b(\varepsilon_{b_6}, f_y) \end{pmatrix}$$

$$\text{Result} := \begin{pmatrix} \text{if}(\varepsilon_{b_1} < \varepsilon_{b\_yield}, \text{"NOT YIELDED"}, \text{"YIELDED"}) \\ \text{if}(\varepsilon_{b_2} < \varepsilon_{b\_yield}, \text{"NOT YIELDED"}, \text{"YIELDED"}) \\ \text{if}(\varepsilon_{b_3} < \varepsilon_{b\_yield}, \text{"NOT YIELDED"}, \text{"YIELDED"}) \\ \text{if}(\varepsilon_{b_4} < \varepsilon_{b\_yield}, \text{"NOT YIELDED"}, \text{"YIELDED"}) \\ \text{if}(\varepsilon_{b_5} < \varepsilon_{b\_yield}, \text{"NOT YIELDED"}, \text{"YIELDED"}) \\ \text{if}(\varepsilon_{b_6} < \varepsilon_{b\_yield}, \text{"NOT YIELDED"}, \text{"YIELDED"}) \end{pmatrix} \qquad \text{Summary} := \begin{pmatrix} N_{r_1} & \varepsilon_{b_1} & \text{Result}_1 \\ N_{r_2} & \varepsilon_{b_2} & \text{Result}_2 \\ N_{r_3} & \varepsilon_{b_3} & \text{Result}_3 \\ N_{r_4} & \varepsilon_{b_4} & \text{Result}_4 \\ N_{r_5} & \varepsilon_{b_5} & \text{Result}_5 \\ N_{r_6} & \varepsilon_{b_6} & \text{Result}_6 \end{pmatrix}$$

Strain in each bamboo layer

$$\varepsilon_b = \begin{pmatrix} 0.016 \\ 0.015 \\ 0.014 \\ 0.012 \\ 0 \\ 0 \end{pmatrix}$$

Stress in bamboo layer

$$\text{Stress} = \begin{pmatrix} 18.7 \\ 17.307 \\ 15.793 \\ 14.279 \\ 0 \\ 0 \end{pmatrix} \cdot \text{ksi}$$

Check to make sure Compression Force = Tension Forces

Check $c = 2.836\,\text{in}$ **Re-try?**

Compression Force $C_c := -(0.85 \cdot f_c) \cdot (0.85 \cdot c) \cdot b = -61.47 \cdot \text{kip}$

Force in Bamboo $F_b := \sum_{i=1}^{N_{rows}} \left(\text{Stress}_i \cdot A_{b_i}\right) = 61.453\,\text{kip}$

$C_c + F_b = -0.017 \cdot \text{kip}$

$\text{Action} := \text{if}\left[(C_c + F_b < -0.2\text{kip}), \text{"DECREASE c"}, \text{if}(C_c + F_b > 0.2\text{kip}, \text{"INCREASE c"}, \text{"GOOD"})\right]$

$$c_{approx} := \frac{F_b}{\frac{-C_c}{c}} = 2.835\,\text{in}$$

$\text{Action} = \text{"GOOD"}$

$c_{approx} = 2.835\,\text{in}$

Calculate Flexural Capacity

$$M_n := \sum_{i=1}^{N_{rows}} \left[\left(Stress_i\right) \cdot \left(x - \frac{\alpha}{2}\right)_i \cdot A_{b_i} \right] = 79.042 ft \cdot kip$$

Calculate Cracking Moment

$d = 16.5 in$

$\alpha = 2.411 in$

$F_b = 61.453 kip$

$\sum A_b = 3.72 in^2$

$$f_r := 7.5\lambda \cdot \sqrt{f_c \cdot psi}$$

$$I_g := \frac{1}{12} \cdot b \cdot h^3$$

$$y_t := \frac{h}{2}$$

$$M_{cr} := \frac{f_r \cdot I_g}{y_t} \qquad M_{cr} = 22.822 kip \cdot ft$$

Action = "GOOD"

$M_n = 79.042 kip \cdot ft$

$M_{cr} = 22.822 kip \cdot ft$

Shear Design - Bamboo

Calculate Shear Capacity of Concrete

$V_c := 2 \cdot \lambda \cdot \sqrt{f_c \cdot psi} \cdot b \cdot d$

$V_c = 18.075\,kip$

Calculate Shear Capacity of Bamboo Stirrups

Stirrup Spacing $s = 4 \cdot in$

Stirrup Area $A_v := 2 \cdot \left(0.20 in^2\right)$

$V_s := \frac{A_v \cdot f_y \cdot d}{s}$

$V_s = 30.855\,kip$

Shear Capacity of Beam

$V_n := V_c + V_s$

$V_n = 48.93\,kip$

$V_n = 48.93\,kip$

UTM Loading

$a = 17 \cdot in$

$P := \min\left(\frac{M_n \cdot 2}{a}, V_n \cdot 2\right) = 97.86 kip$

$P_{cr} := \frac{2 \cdot M_{cr}}{a} = 32.219 kip$

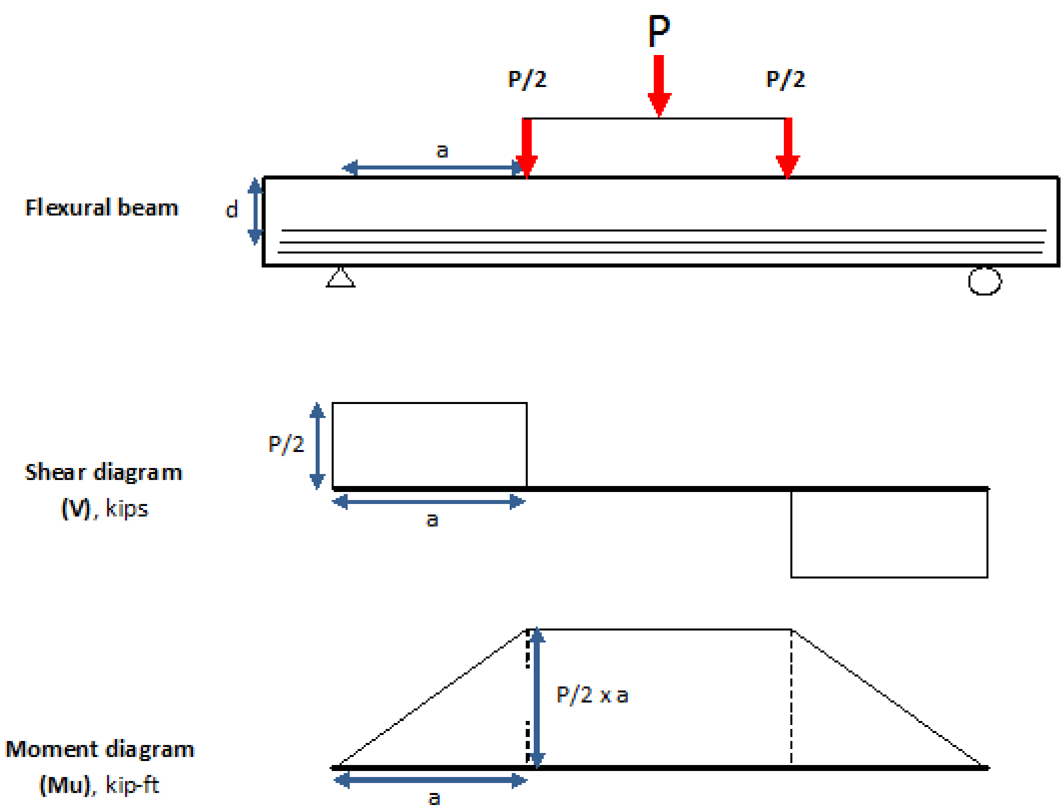
$V_u := \frac{P}{2}$

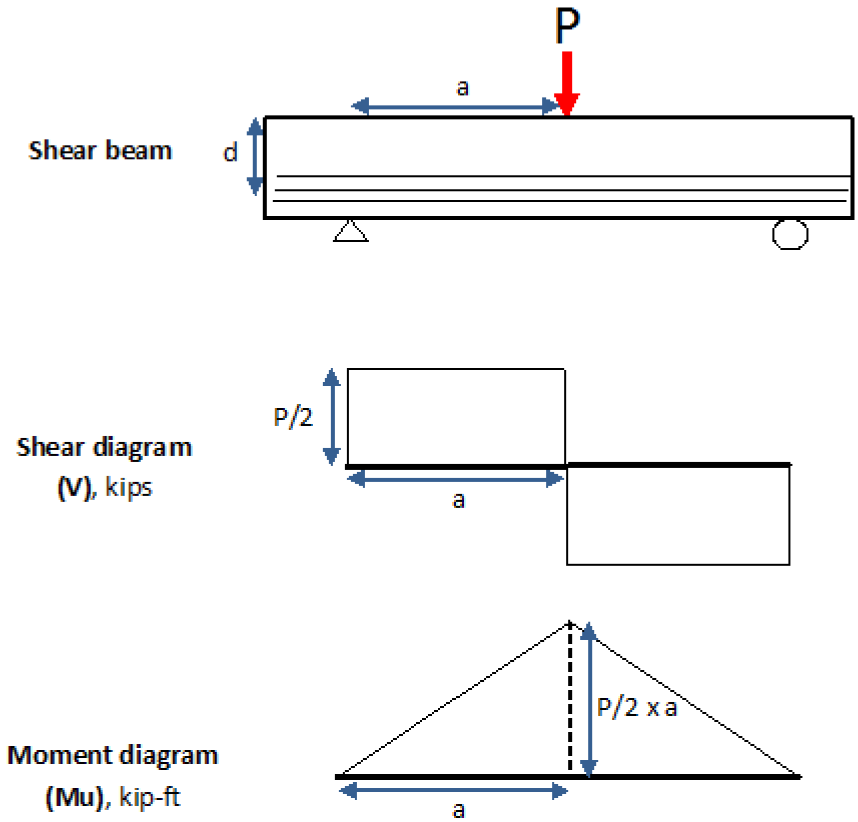
$M_u := V_u \cdot a$

For Shear Beams (S1, S2, S3, S4, S5, S6) use an "a" = Leff/2 = 17in
For Flexure Beams (F1, F2) and Lap-Spliced Beams (L1, L2, L3) use an "a" = 25in

For Flexure Beam (F3) use an "a" = 30in

For Flexure Beam (F4) use an "a" = 38in





$$Status := if\left(M_u \geq M_n, \text{"FLEXURE FAILURE"}, if\left(V_u \geq V_n, \text{"SHEAR FAILURE"}, \text{"NO FAILURE"}\right)\right)$$

$$Status := \begin{cases} \text{"TENSION FLEXURE FAILURE"} & \text{if } \begin{cases} M_u \geq M_n \\ \left(\varepsilon_{b_1} > \varepsilon_{b\_yield}\right) \end{cases} \\ \text{"OVER-REINFORCED FLEXURE FAILURE"} & \text{if } \varepsilon_{b_1} < \varepsilon_{b\_yield} \\ \text{"BALANCED FLEXURE FAILURE"} & \text{if } \varepsilon_{b_1} = \varepsilon_{b\_yield} \\ \text{"SHEAR FAILURE"} & \text{if } V_u \geq V_n \end{cases}$$

$P = 97.86\,kip$

$P_{cr} = 32.219\,kip$

$V_u = 48.93\,kip$

$M_u = 69.317\,kip \cdot ft$

Summary

Action = "GOOD"

Status = "SHEAR FAILURE"

Expected Load	$P = 97.86\,kip$
Cracking Load	$P_{cr} = 32.219\,kip$
Cracking Moment	$M_{cr} = 22.822\,ft \cdot kip$
Moment Capacity	$M_n = 79.042\,kip \cdot ft$
Ultimate Moment	$M_u = 69.317\,kip \cdot ft$
Shear Capacity	$V_n = 48.93\,kip$
Ultimate Shear	$V_u = 48.93\,kip$

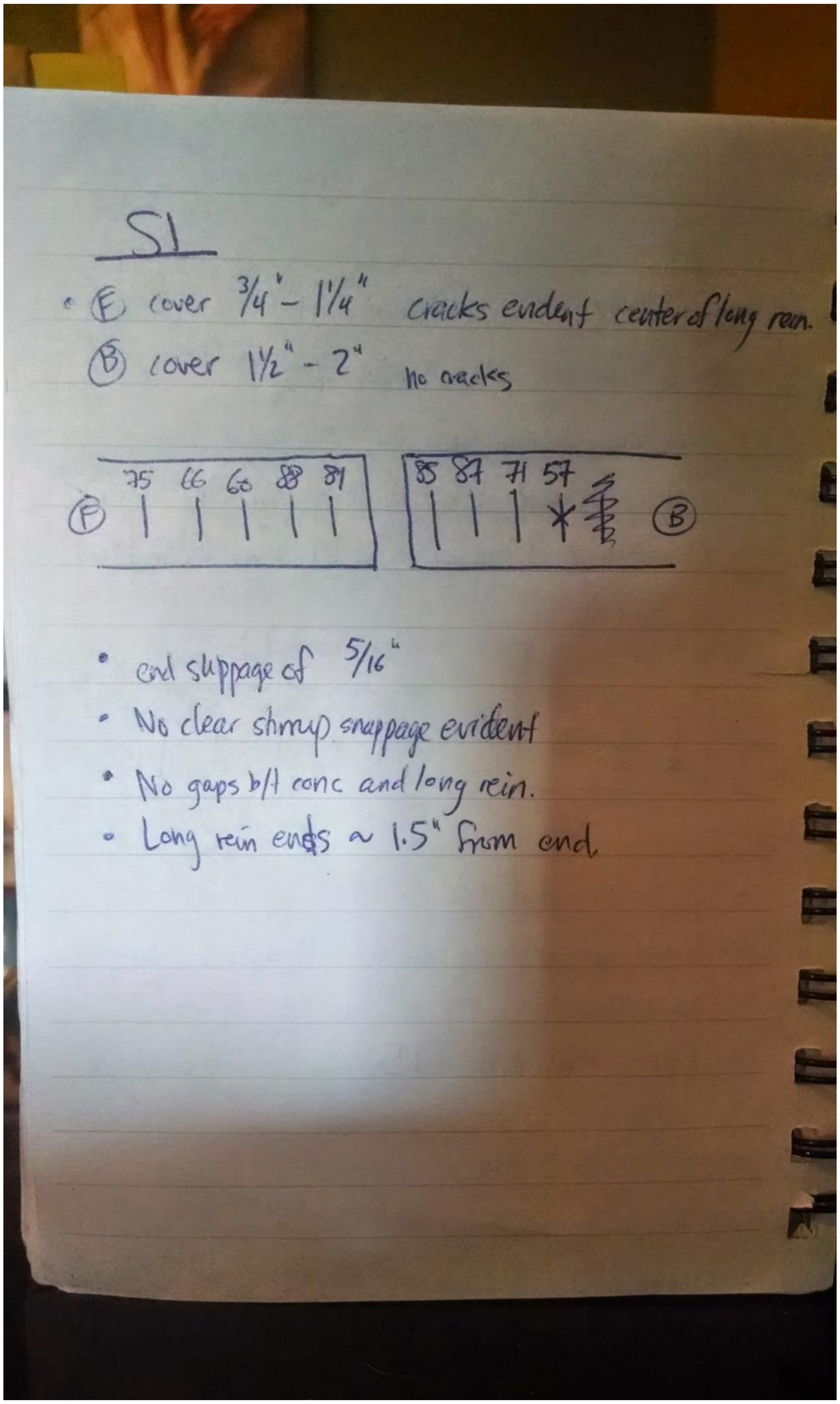
Yielding bamboo assumes stress in bamboo remains fb when yielding strain is reached

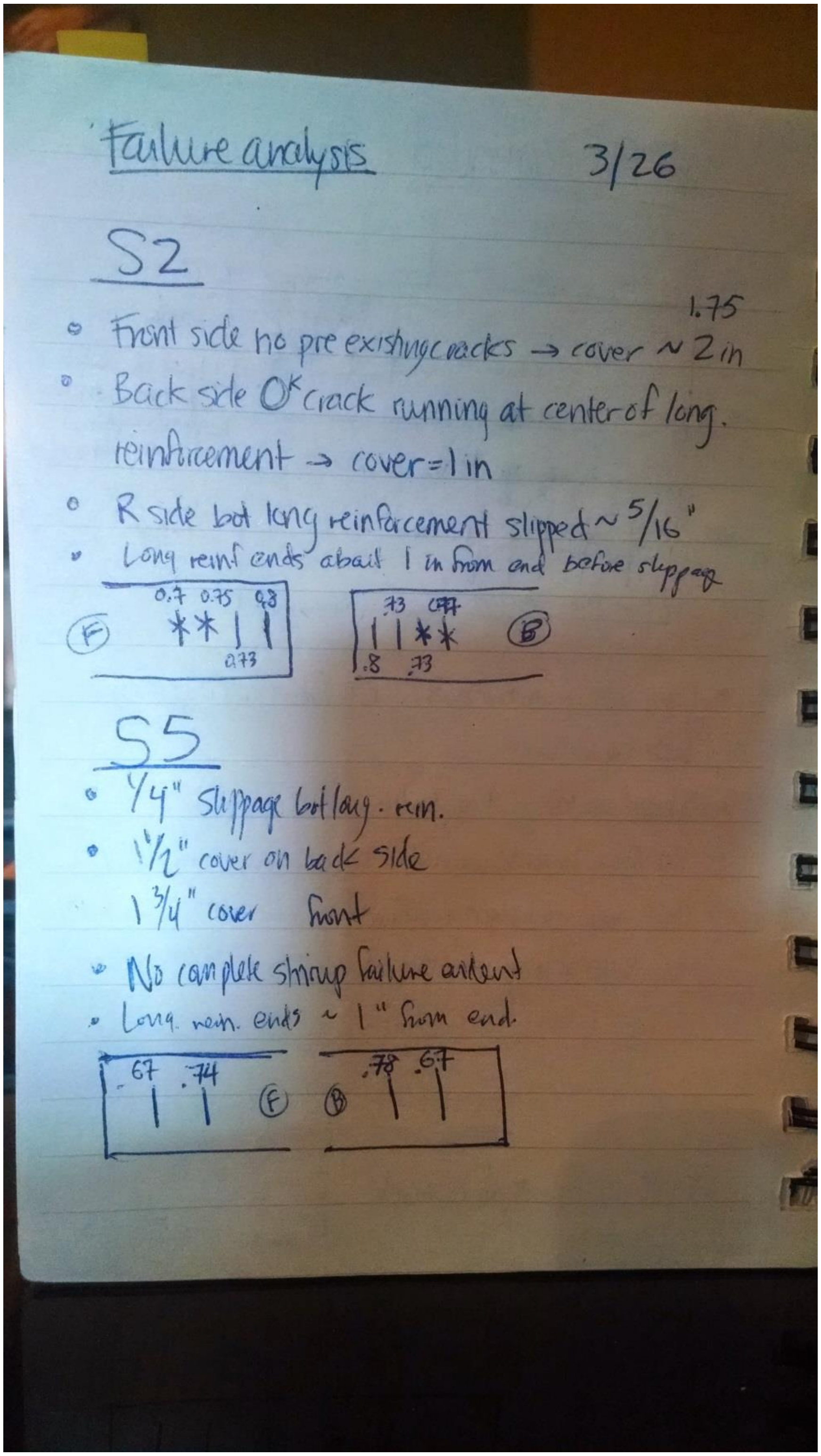
Rupture bamboo assumes stress in bamboo drops to 2/3 fb when rupture strain is reached

Layer Strain Status

$$\text{Summary} = \begin{pmatrix} 1 & 0.016 & \text{"YIELDED"} \\ 1 & 0.015 & \text{"NOT YIELDED"} \\ 1 & 0.014 & \text{"NOT YIELDED"} \\ 1 & 0.012 & \text{"NOT YIELDED"} \\ 0 & 0 & \text{"NOT YIELDED"} \\ 0 & 0 & \text{"NOT YIELDED"} \end{pmatrix} \qquad \varepsilon_{b\_yield} = 0.016$$

Appendix D – Beam examination notes





Failure analysis 3/26

S2

- Front side no pre existing cracks → cover ~ 2 in (1.75)
- Back side OK crack running at center of long. reinforcement → cover = 1 in
- R side bot long reinforcement slipped ~ 5/16"
- Long reinf ends about 1 in from end before slippage

(F) 0.7 0.75 0.8 0.73 | .73 0.77 .8 .73 (B)

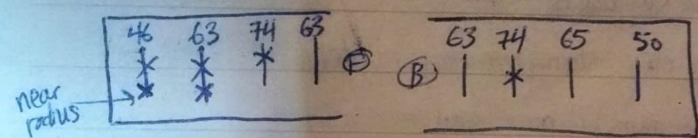
S5

- 1/4" slippage bot long. rein.
- 1 1/2" cover on back side
  1 3/4" cover front
- No complete stirrup failure evident
- Long. rein. ends ~ 1" from end

.67 .74 (F) | (B) .78 .67

S3

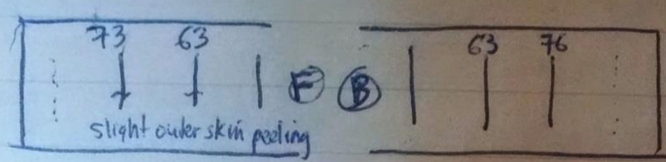
- (F) 1¼" cover → crack evident
  (B) 2¾" cover → no crack
- Once shear failure occured large 45° crack formed and confinement of long reinf. comprimised. Conc. slipped off bamboo + on bot. layer diaphragm sheared off.
- Huge shear crack → seeing daylight



- slippage on end of 1⅛"
- Maybe amt of slippage on ends corresponds to shear crack size and gap b/t conc. + long reinf.
- Long rein ends ~1.5" from end before slippage

S4

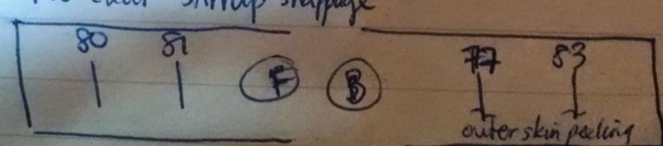
- Cover Ⓕ 1¼ - 1½" no crack
  - Ⓑ 1⅞" no crack



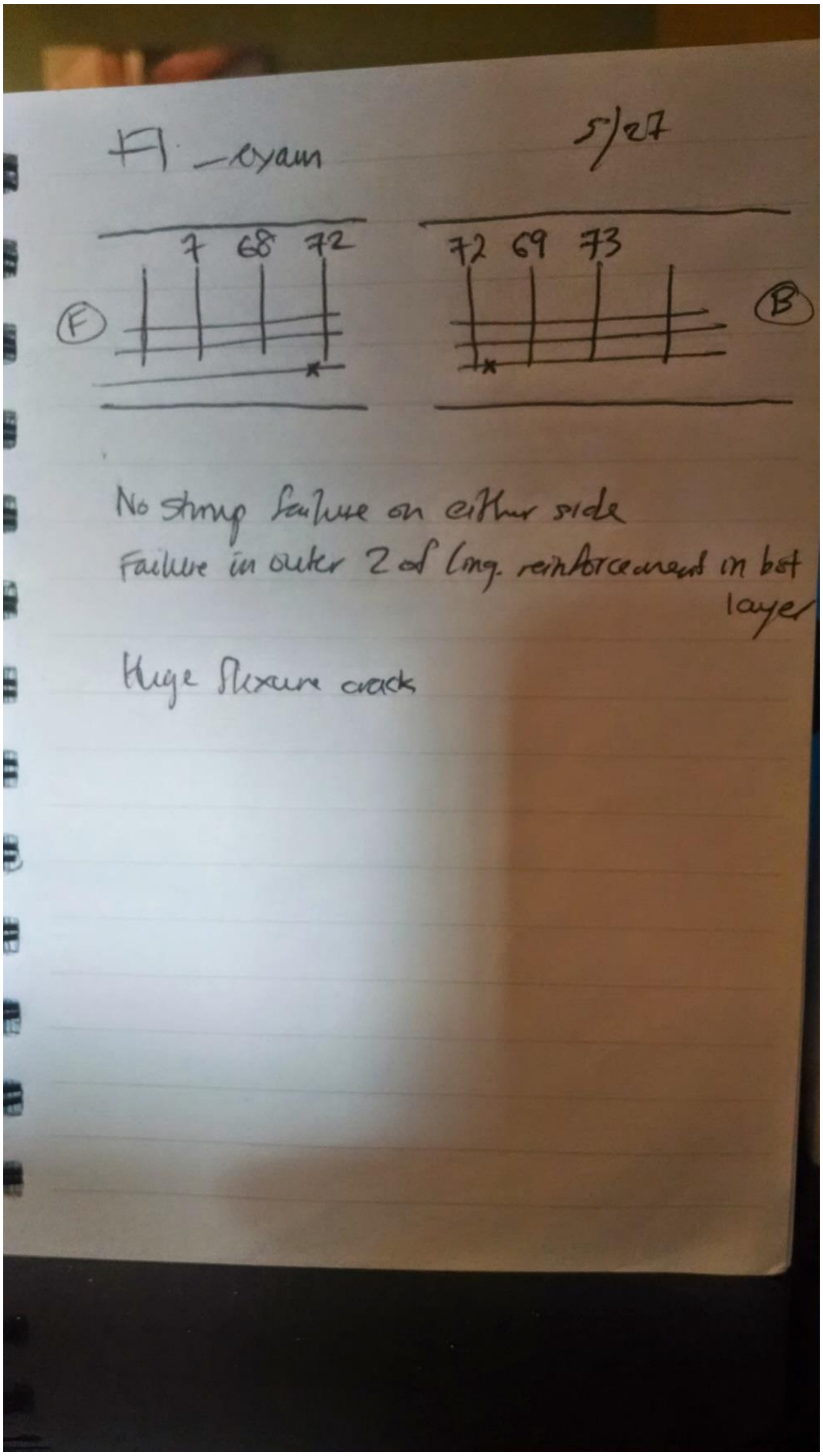
- No clear stirrup snappage
- No gaps b/t conc. & long rein.

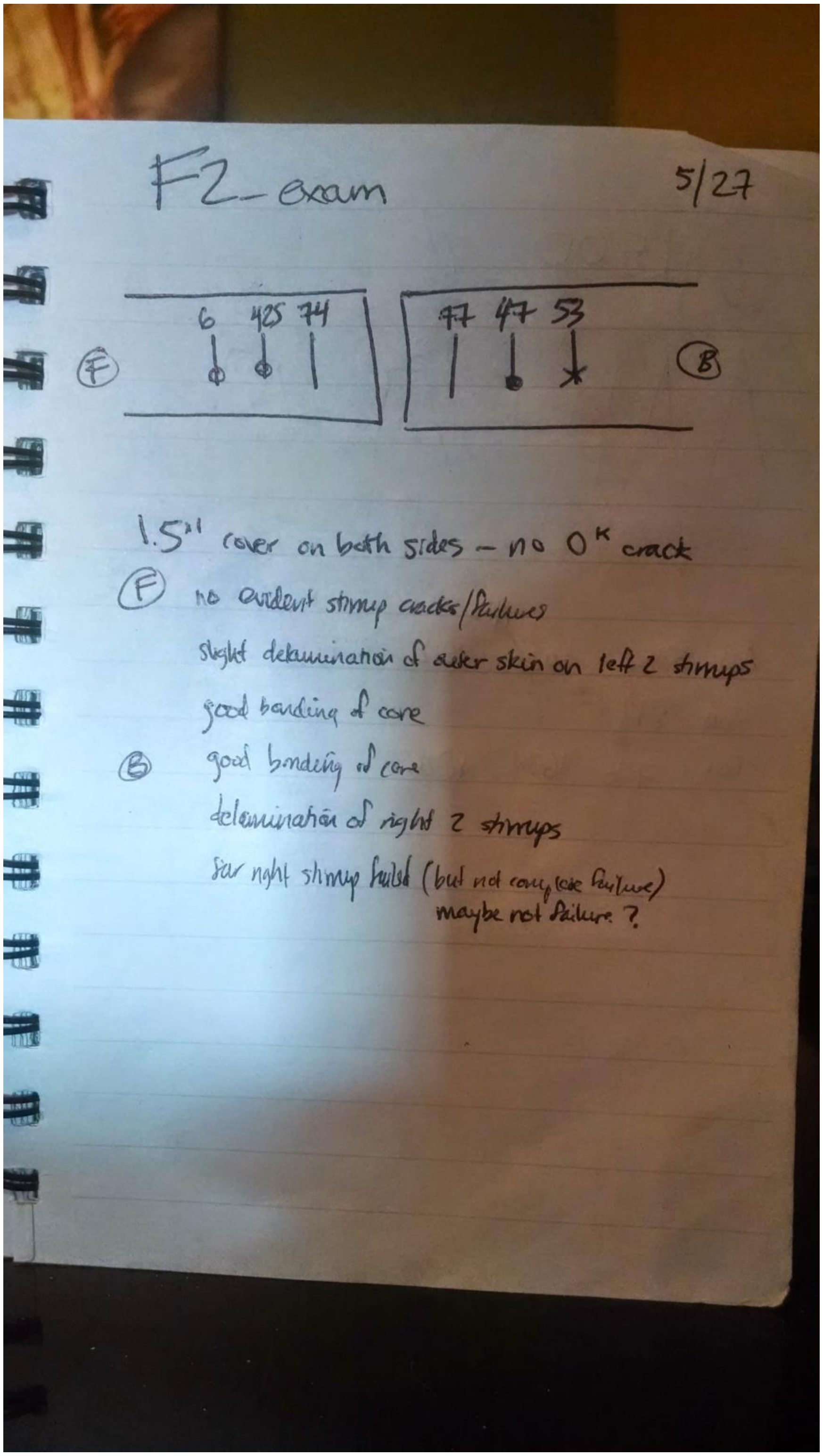
S6

- Cover Ⓕ 2" no crack
  - Ⓑ 1¼ - 1½ no crack
- No clear stirrup snappage

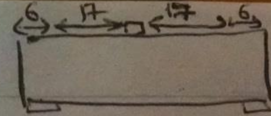


- Cracks form above diaphragms
- No gap b/t conc. & long rein.
- End slipped ¼" • Bam. ~1½" from end before slippage


5/27
7 68 72
72 69 73
F
B
No stirrup failure on either side
Failure in outer 2 of long. reinforcement in bot layer
Huge flexure crack


F2_exam
5/27
6 425 74
47 47 53
F
B
1.5" cover on both sides – no 0K crack
F no evident stirrup cracks/failures
slight delamination of outer skin on left 2 stirrups
good bonding of core
B good bonding of core
delamination of right 2 stirrups
far right stirrup failed (but not complete failure)
maybe not failure ?

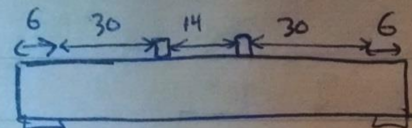
3/24 F3



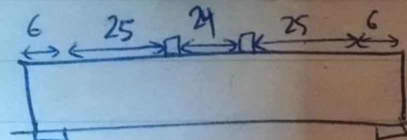
S

2 flexure cracks prior to loading

→ we chained it up but not high enough



F3 & (F4) → single point load



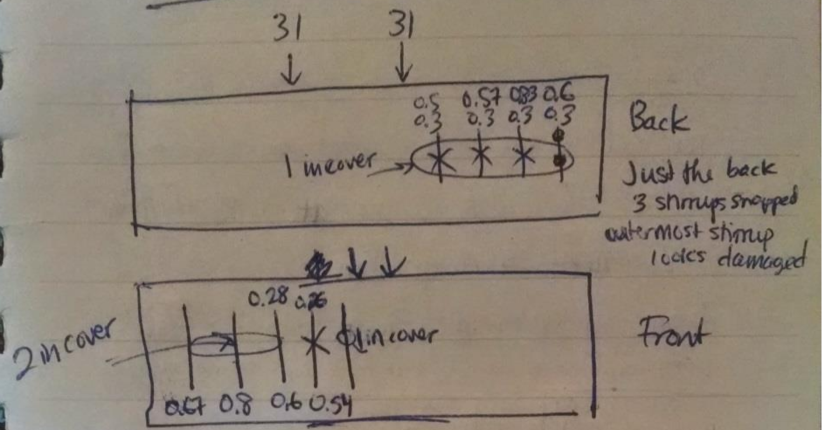
F1 & F2

At 1.2 in displacement we paused it.

- ~~The 3 stirrups on the back side (R) may have been damaged by the chisel~~ (Disregard)
- Delamination of top reinforcement caused by stirrups snapping (shear failure)
- Bearing failure b/c reinforcement ends 6" from end
- 1" cover on back side → same side as initial cracks
- vertical cracks align w/ stirrups

F3 continued

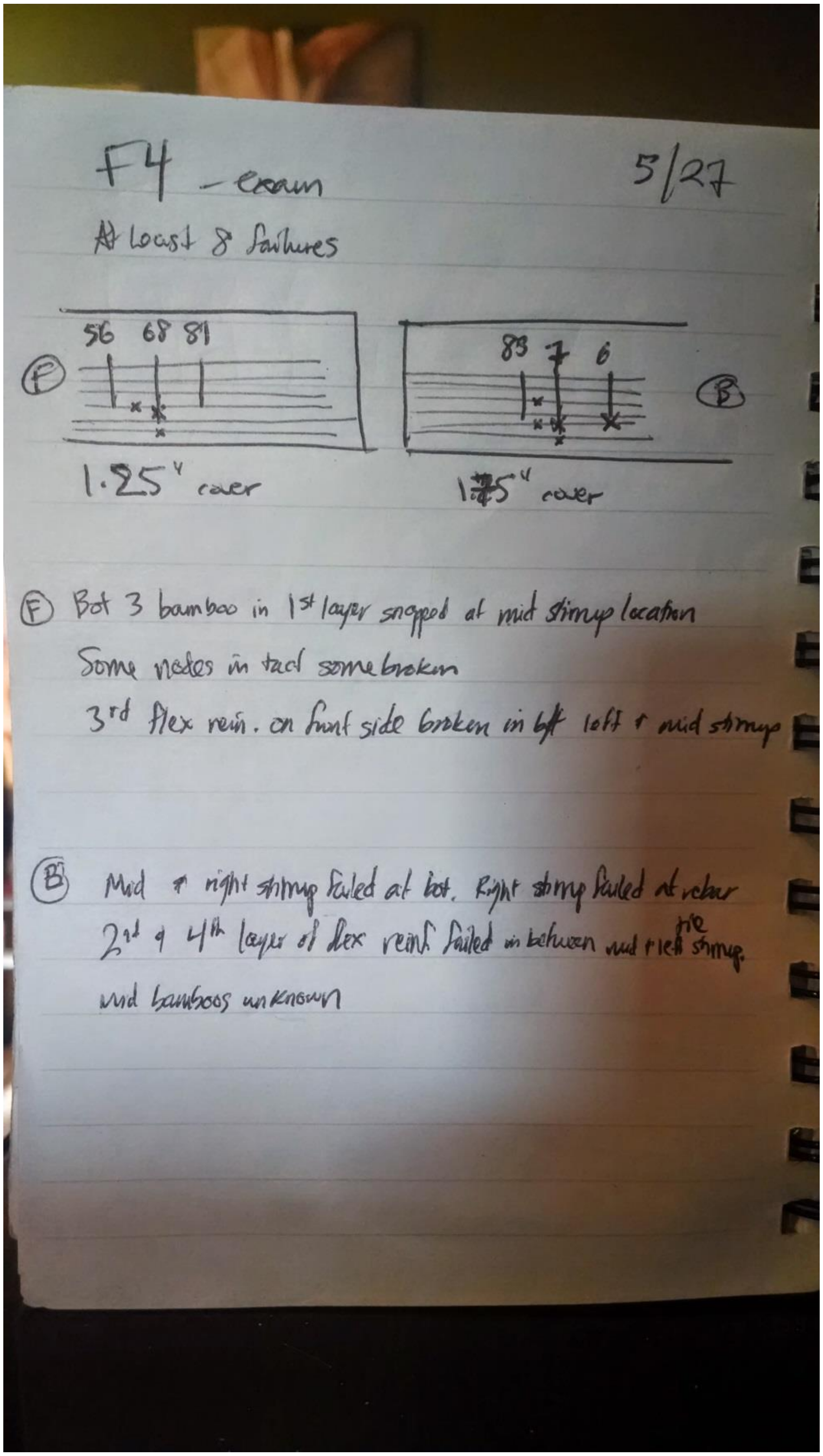
3/25



- Long. reinforcement ends ~6 in from end resulting in bearing failure
- Shear cracks point to loading points on top
- Top flex. layer delaminated once stirrups snapped & conc. was no longer confined
- Bot layer of long rein. slipped ~ 0.5 in

Find stiffness of concrete $E = 1$

$E_{bamboo} = 1150$ ksi



F4 - exam 5/27

At least 8 failures

56 68 81

F

1.25" cover

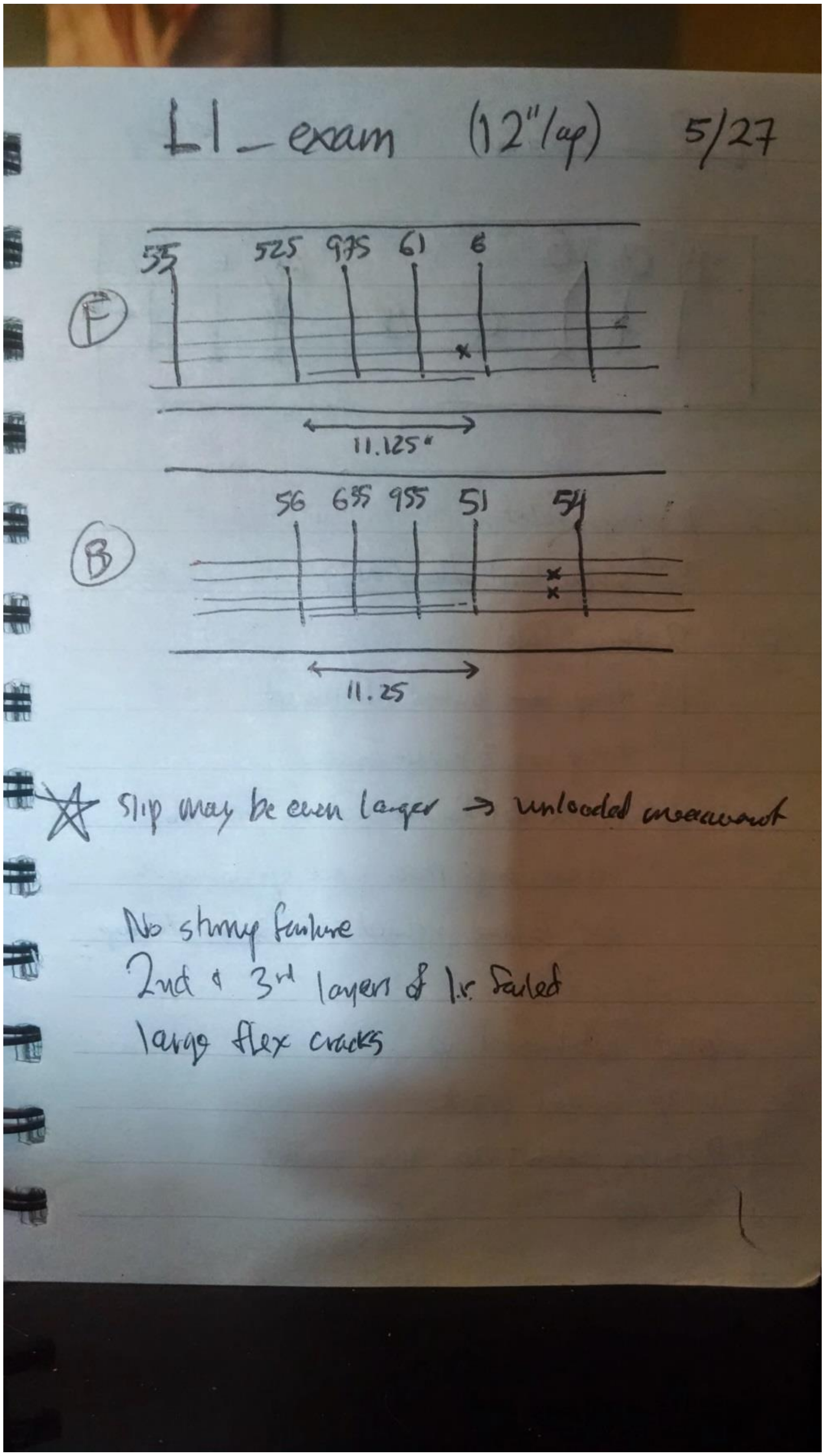
83 7 6

B

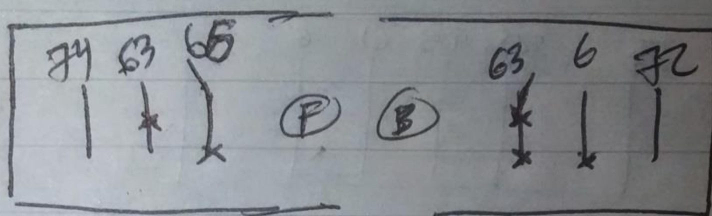
1.[illegible]5" cover

(F) Bot 3 bamboo in 1st layer snapped at mid stirrup location
Some nodes in tact some broken
3rd flex rein. on front side broken in b/t left & mid stirrups

(B) Mid & right stirrup failed at bot. Right stirrup failed at rebar tie
2nd & 4th layer of flex reinf failed in between mid & left stirrup.
mid bamboos unknown


L1_exam (12"/ap) 5/27
55
525
975
61
6
F
11.125"
56
655
955
51
54
B
11.25
Slip may be even larger → unloaded measurement
No strong failure
2nd & 3rd layers of l.r. failed
large flex cracks

L2 - exam (18") 5/27



(F) 2 stirrups failed

3rd stirrup bent ~~[illegible]~~ also

(B) 2 stirrups failed

1st stirrup ~~bent~~ out of alignments

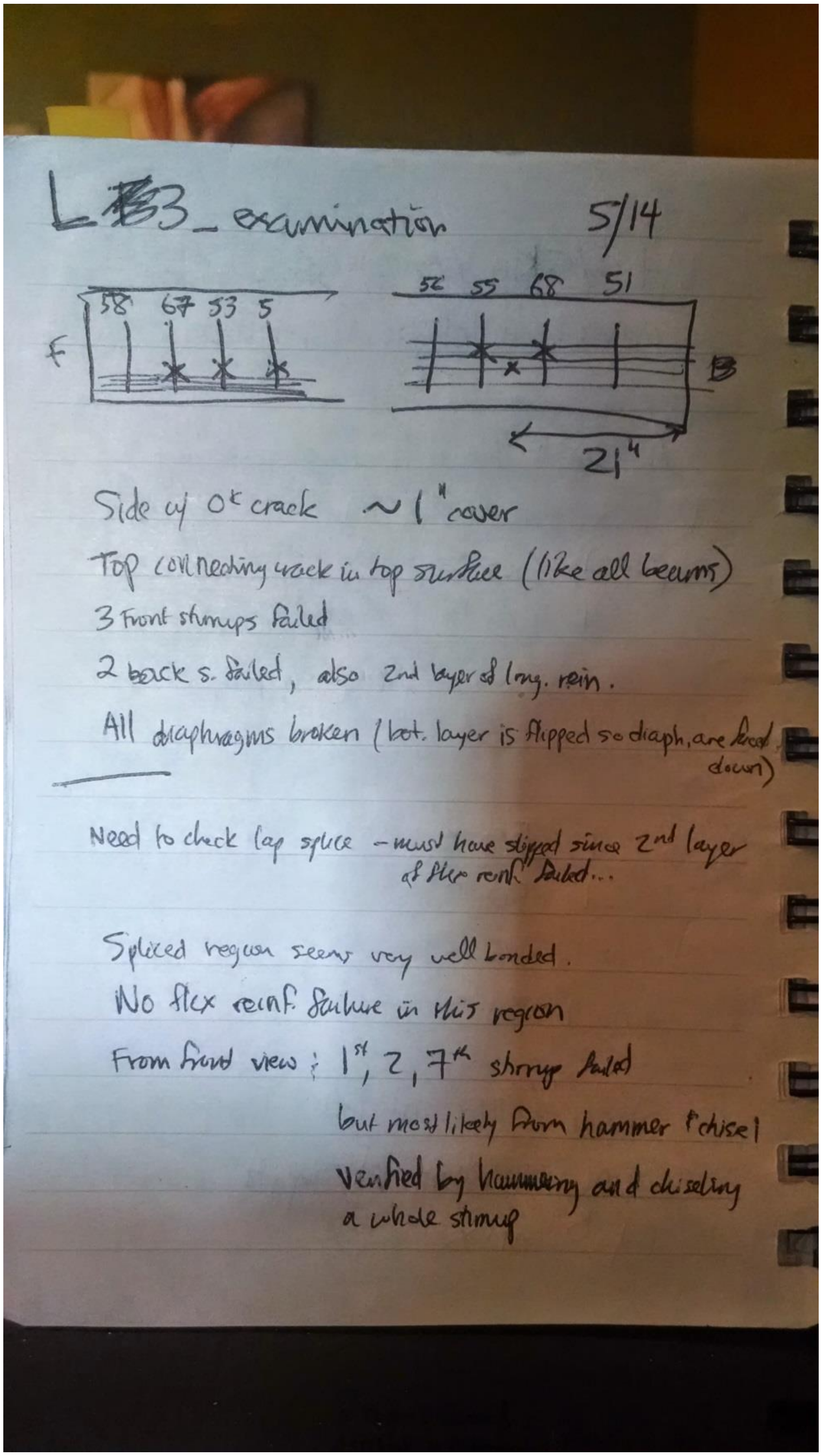
1st stirrup has 2 failure locations

think bot failure occured first then when large shear crack opened up the 2nd failure occured in the same stirrup

conc. all busted up

large shear crack

flexure cracks are very small



LB3_examination 5/14

Side w/ 0k crack ~1" cover

Top connecting crack in top surface (like all beams)

3 front stirrups failed

2 back s. failed, also 2nd layer of long. rein.

All diaphragms broken (bot. layer is flipped so diaph. are faced down)

Need to check lap splice - must have slipped since 2nd layer of flex reinf. failed...

Spliced region seems very well bonded.

No flex reinf. failure in this region

From front view: 1st, 2, 7th stirrup failed

but most likely from hammer & chisel

verified by hammering and chiseling a whole stirrup

Appendix E – Sensors channel spreadsheet

A/D Board 00	Sensor	Cable	RDP	A/D	Units	S.F.	Column1	Column2	Column3	Column4	Column5	Column6	Column7	Column8	Column9
Description	Connection	Number	Slot	Channel		(units/volt)	BDI	GGF	Position	Gage Length (in)	V_{zero} (volts)	V_{exc}	V_{cal}	Gain	Vcollect/volt
Sync Channel	A/D Pin Board			0	volts	1									
BDI-top left center	RDP Patch Panel	7209	8A	1	243.71	129.43	7029	350.9	ddd	3.0	4.117	10	8.196	271.12	6
BDI-bot-center +4"	RDP Patch Panel	6483	8B	2	1829.90	144.84	6483	345.9	eee	3.0	-6.634	10	-3.041	238.82	6
BDI-top center center	RDP Patch Panel	6482	9A	3	234.40	127.32	6482	343.0	ee	3.0	4.159	10	8.212	269.39	6
BDI-top-center +4"	RDP Patch Panel	6485	9B	4	−128.60	82.44	6485	339.5	f	3.0	7.56	10	13.756	411.83	6
BDI-top right center	RDP Patch Panel	6481	10A	5	249.80	128.17	6481	344.0	e	3.0	4.051	10	8.089	268.39	6
BDI-bottom left center	RDP Patch Panel	7243	10B	6	1165.14	122.74	7243	326.4		3.0	-3.493	10	0.508	265.94	6
BDI-bottom center back	RDP Patch Panel	6374	11A	7	1584.46	131.08	6374	349.8	bbb	3.0	-6.088	10	-2.073	266.87	6
BDI-bottom center center	RDP Patch Panel	6373	11B	8	1419.91	128.99	6373	343.2	bb	3.0	-5.008	10	-1.005	266.07	6
BDI-bottom center front	RDP Patch Panel	6375	12A	9	793.92	132.32	6375	351.8	c	3.0	0	10	4	265.87	6
BDI-bottom right center	RDP Patch Panel	6376	12B	10	779.26	129.88	6376	345.3	cc	3.0	0	10	4	265.87	6
String Pot LVDT-left	A/D Pin Board	1		11	inches	-0.518885695									
String Pot LVDT-right	A/D Pin Board	2		12	inches	-0.5305									
UNUSED				13											
UNUSED				14											
UNUSED				15											

A/D Board 01	Sensor	Cable	RDP	A/D	Units	S.F.	Column2	Column5	Column6	Column7	Column8	Column9
Description	Connection	Number	Slot	Channel		(units/volt)	GF	V_{zero}/volt	V_{exc}	V_{cal}	Gain	Vcollect/volt
Sync Channel	A/D Pin Board			0	volts	1						
AC LVDT-top front	RDP Patch Panel	1	1A	1	inches	0.0295		0	10			
AC LVDT-top back	RDP Patch Panel	2	1B	2	inches	0.0311		0	10			
AC LVDT-bottom front	RDP Patch Panel	3	2A	3	inches	0.0308		0	10			
AC LVDT-bottom back	RDP Patch Panel	4	2B	4	inches	0.0285		0	10			
AC LVDT-bottom vertical	RDP Patch Panel	5	3A	5	inches	0.0433		0	10			
UNUSED				6				0	10			
SG-stirrup 3	1/4 Box -> RDP Patch	1	4A	7	42788.80	2316.66	2.100	-12.47	10	-11.233	82.22	6
SG-stirrup 4	1/4 Box -> RDP Patch	2	4B	8	39973.98	2281.62	2.100	-11.52	10	-10.264	83.48	6
SG-flexural mid	1/4 Box -> RDP Patch	3	5A	9	26896.34	2201.01	2.100	-6.22	10	-4.918	86.54	6
SG-stirrup 8	1/4 Box -> RDP Patch	4	5B	10	24387.18	2201.01	2.100	-5.08	10	-3.778	86.54	6
SG-stirrup 9	1/4 Box -> RDP Patch	5	6A	11	27246.33	2204.40	2.100	-6.36	10	-5.06	86.41	6
SG-right center top	1/4 Box -> RDP Patch	6	6B	12	32517.80	2206.09	2.100	-8.74	10	-7.441	86.34	6
SG-right back bot	1/4 Box -> RDP Patch	7	7A	13	22750.71	2187.57	2.100	-4.4	10	-3.09	87.07	6
UNUSED			7B	14								
UNUSED				15								

Appendix F – Strain analysis steps

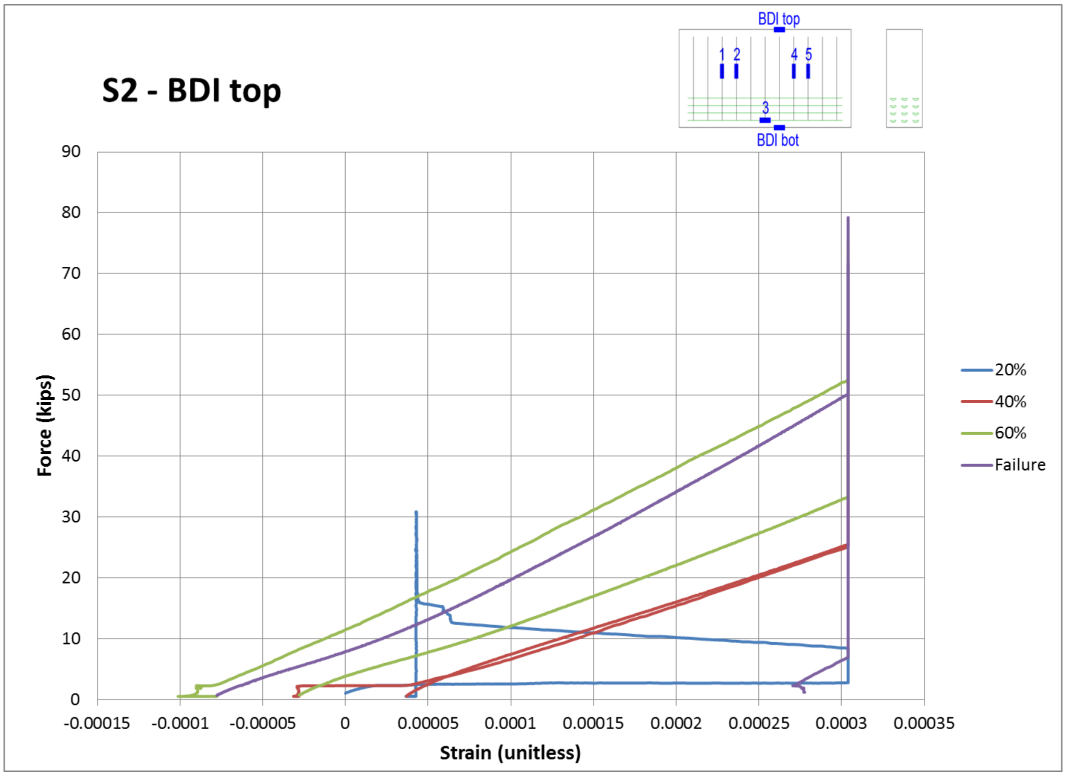
The following steps were taken to synchronize the strain gauge data with the actuator data. The computer collecting the strain gauge data continued to gather data even while the actuator was paused, resulting in many more strain gauge data points than actuator data points. A function generator was used to time-stamp each of the output data files (actuator, B00, B01) to aid in synchronization.

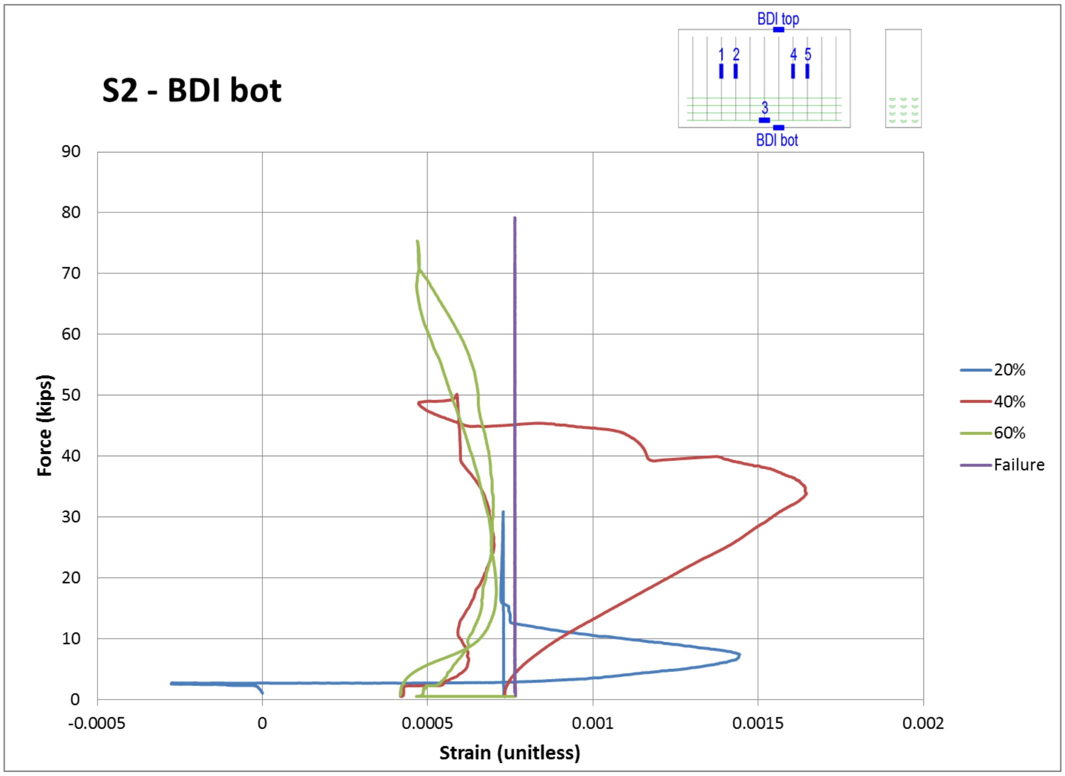
1. Copy and paste Actuator raw data into a new excel spreadsheet
2. Delete time step gaps
3. Move function generator data to left-most column
4. Add in B00 and B01 data including both function generators
5. Include date and time for all three data sets
6. Add in consecutive numbers in left-most column. Don't separate numbers in this column.
7. Set conditional formatting rules for the following rules:
    a. Force column: <700
    b. Actuator function generator: <4.7, >5.3
    c. B00 function generator: = -1.41133, -1.416016, 1.362305, 1.367187
    d. B01 function generator: = -1.41133, -1.416016, 1.362305, 1.367187
8. Segment each data set at function change
9. Check synchronization with the following graphs:
    a. Actuator function with force and B00, B01
    b. Actuator deflection and string pot deflection

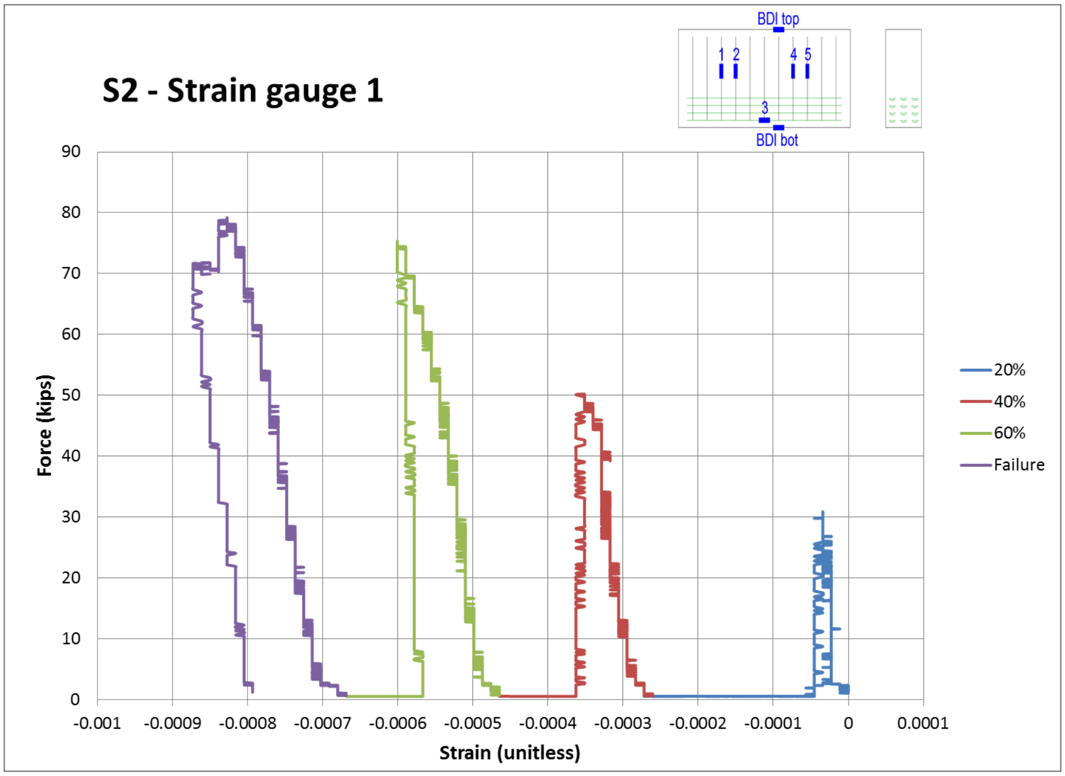
10. Count number of data points for each data set segment
11. Save file as *BeamID_sync_deleted*
12. Delete B00 and B01 data points to match actuator data started when the actuator force is at a minimum
13. Once all data is matched, copy and paste into a new sheet
14. Delete all breaks between function type
15. Add in breaks at time step jump

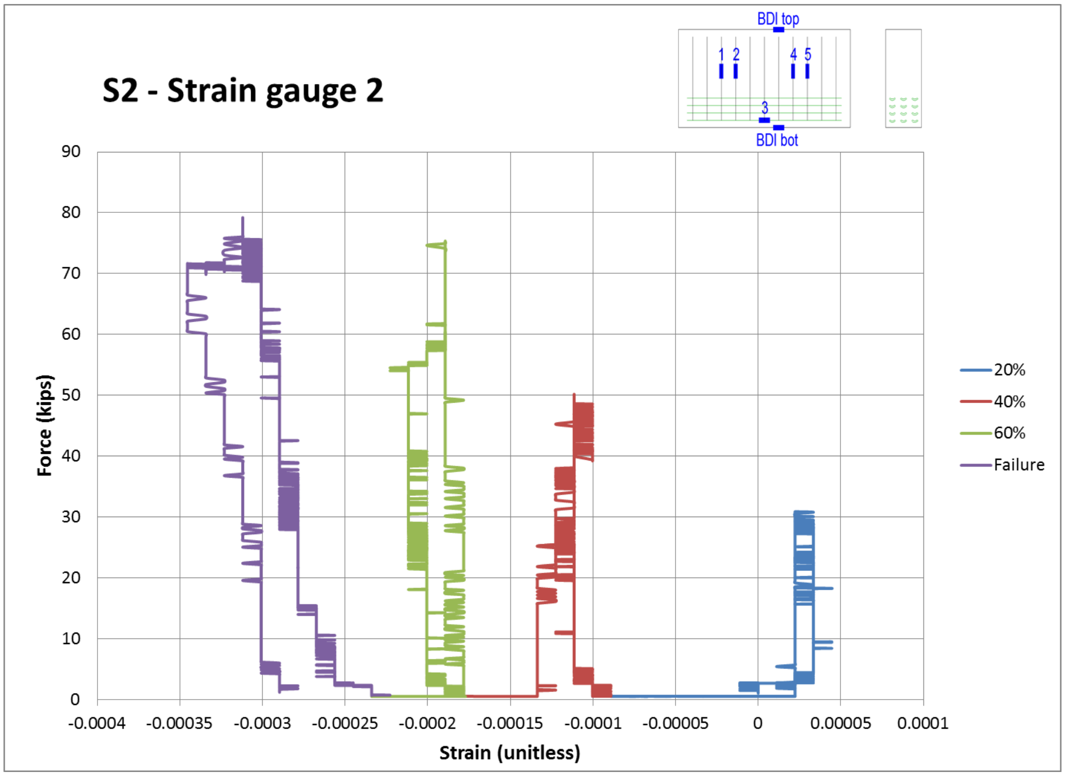
Appendix G – Strain gauge graphs

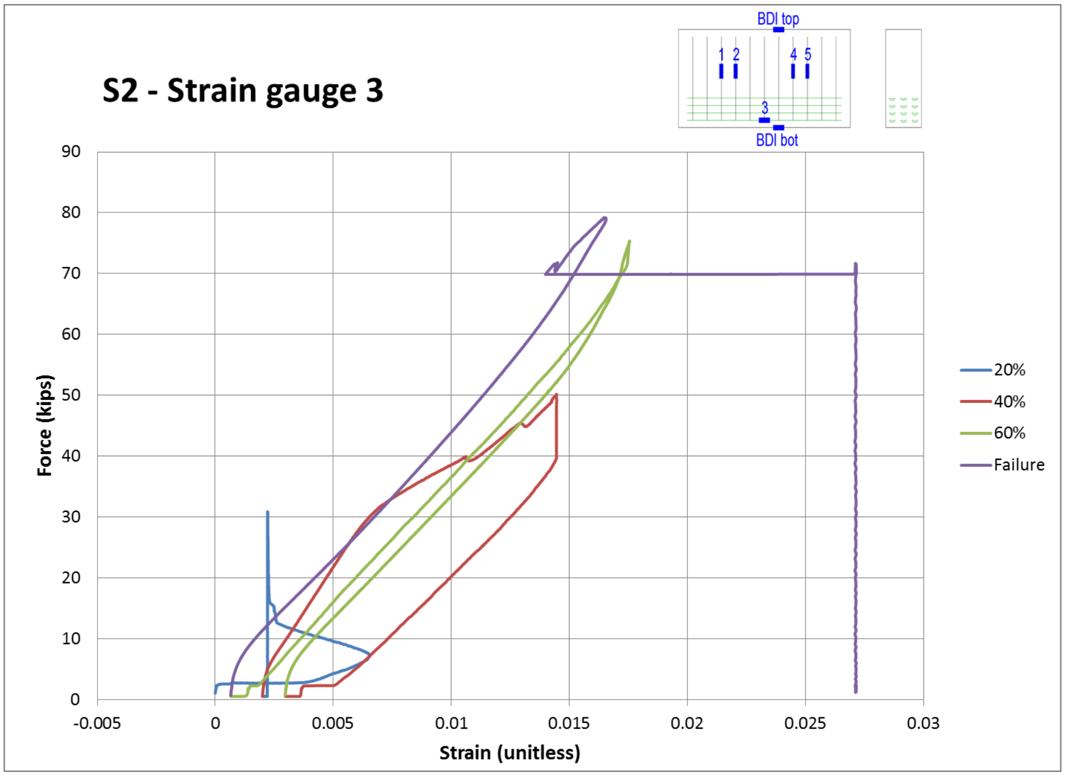
Appendix G.1 Shear beams

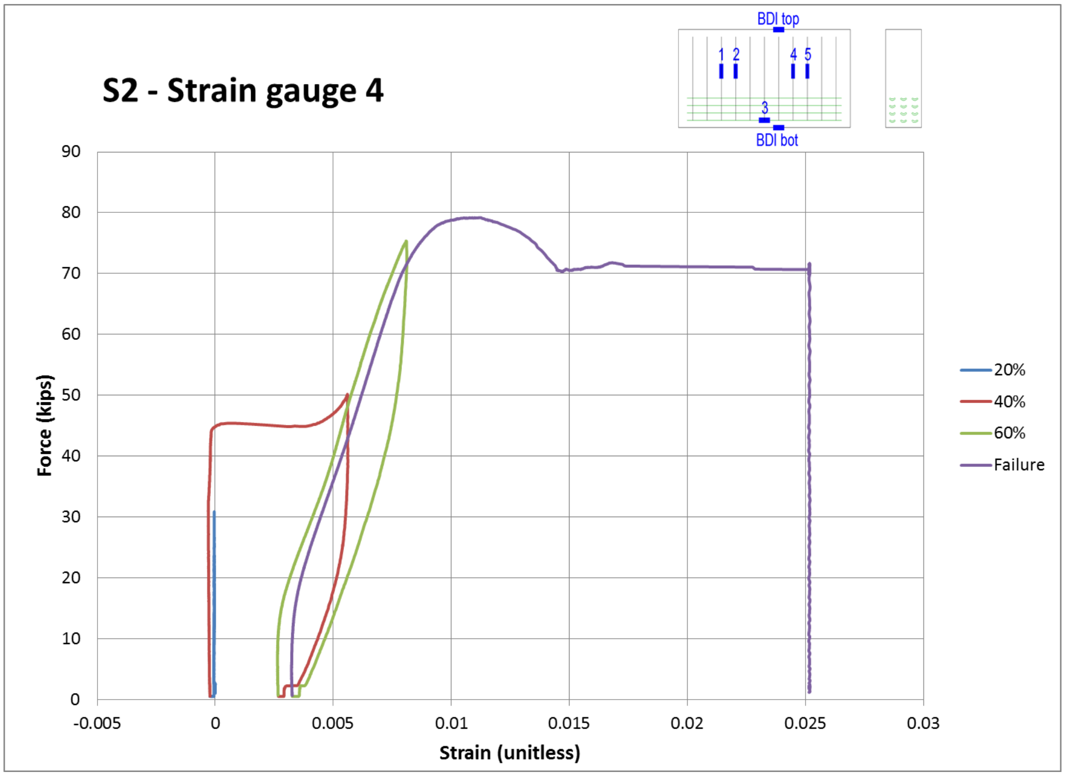


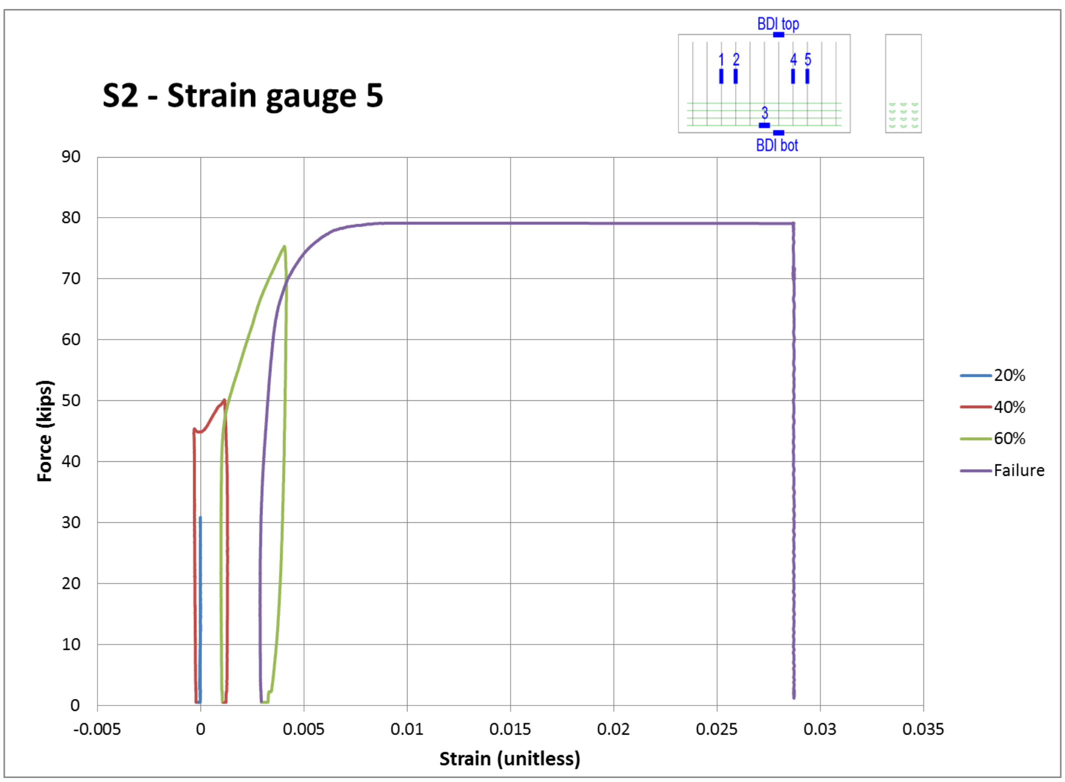

S2 - BDI bot
BDI top
1 2
4 5
3
BDI bot
Force (kips)
Strain (unitless)
90
80
70
60
50
40
30
20
10
0
-0.0005
0
0.0005
0.001
0.0015
0.002
20%
40%
60%
Failure

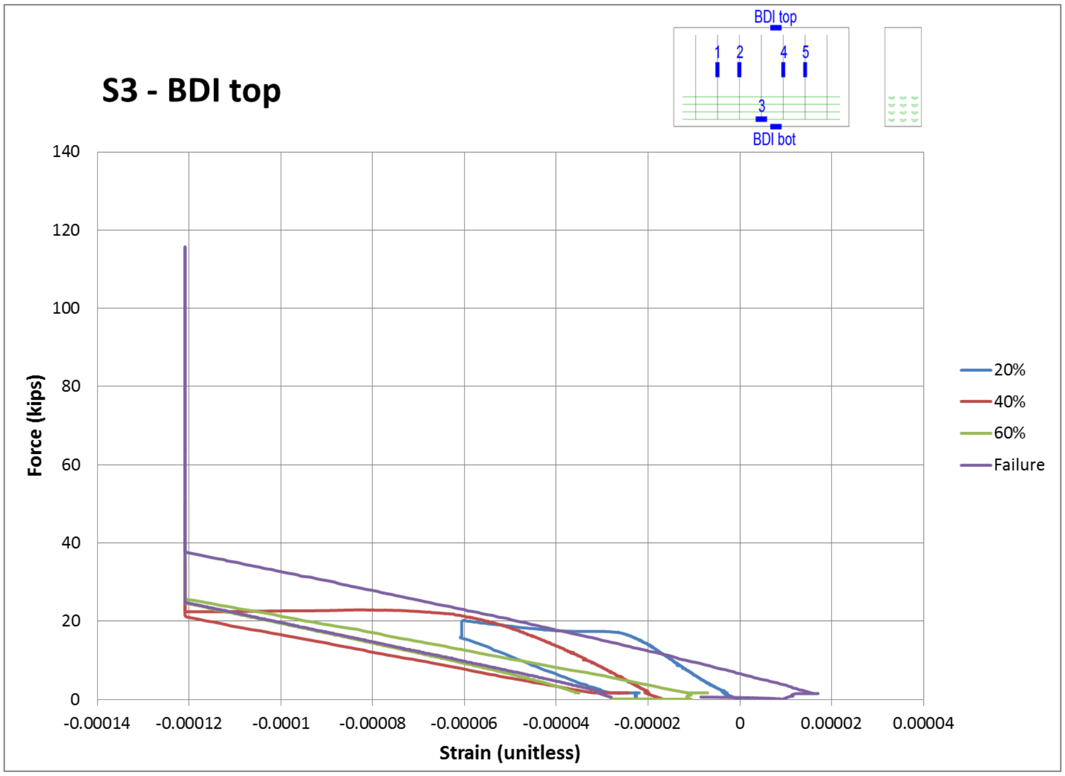

S2 - Strain gauge 1
BDI top
1 2
4 5
3
BDI bot
Force (kips)
Strain (unitless)
90
80
70
60
50
40
30
20
10
0
-0.001
-0.0009
-0.0008
-0.0007
-0.0006
-0.0005
-0.0004
-0.0003
-0.0002
-0.0001
0
0.0001
20%
40%
60%
Failure


S2 - Strain gauge 2
BDI top
1 2
4 5
3
BDI bot
Force (kips)
90
80
70
60
50
40
30
20
10
0
-0.0004
-0.00035
-0.0003
-0.00025
-0.0002
-0.00015
-0.0001
-0.00005
0
0.00005
0.0001
Strain (unitless)
20%
40%
60%
Failure

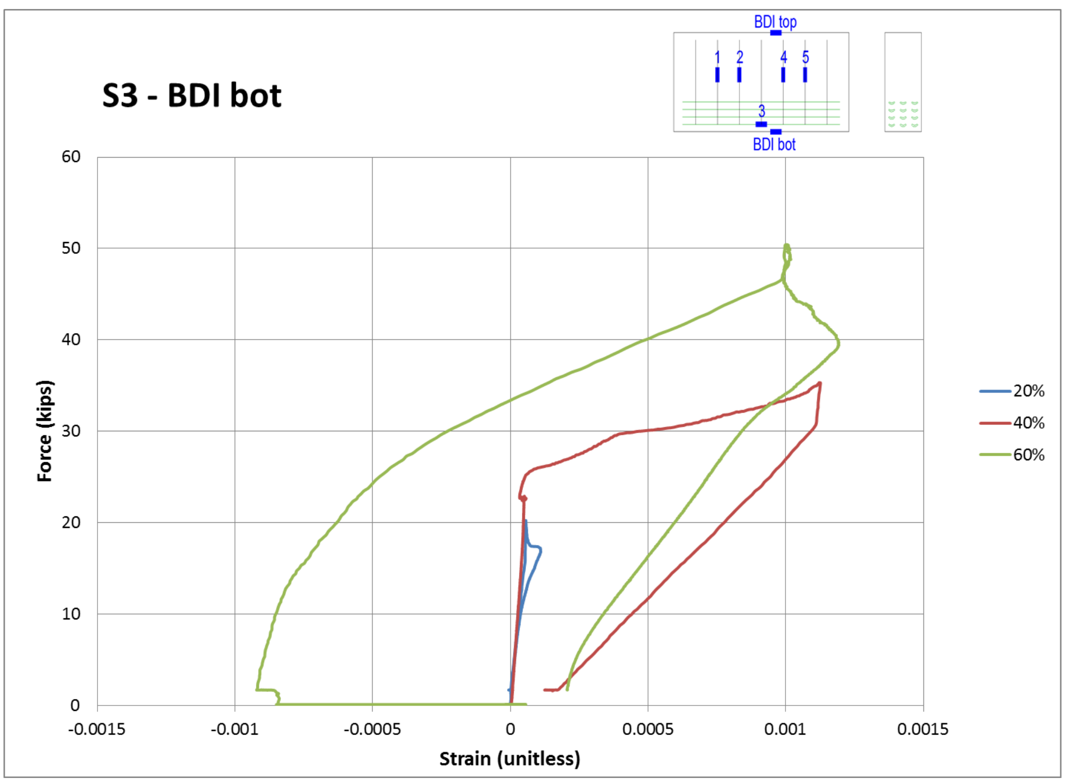

S2 - Strain gauge 3
BDI top
1 2
4 5
3
BDI bot
Force (kips)
90
80
70
60
50
40
30
20
10
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
Strain (unitless)
20%
40%
60%
Failure


S2 - Strain gauge 4
BDI top
1 2
4 5
3
BDI bot
90
80
70
60
50
40
30
20
10
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

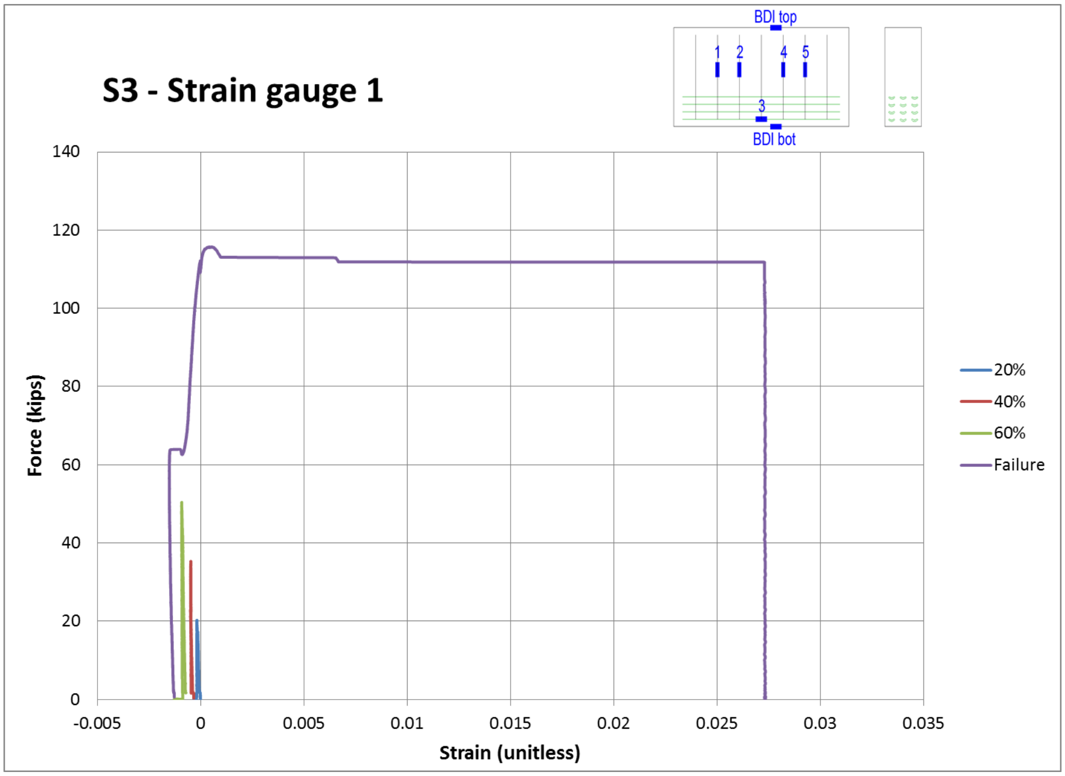

S2 - Strain gauge 5
BDI top
1 2
4 5
3
BDI bot
90
80
70
60
50
40
30
20
10
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

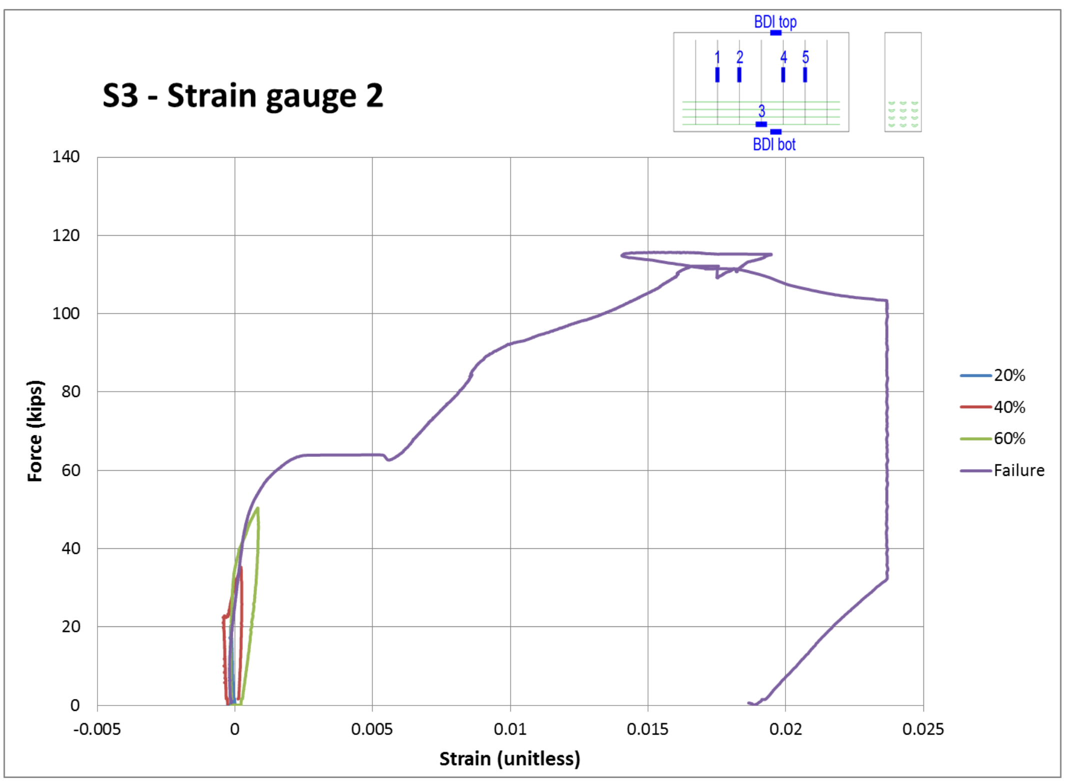


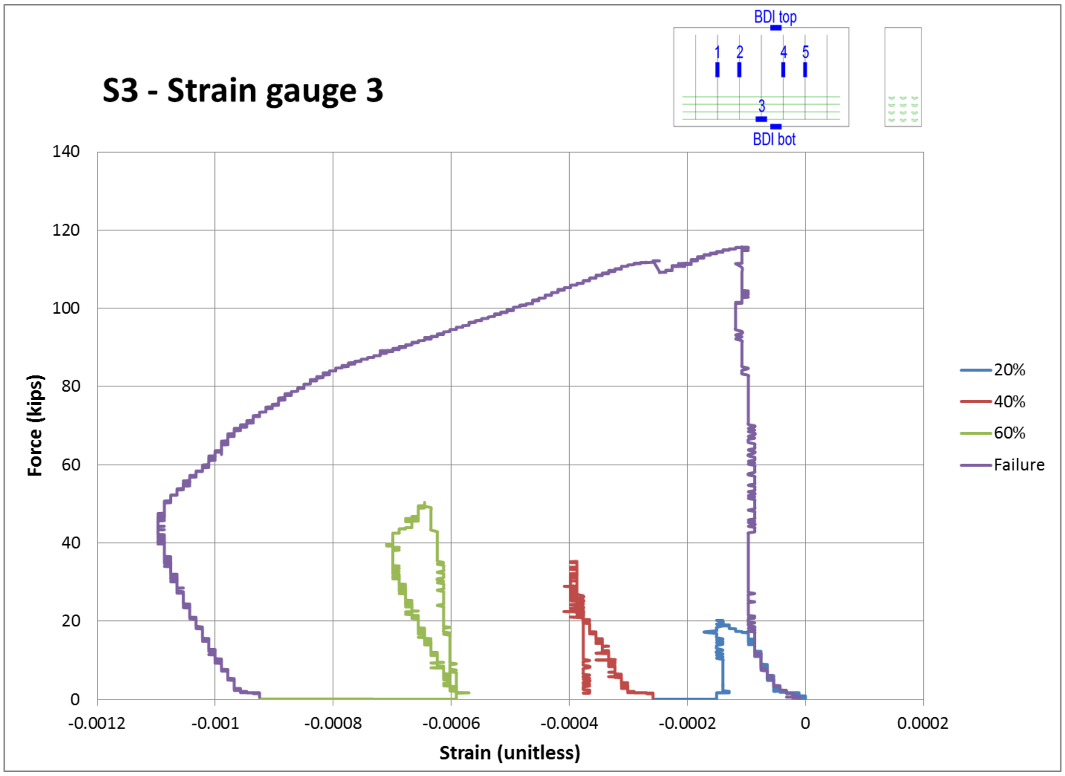
S3 - BDI top
BDI top
1
2
4
5
3
BDI bot
140
120
100
80
60
40
20
0
Force (kips)
-0.00014
-0.00012
-0.0001
-0.00008
-0.00006
-0.00004
-0.00002
0
0.00002
0.00004
Strain (unitless)
20%
40%
60%
Failure

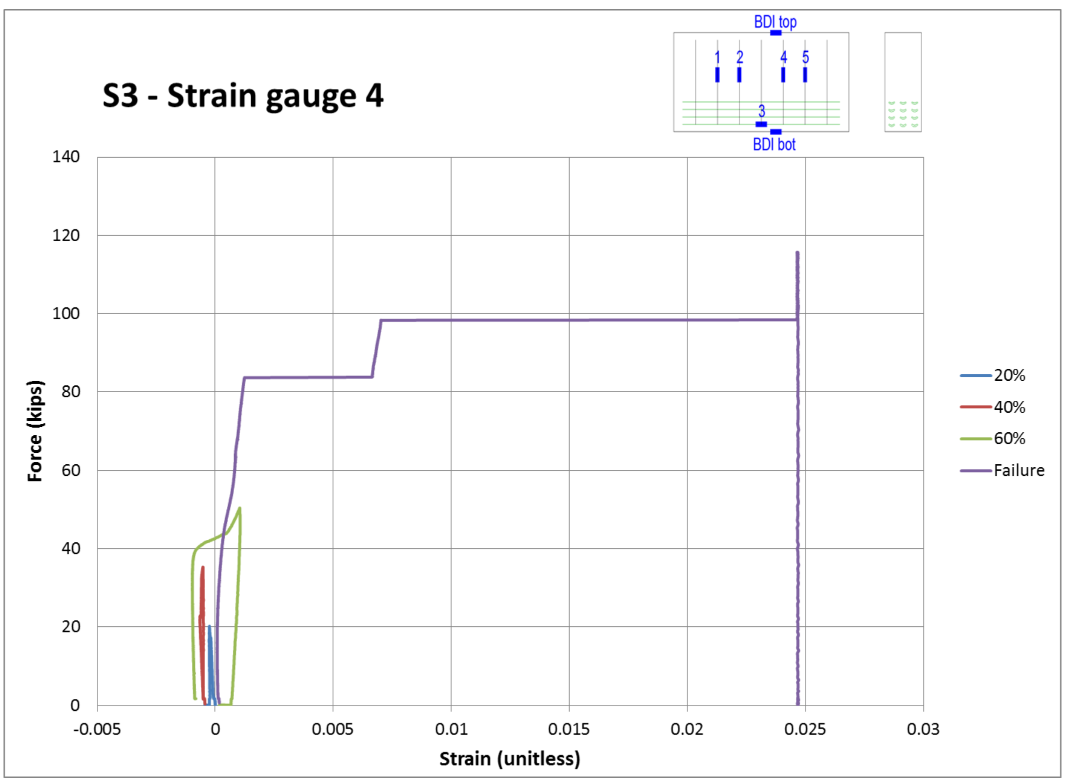


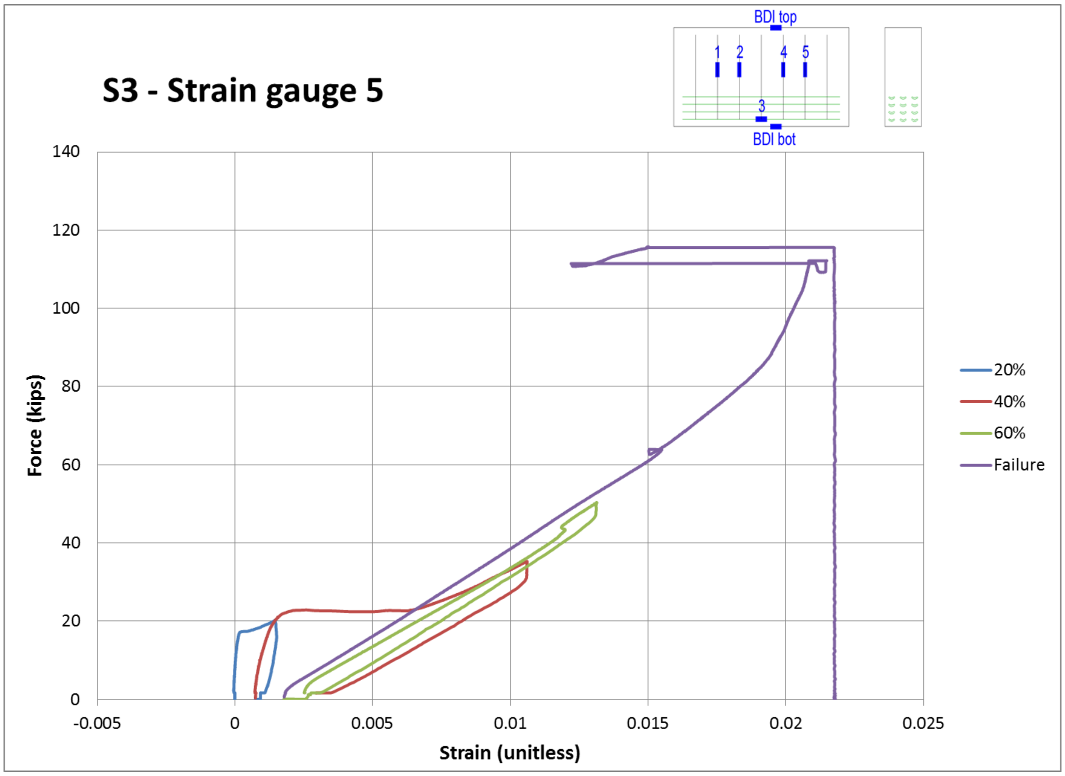
S3 - BDI bot
BDI top
1
2
4
5
3
BDI bot
60
50
40
30
20
10
0
Force (kips)
-0.0015
-0.001
-0.0005
0
0.0005
0.001
0.0015
Strain (unitless)
20%
40%
60%

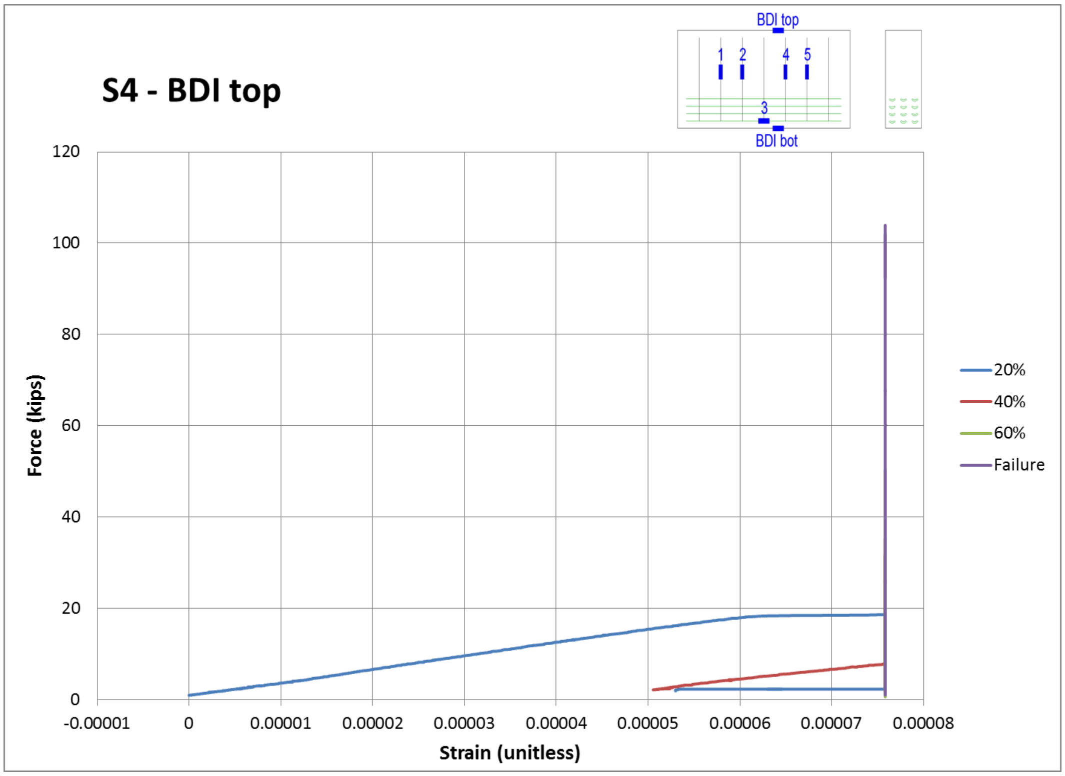

S3 - Strain gauge 1
BDI top
1
2
4
5
3
BDI bot
140
120
100
80
60
40
20
0
Force (kips)
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
0.035
Strain (unitless)
20%
40%
60%
Failure

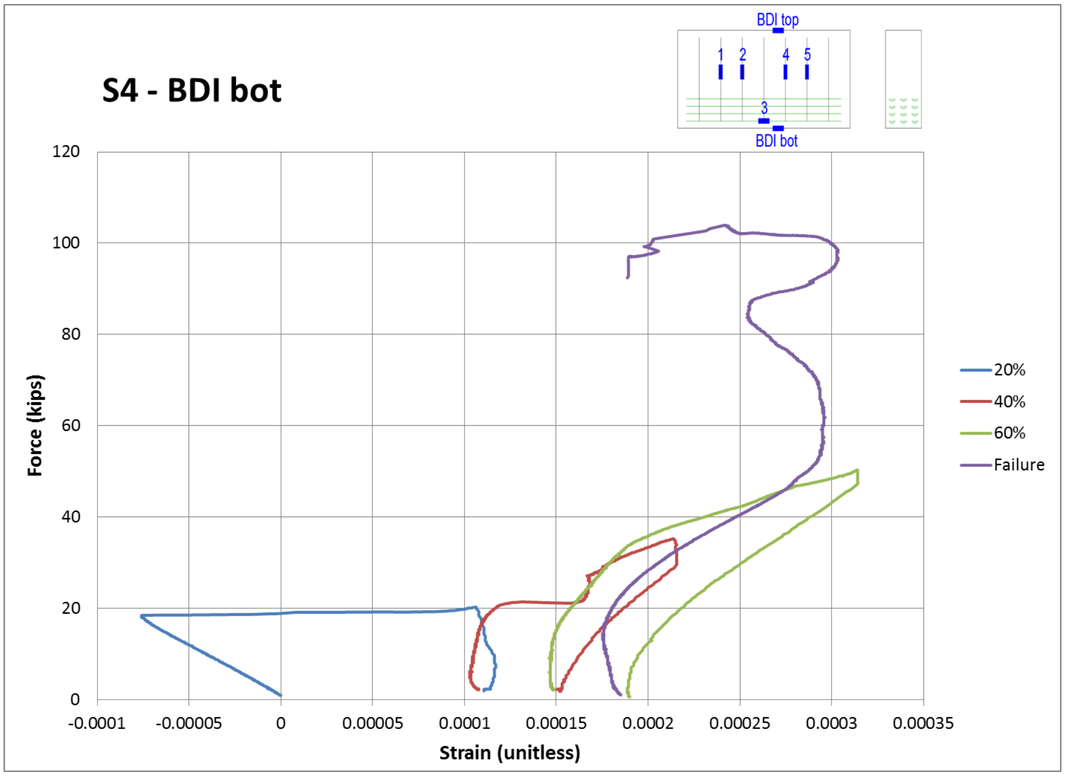

S3 - Strain gauge 2
BDI top
1
2
4
5
3
BDI bot
140
120
100
80
60
40
20
0
Force (kips)
-0.005
0
0.005
0.01
0.015
0.02
0.025
Strain (unitless)
20%
40%
60%
Failure

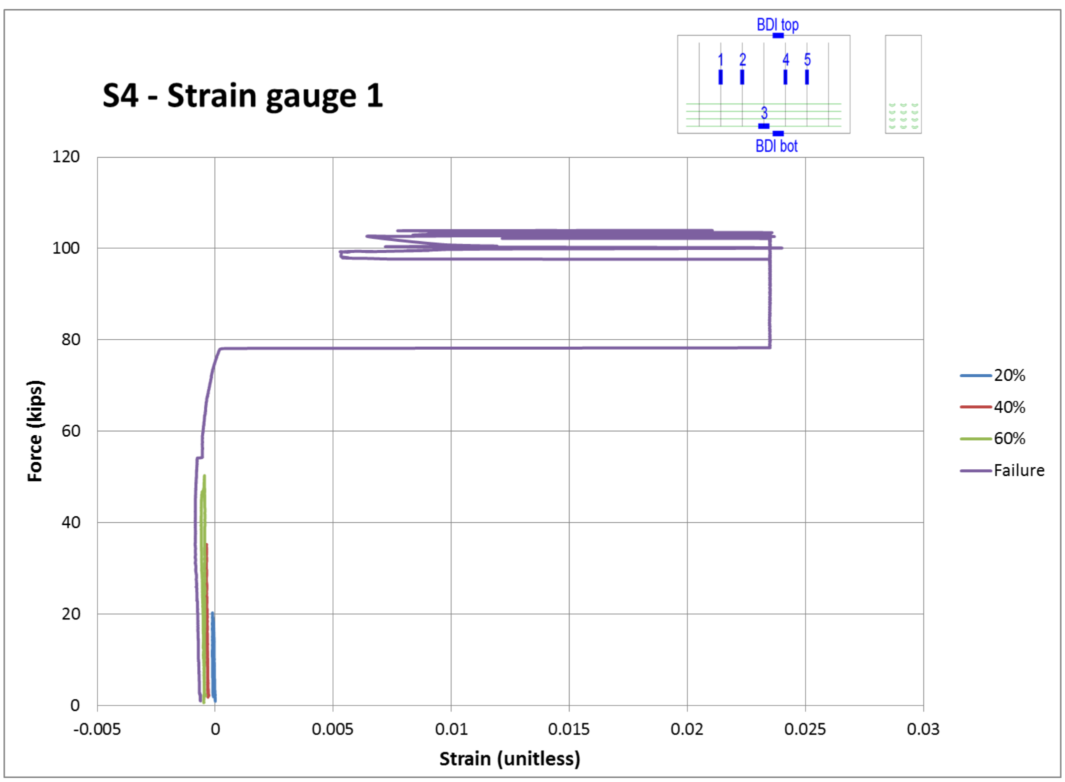

S3 - Strain gauge 3
BDI top
1 2 4 5
3
BDI bot
140
120
100
80
60
40
20
0
-0.0012
-0.001
-0.0008
-0.0006
-0.0004
-0.0002
0
0.0002
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

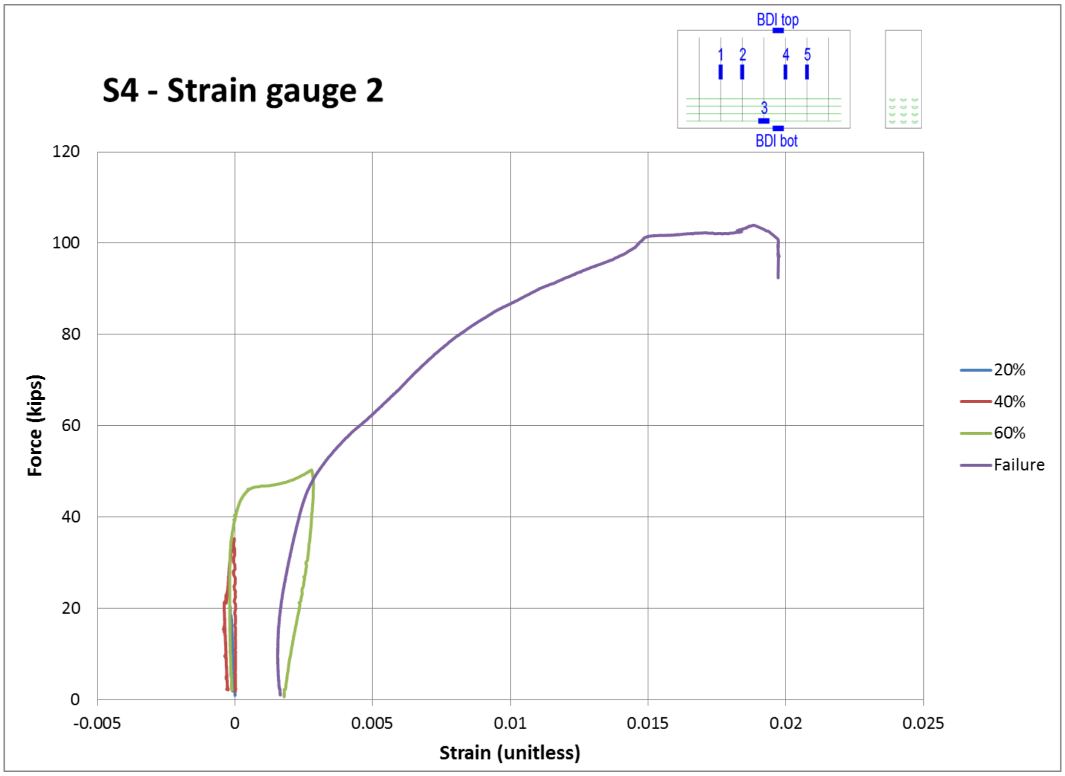

S3 - Strain gauge 4
BDI top
1 2 4 5
3
BDI bot
140
120
100
80
60
40
20
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

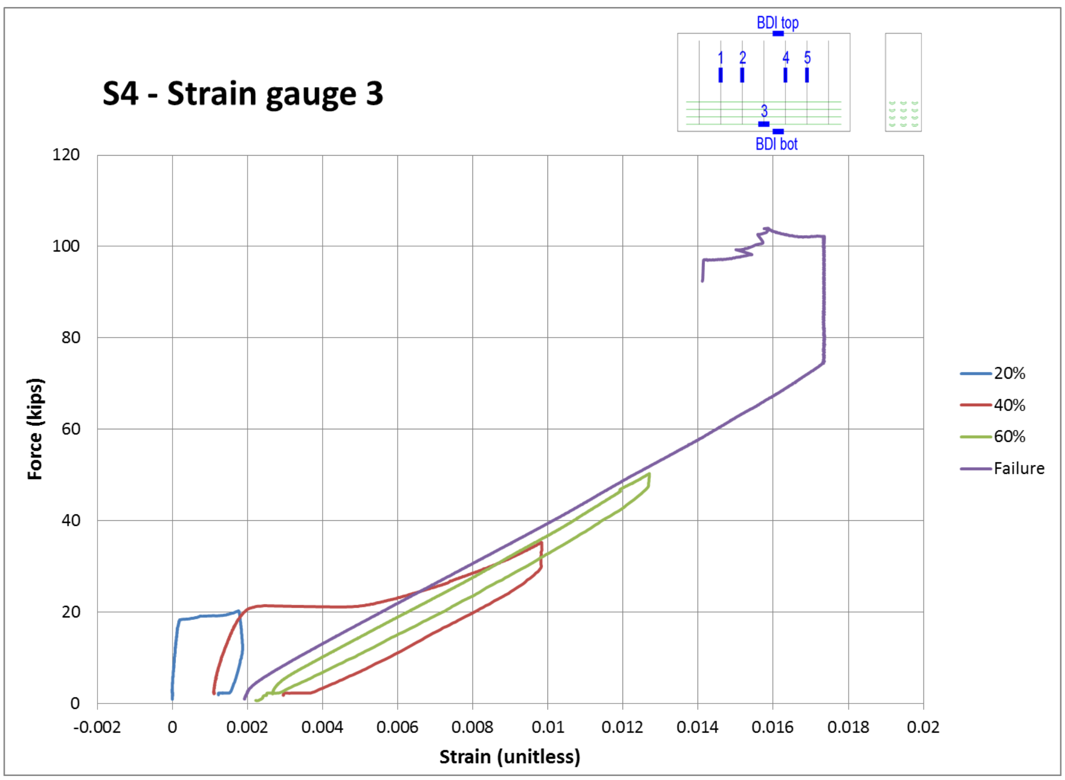

S3 - Strain gauge 5
BDI top
1
2
4
5
3
BDI bot
Force (kips)
140
120
100
80
60
40
20
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
Strain (unitless)
20%
40%
60%
Failure

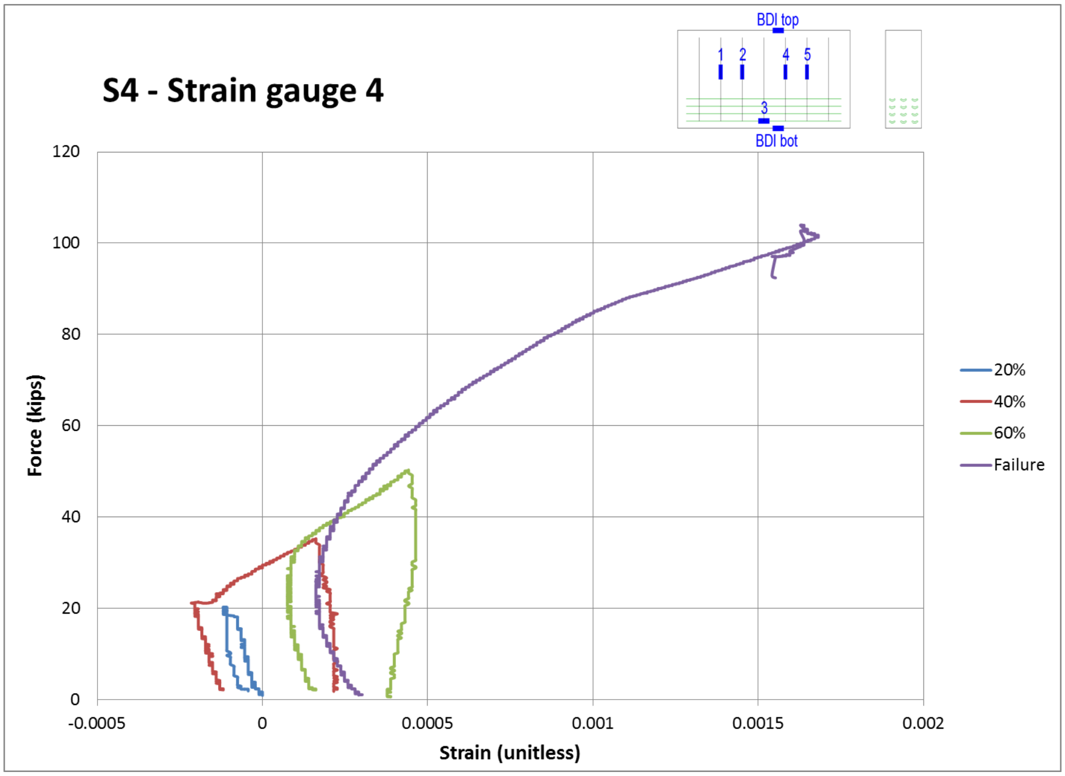

S4 - BDI top
BDI top
1
2
4
5
3
BDI bot
Force (kips)
120
100
80
60
40
20
0
-0.00001
0
0.00001
0.00002
0.00003
0.00004
0.00005
0.00006
0.00007
0.00008
Strain (unitless)
20%
40%
60%
Failure

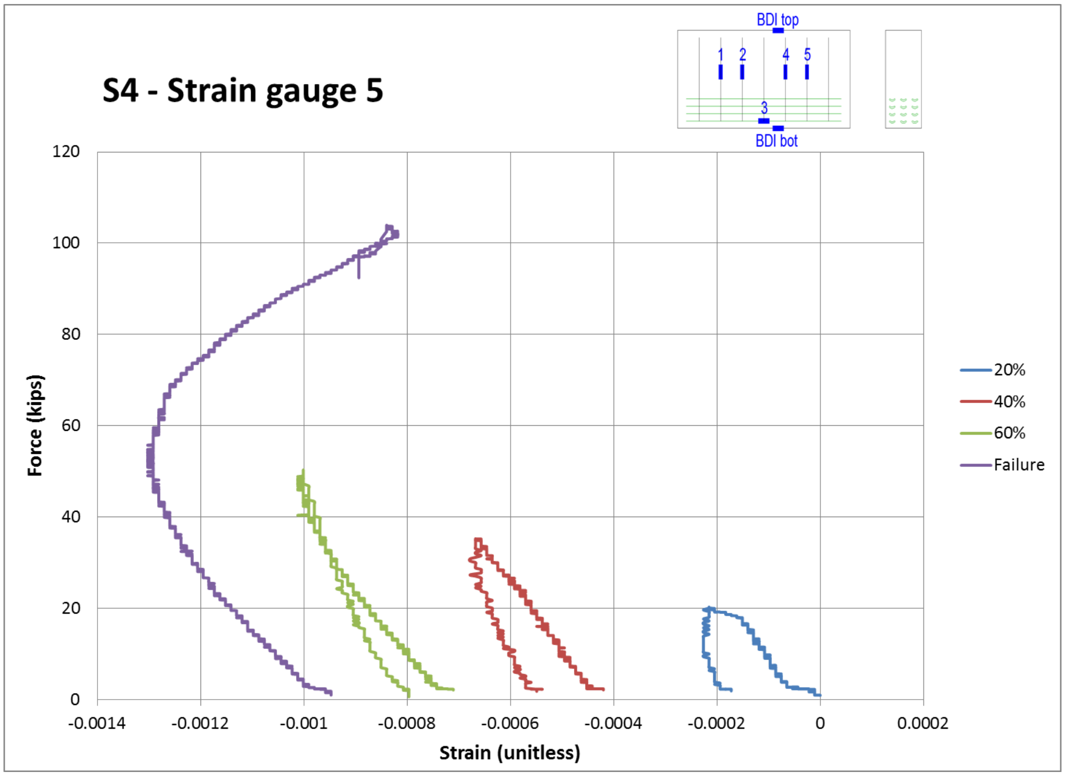

S4 - BDI bot
BDI top
1
2
4
5
3
BDI bot
Force (kips)
120
100
80
60
40
20
0
-0.0001
-0.00005
0
0.00005
0.0001
0.00015
0.0002
0.00025
0.0003
0.00035
Strain (unitless)
20%
40%
60%
Failure

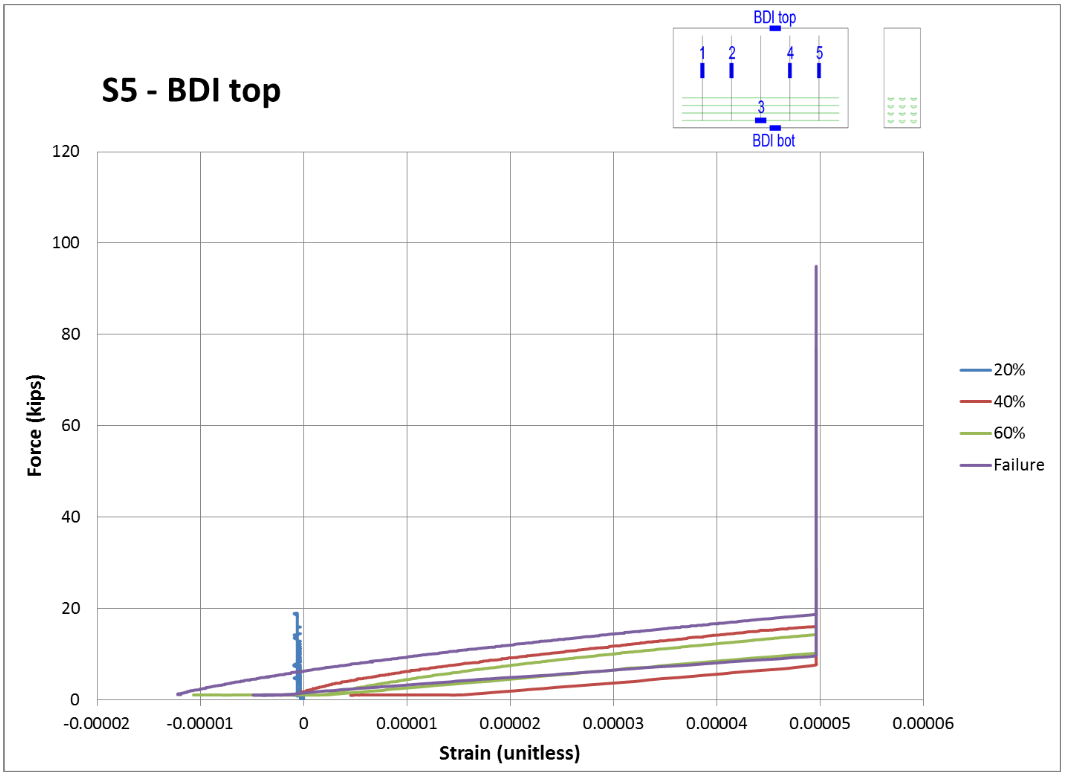

S4 - Strain gauge 1
BDI top
1
2
4
5
3
BDI bot
Force (kips)
120
100
80
60
40
20
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
Strain (unitless)
20%
40%
60%
Failure

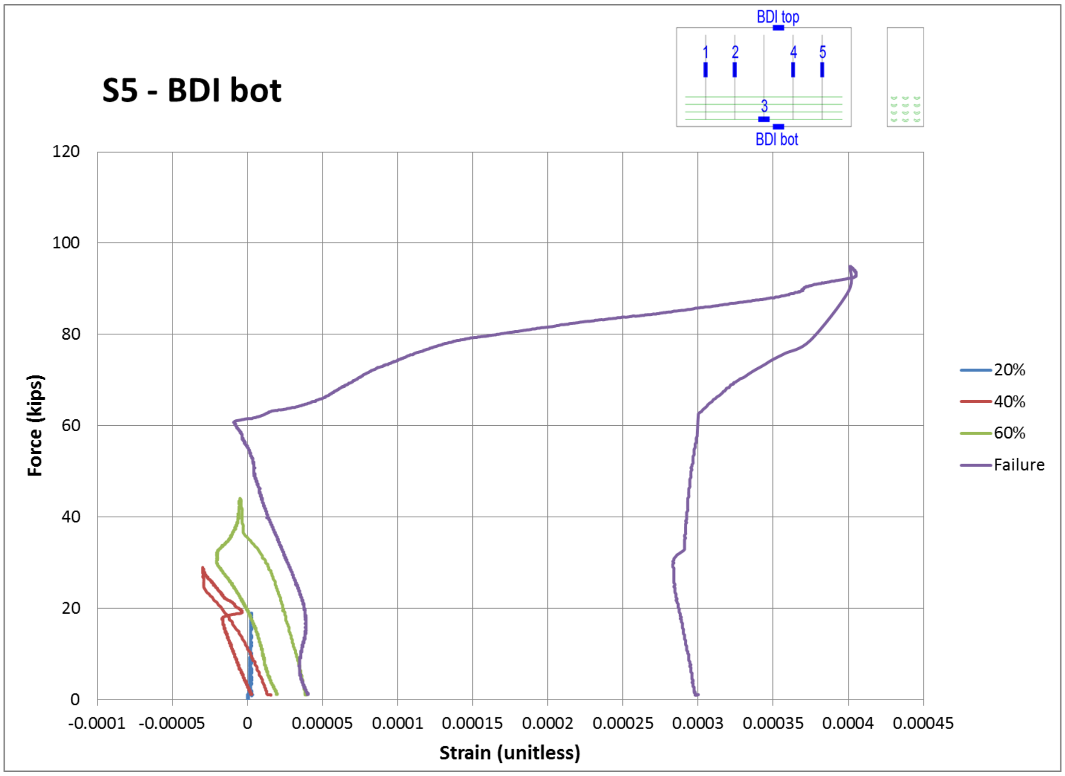

S4 - Strain gauge 2
BDI top
1
2
4
5
3
BDI bot
Force (kips)
Strain (unitless)
120
100
80
60
40
20
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
20%
40%
60%
Failure

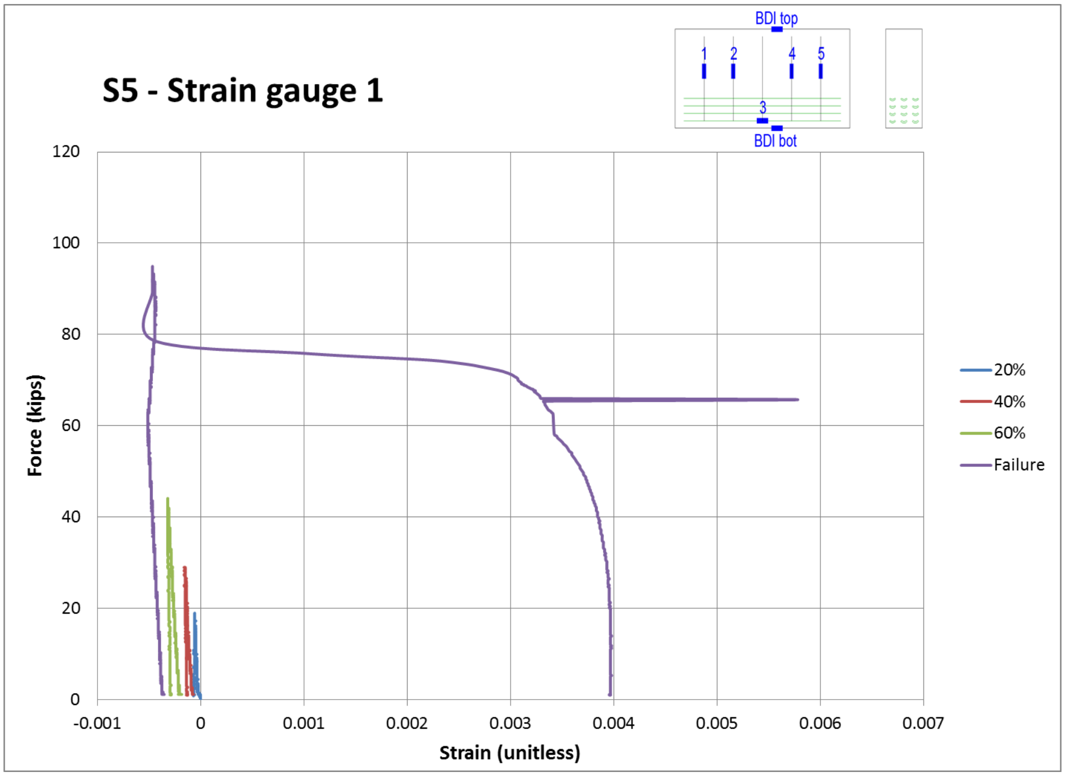

S4 - Strain gauge 3
BDI top
1
2
4
5
3
BDI bot
Force (kips)
Strain (unitless)
120
100
80
60
40
20
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02
20%
40%
60%
Failure

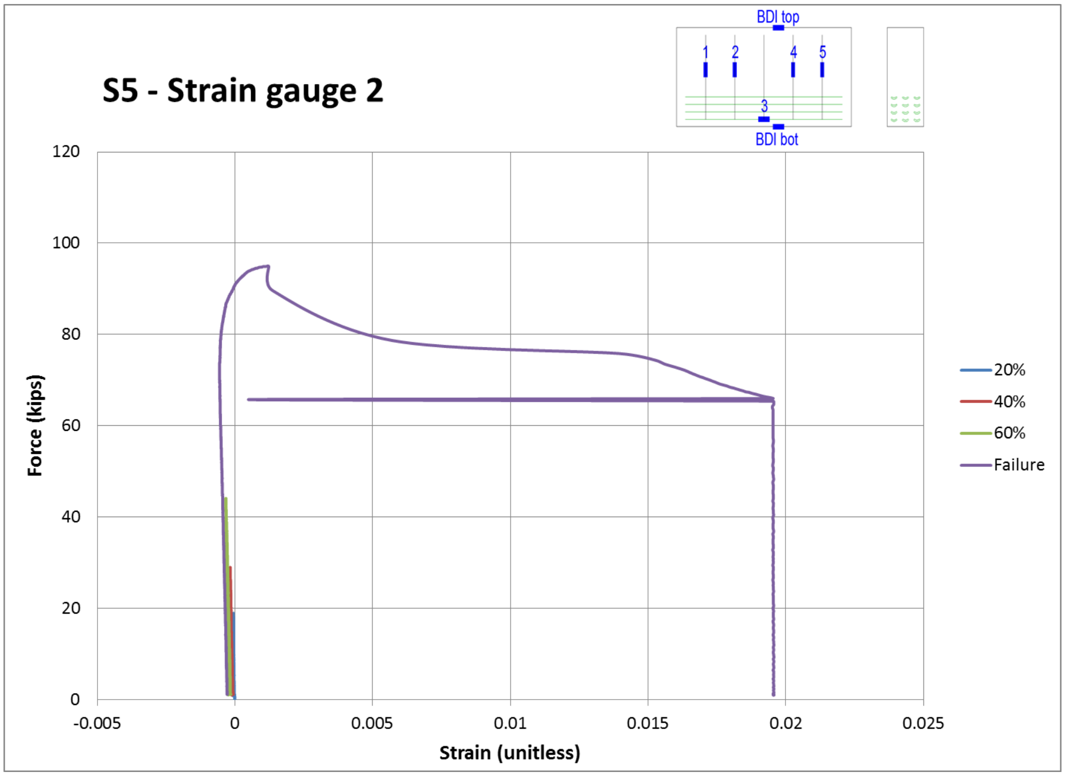

S4 - Strain gauge 4
BDI top
1 2 4 5
3
BDI bot
120
100
80
60
40
20
0
-0.0005
0
0.0005
0.001
0.0015
0.002
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

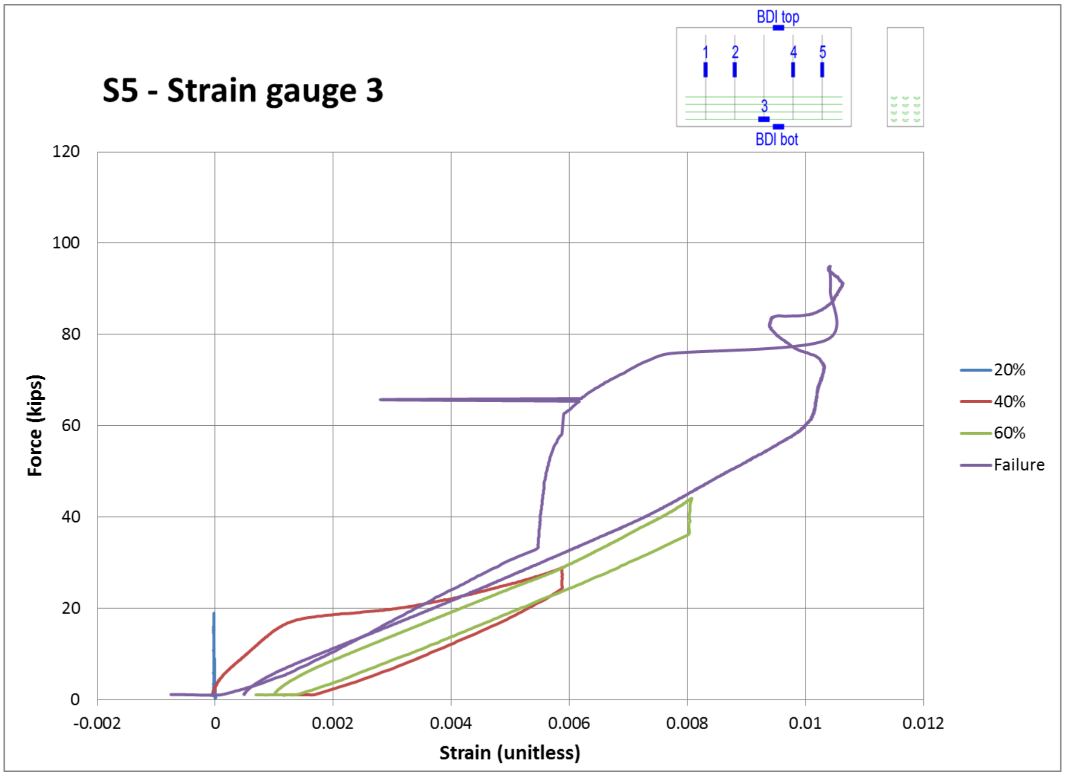

S4 - Strain gauge 5
BDI top
1 2 4 5
3
BDI bot
120
100
80
60
40
20
0
-0.0014
-0.0012
-0.001
-0.0008
-0.0006
-0.0004
-0.0002
0
0.0002
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

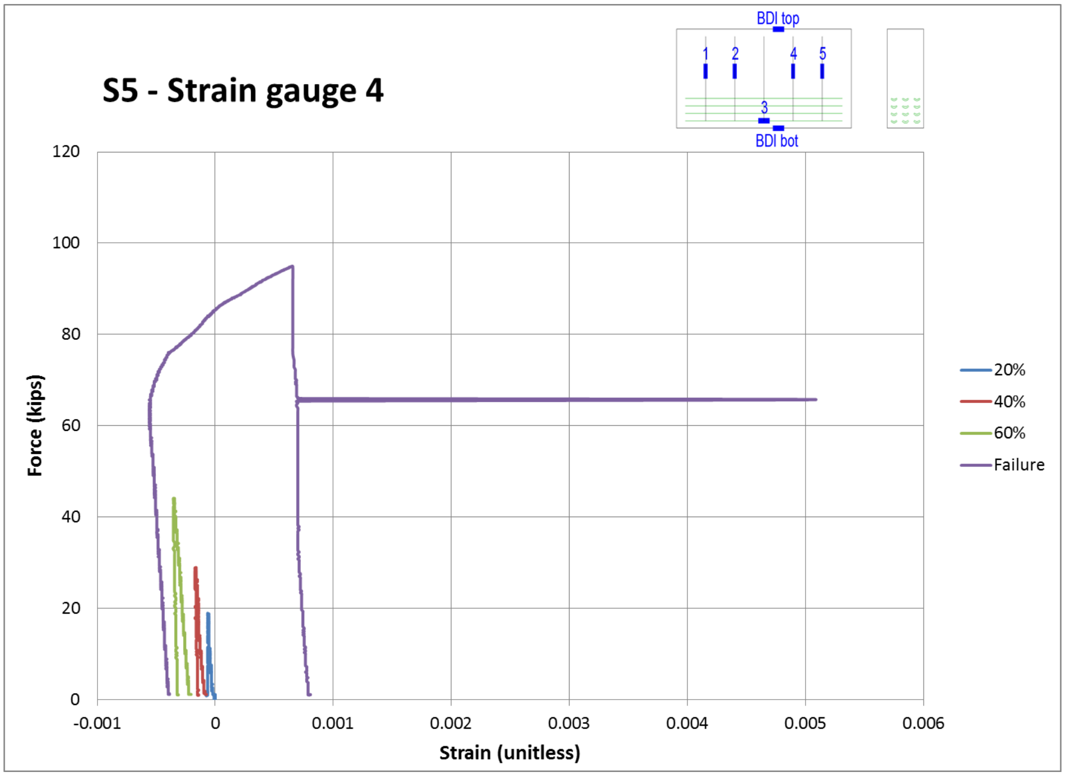

S5 - BDI top
BDI top
1
2
4
5
3
BDI bot
Force (kips)
120
100
80
60
40
20
0
-0.00002
-0.00001
0
0.00001
0.00002
0.00003
0.00004
0.00005
0.00006
Strain (unitless)
20%
40%
60%
Failure

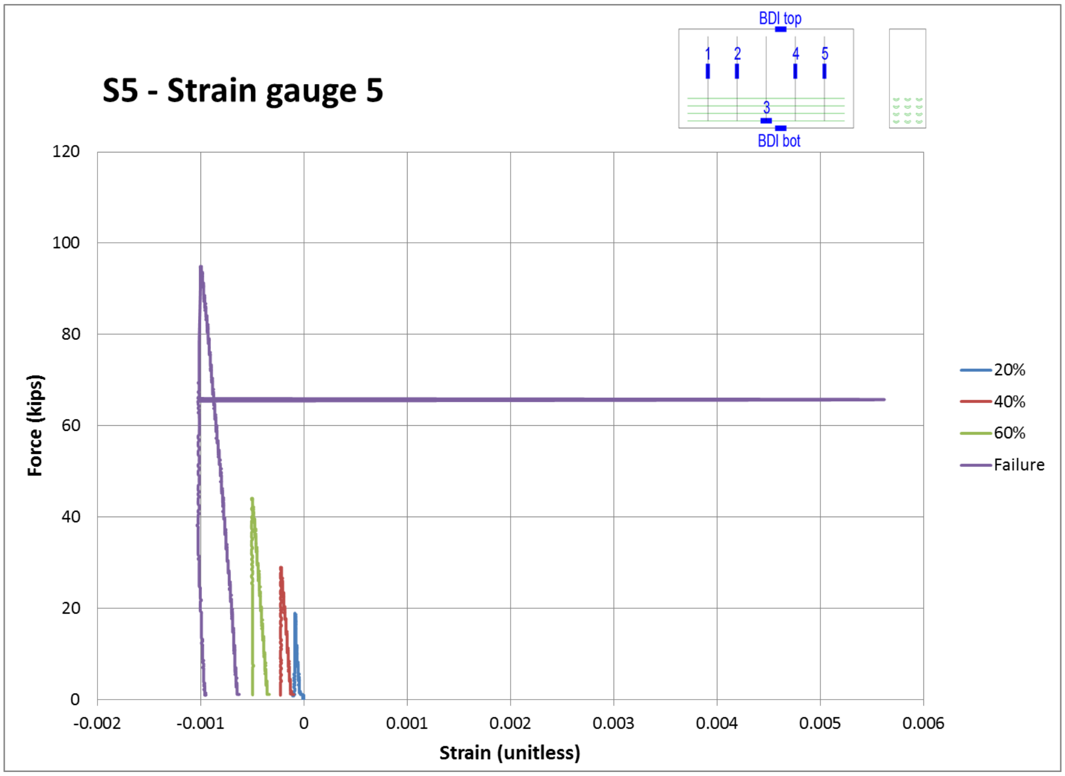

S5 - BDI bot
BDI top
1
2
4
5
3
BDI bot
Force (kips)
120
100
80
60
40
20
0
-0.0001
-0.00005
0
0.00005
0.0001
0.00015
0.0002
0.00025
0.0003
0.00035
0.0004
0.00045
Strain (unitless)
20%
40%
60%
Failure

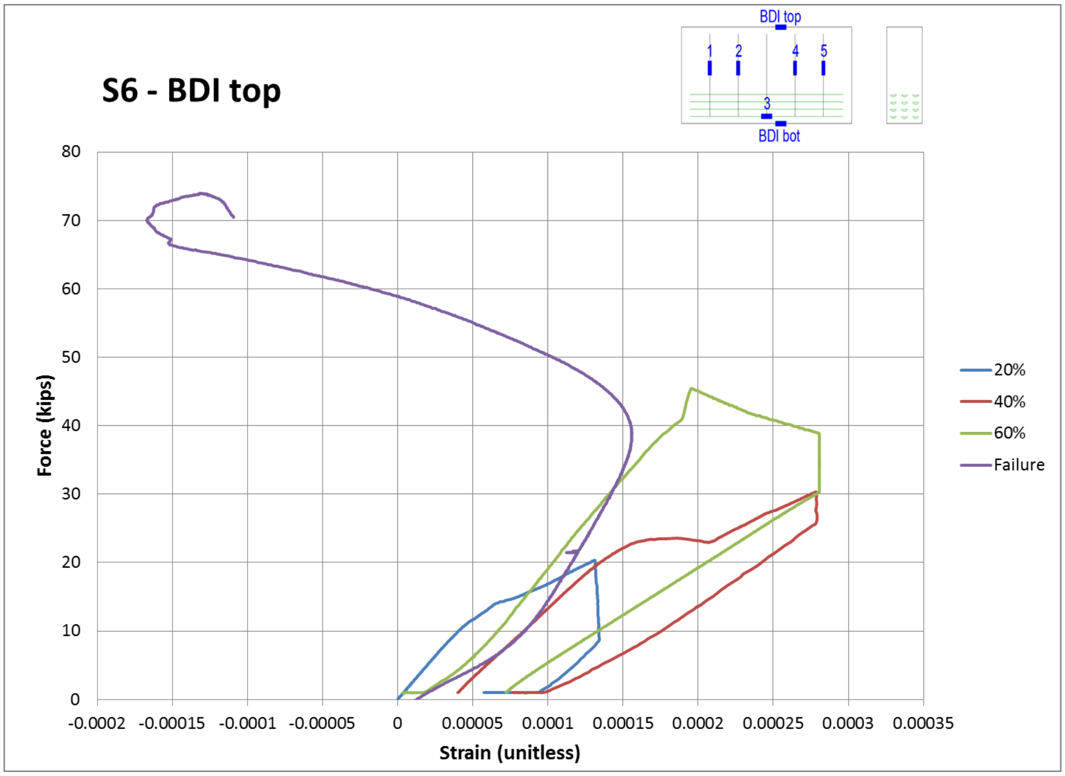

S5 - Strain gauge 1
BDI top
1
2
4
5
3
BDI bot
120
100
80
60
40
20
0
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
0.007
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

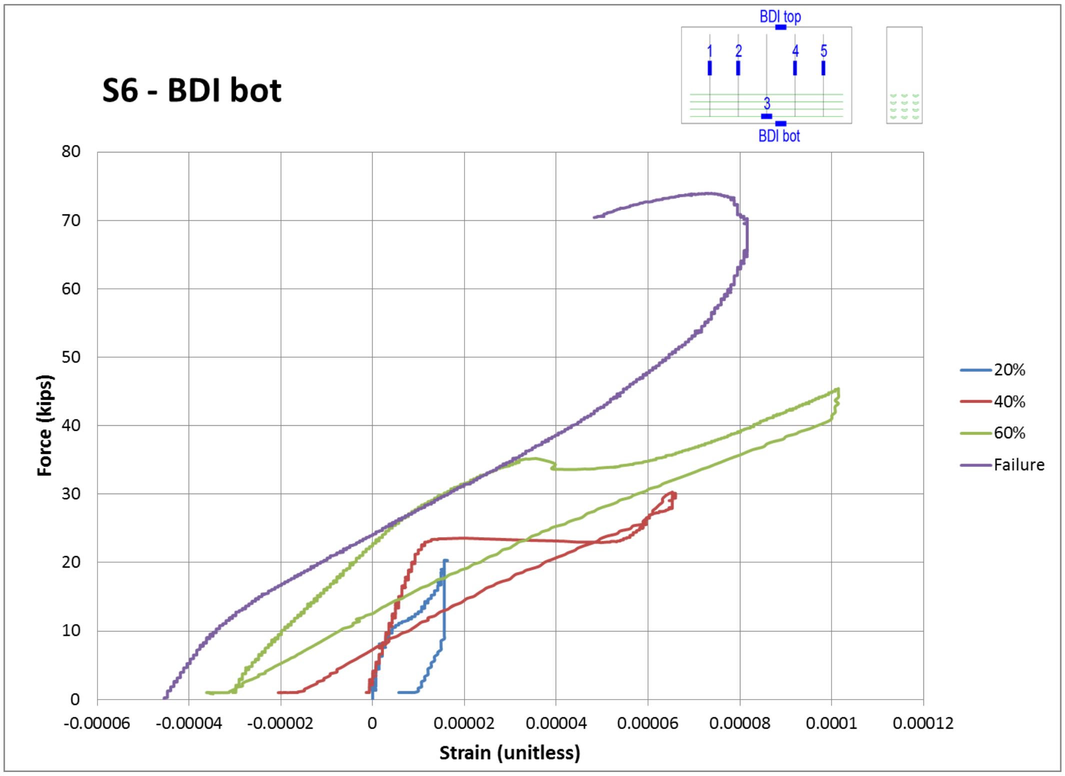

S5 - Strain gauge 2
BDI top
1
2
4
5
3
BDI bot
120
100
80
60
40
20
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

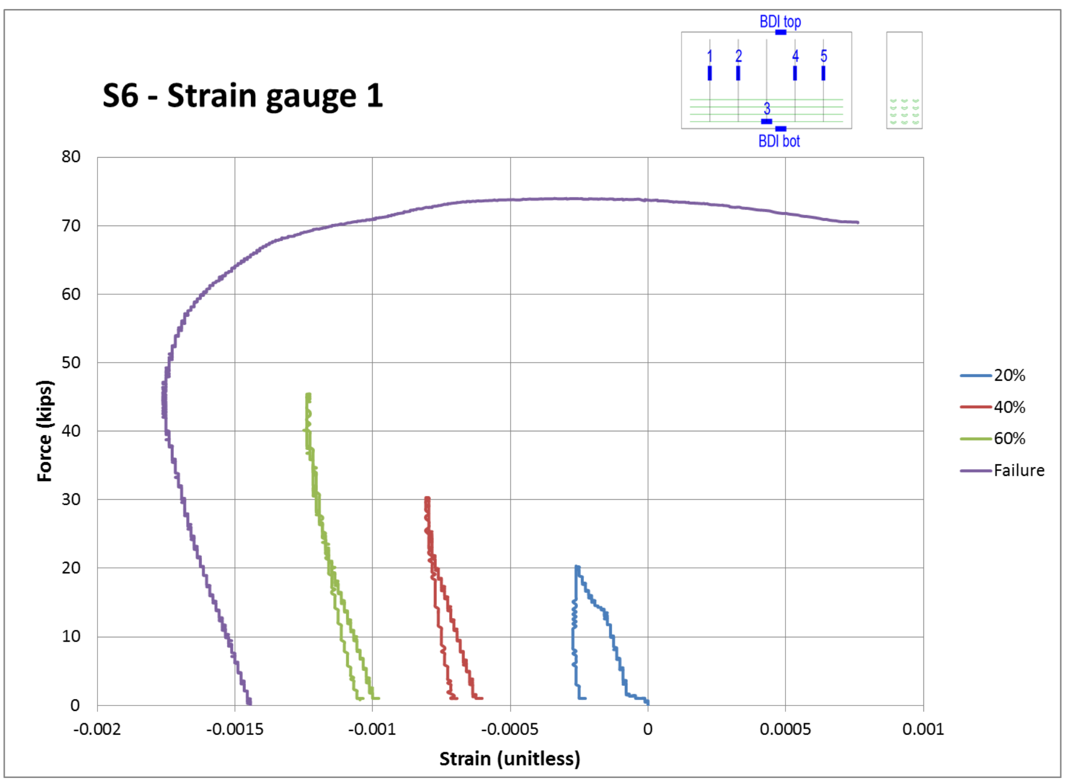

S5 - Strain gauge 3
BDI top
1
2
4
5
3
BDI bot
Force (kips)
120
100
80
60
40
20
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
Strain (unitless)
20%
40%
60%
Failure

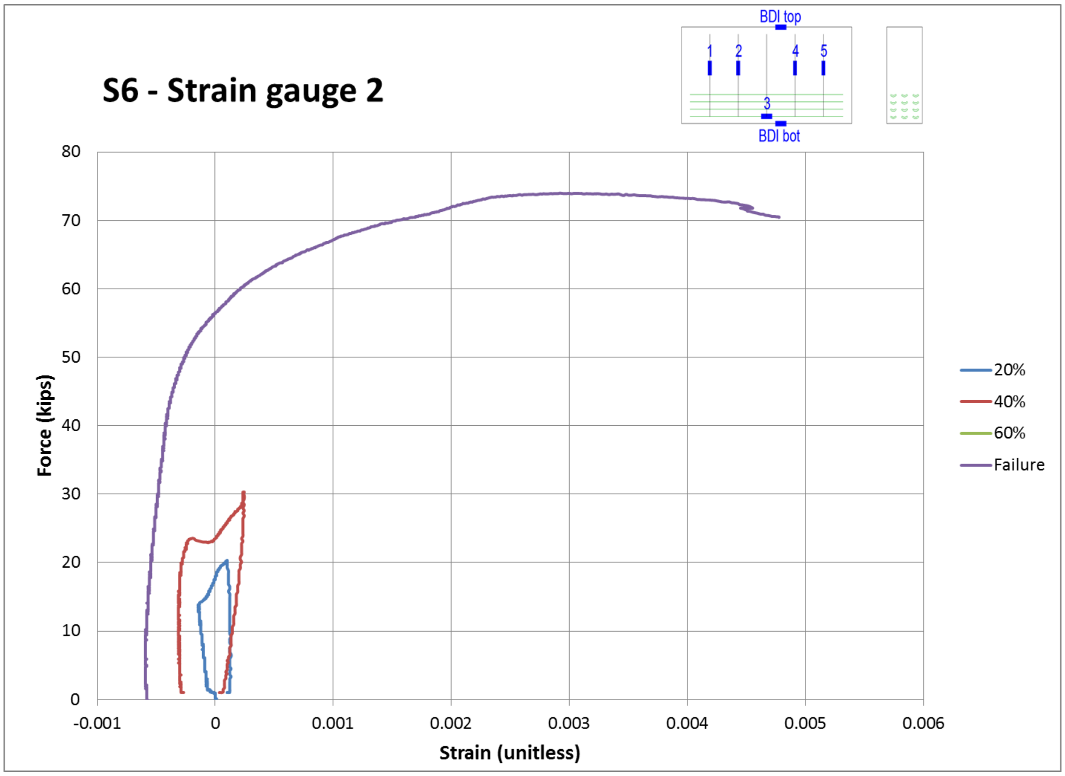

S5 - Strain gauge 4
BDI top
1
2
4
5
3
BDI bot
Force (kips)
120
100
80
60
40
20
0
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
Strain (unitless)
20%
40%
60%
Failure

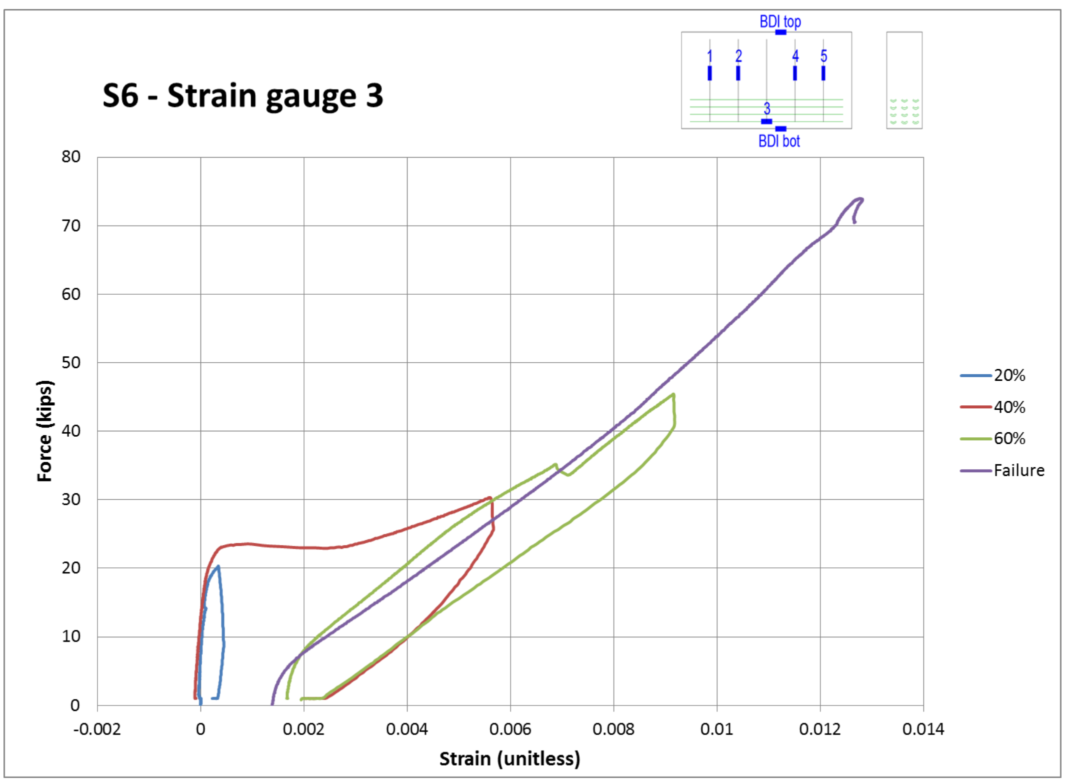

S5 - Strain gauge 5
BDI top
1
2
3
4
5
BDI bot
Force (kips)
Strain (unitless)
120
100
80
60
40
20
0
-0.002
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
20%
40%
60%
Failure

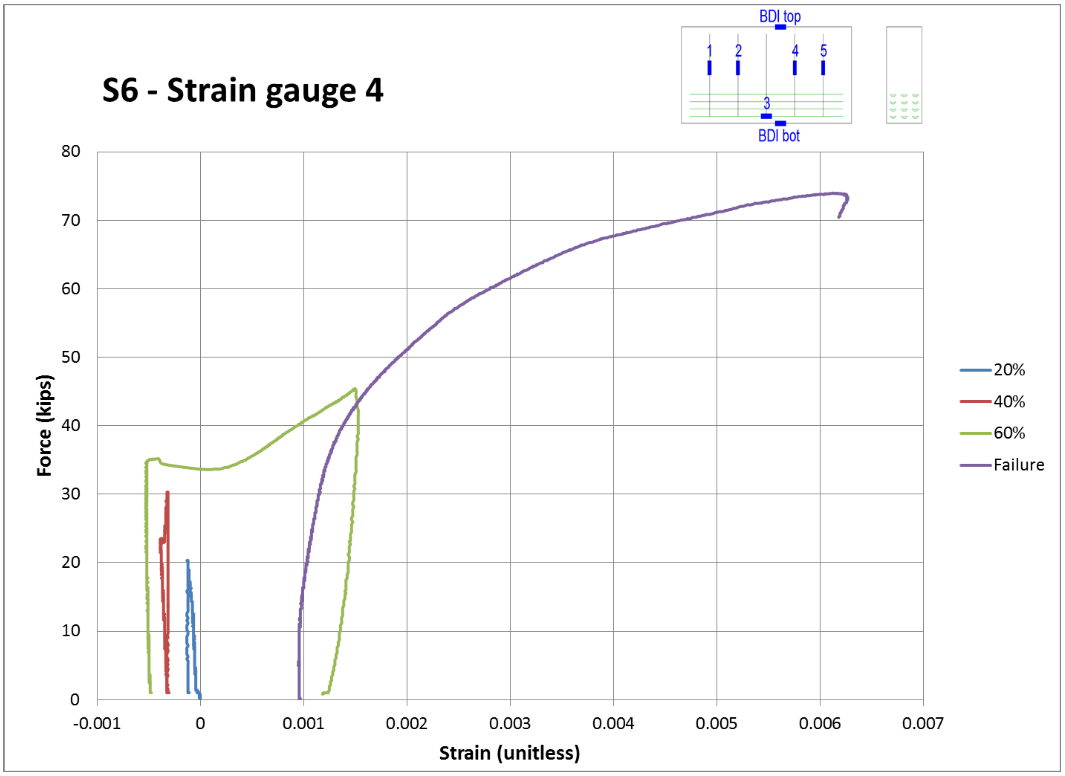

S6 - BDI top
BDI top
1
2
3
4
5
BDI bot
Force (kips)
Strain (unitless)
80
70
60
50
40
30
20
10
0
-0.0002
-0.00015
-0.0001
-0.00005
0
0.00005
0.0001
0.00015
0.0002
0.00025
0.0003
0.00035
20%
40%
60%
Failure

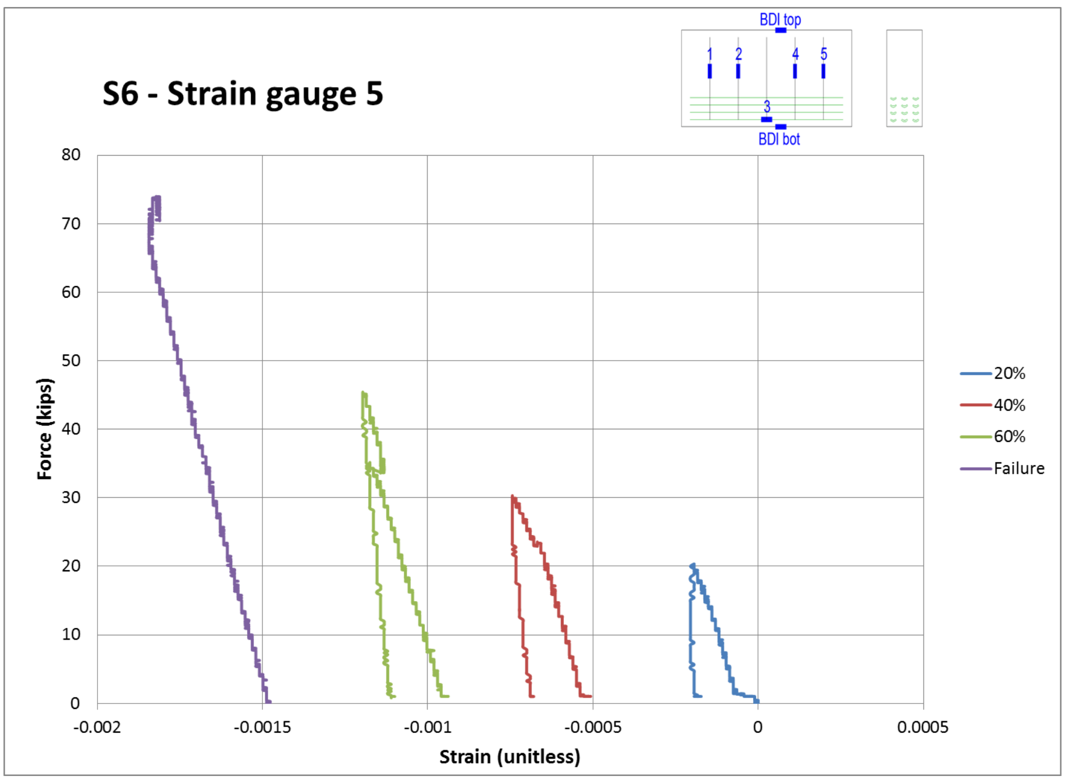

S6 - BDI bot
BDI top
1
2
3
4
5
BDI bot
Force (kips)
Strain (unitless)
80
70
60
50
40
30
20
10
0
-0.00006
-0.00004
-0.00002
0
0.00002
0.00004
0.00006
0.00008
0.0001
0.00012
20%
40%
60%
Failure


S6 - Strain gauge 1
BDI top
1
2
3
4
5
BDI bot
Force (kips)
Strain (unitless)
80
70
60
50
40
30
20
10
0
-0.002
-0.0015
-0.001
-0.0005
0
0.0005
0.001
20%
40%
60%
Failure

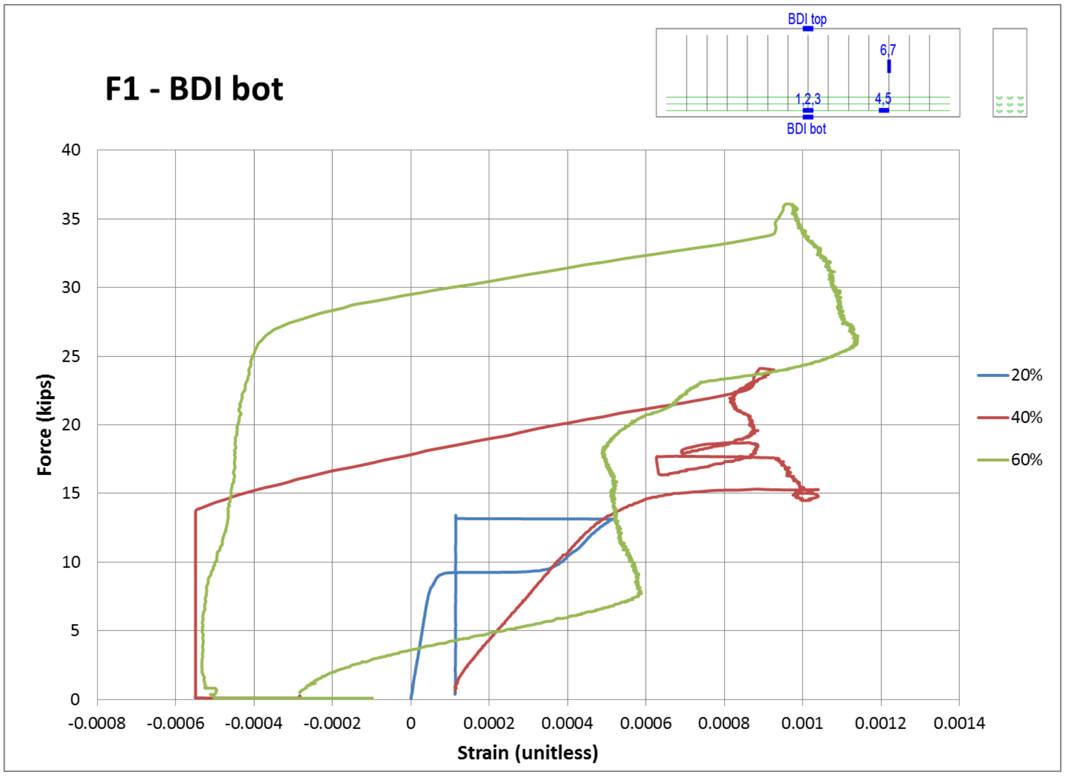

S6 - Strain gauge 2
BDI top
1
2
4
5
3
BDI bot
Force (kips)
80
70
60
50
40
30
20
10
0
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
Strain (unitless)
20%
40%
60%
Failure

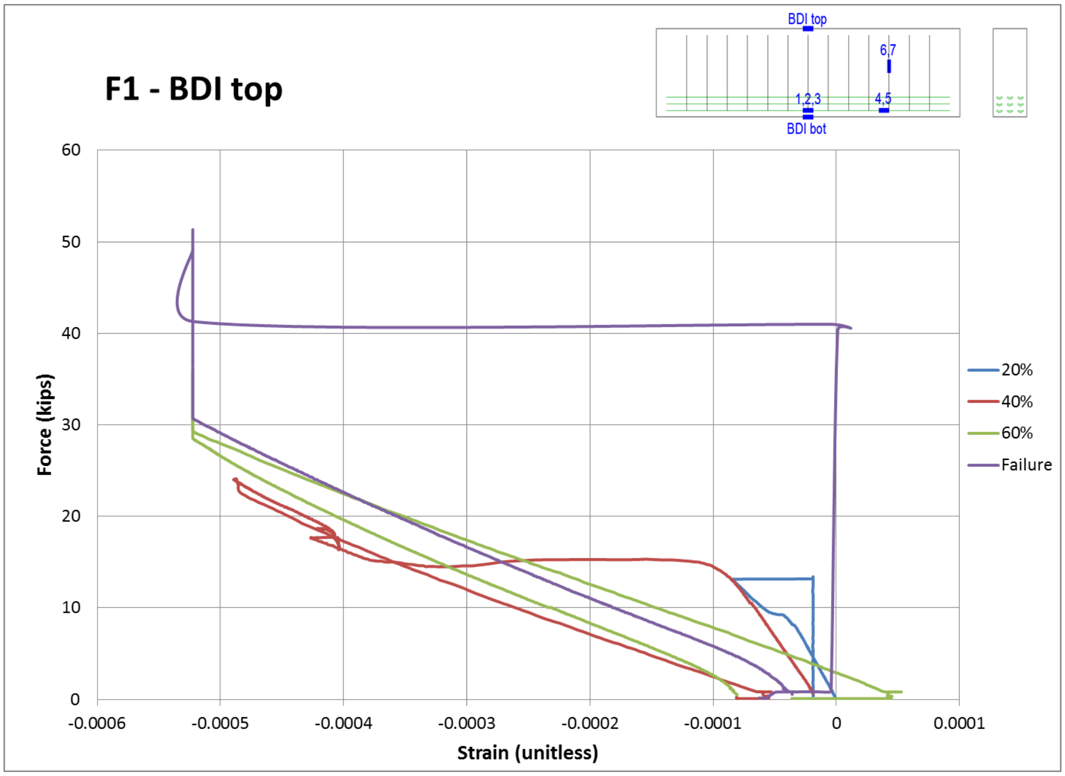

S6 - Strain gauge 3
BDI top
1
2
4
5
3
BDI bot
Force (kips)
80
70
60
50
40
30
20
10
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
Strain (unitless)
20%
40%
60%
Failure

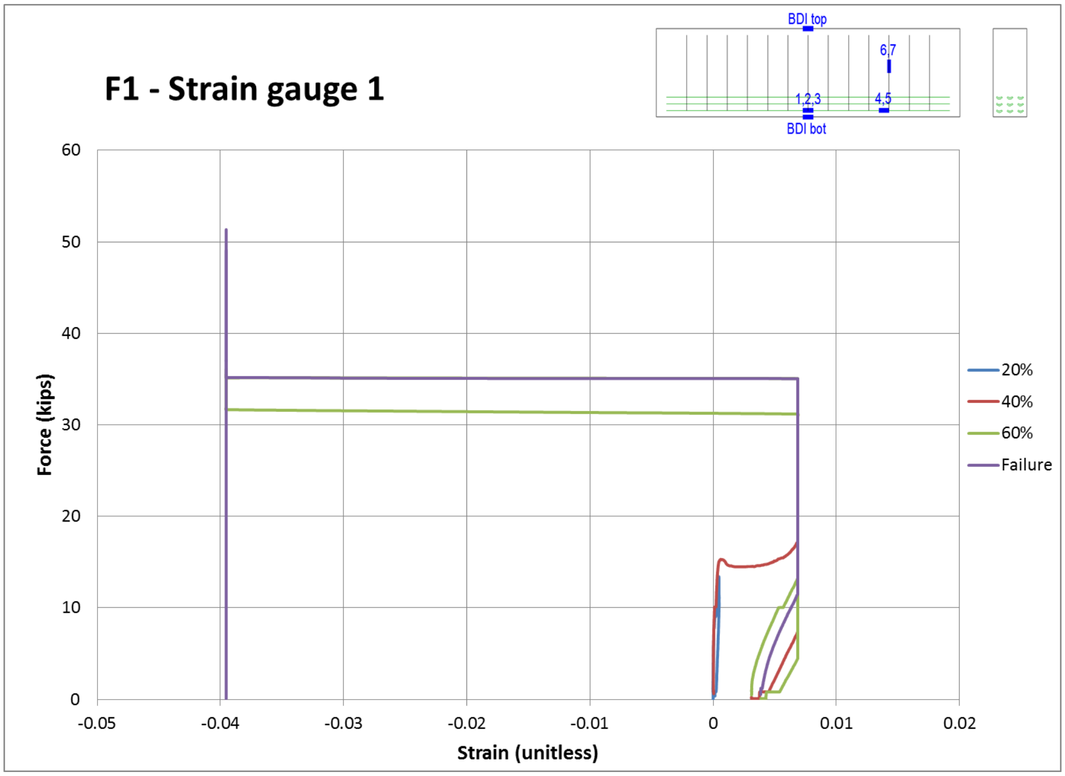

S6 - Strain gauge 4
BDI top
1
2
3
4
5
BDI bot
80
70
60
50
40
30
20
10
0
Force (kips)
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
0.007
Strain (unitless)
20%
40%
60%
Failure

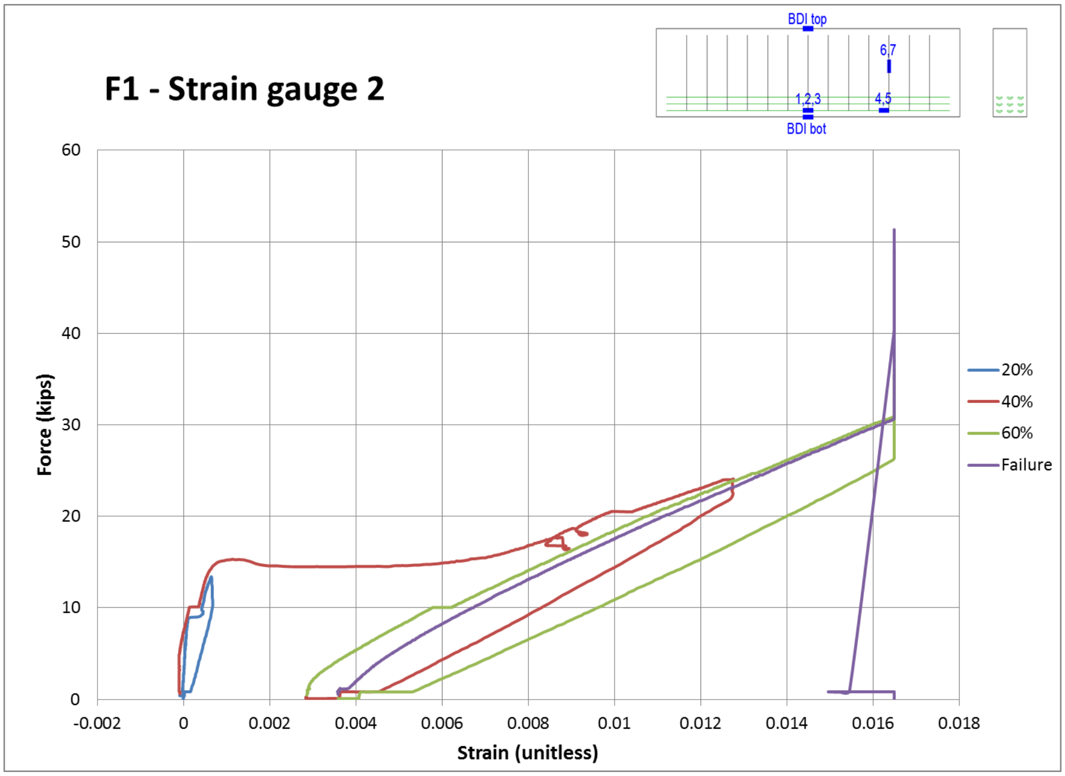

S6 - Strain gauge 5
BDI top
1
2
3
4
5
BDI bot
80
70
60
50
40
30
20
10
0
Force (kips)
-0.002
-0.0015
-0.001
-0.0005
0
0.0005
Strain (unitless)
20%
40%
60%
Failure

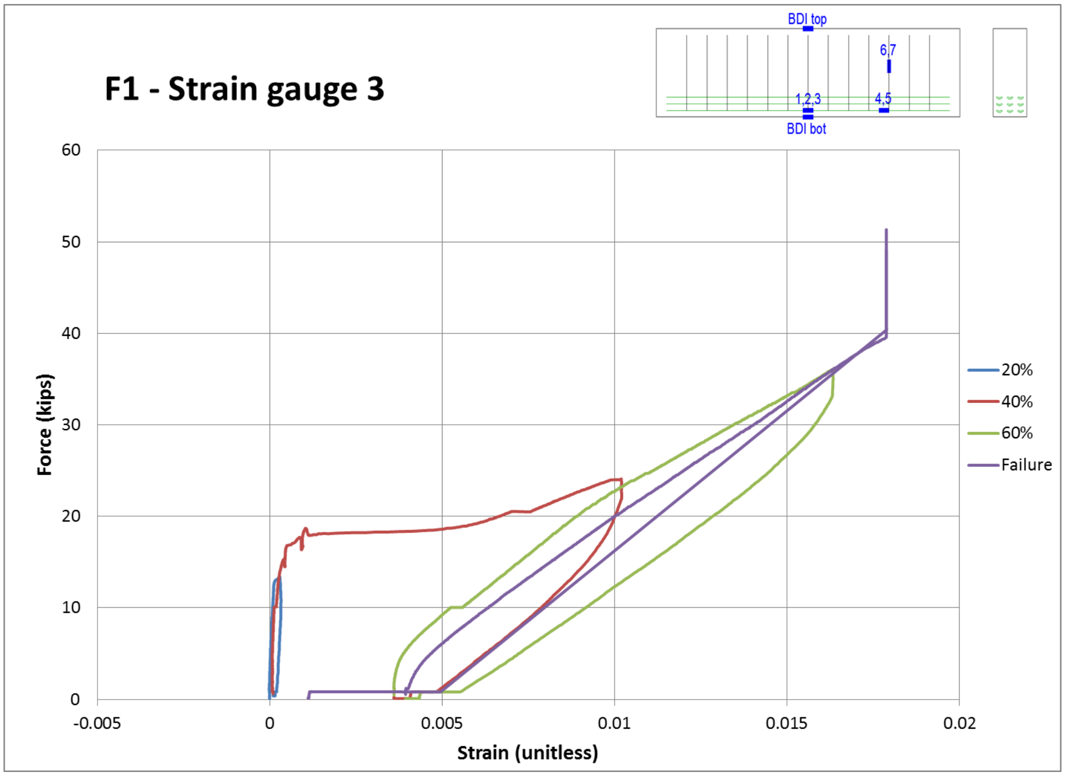
Appendix G.2 Flexure beams

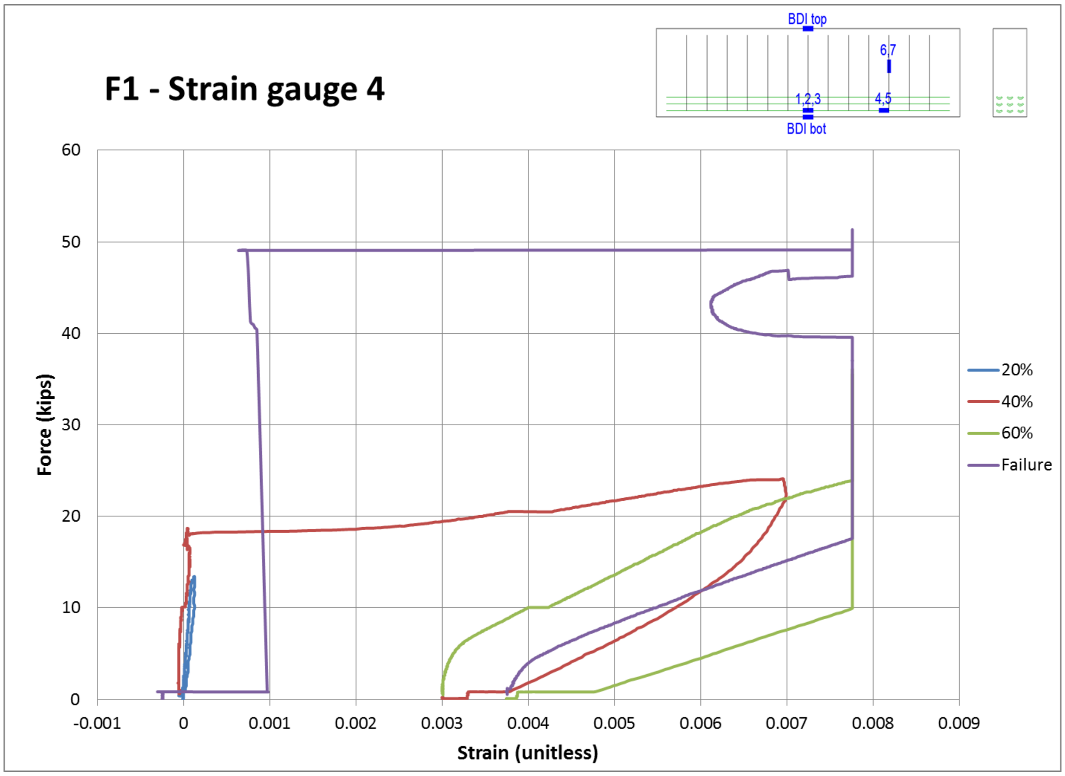


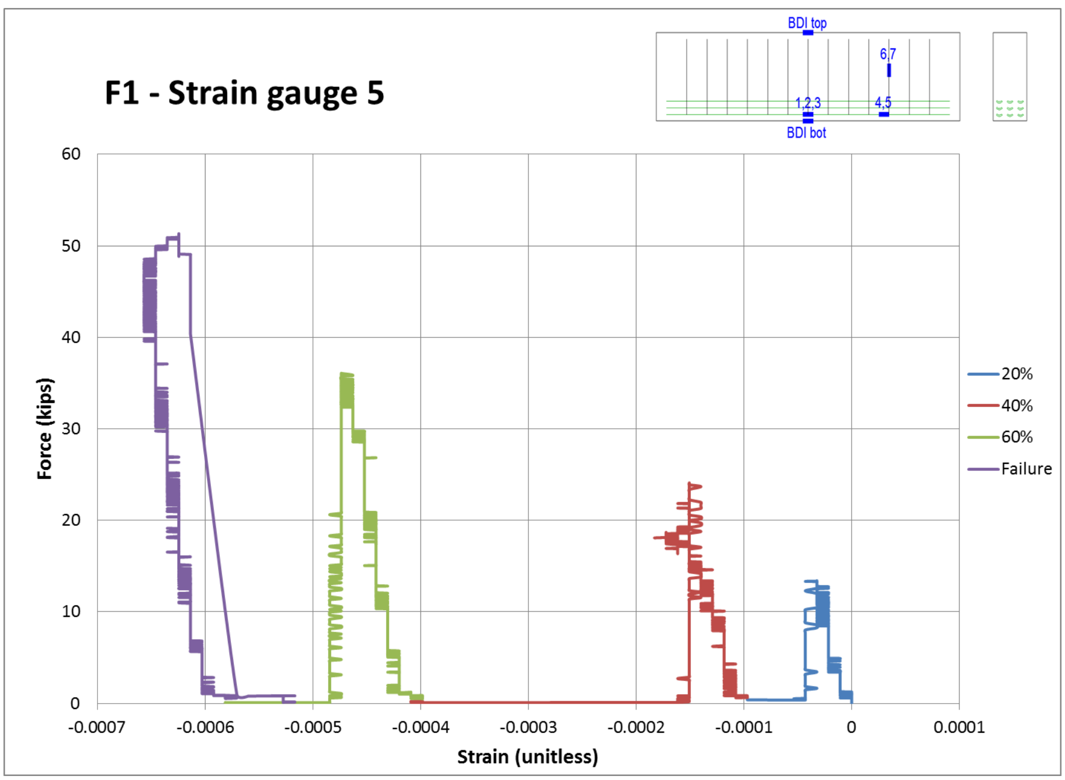

F1 - BDI top
BDI top
6,7
1,2,3
4,5
BDI bot
60
50
40
30
20
10
0
Force (kips)
-0.0006
-0.0005
-0.0004
-0.0003
-0.0002
-0.0001
0
0.0001
Strain (unitless)
20%
40%
60%
Failure

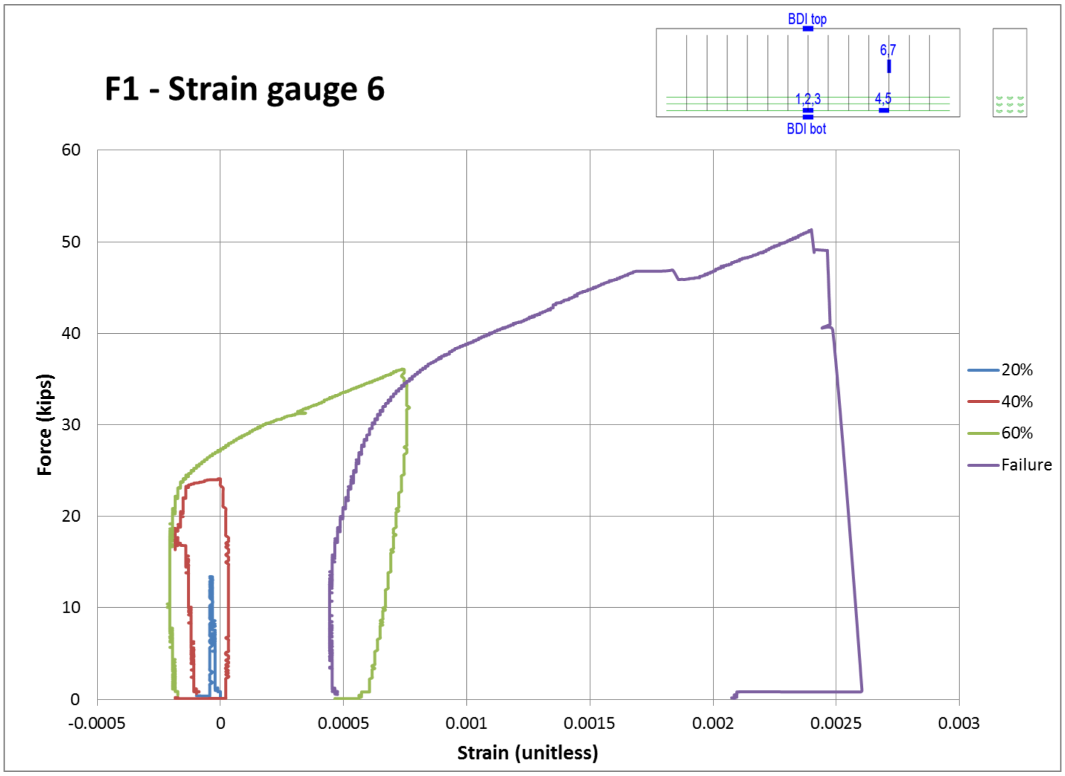

F1 - Strain gauge 1
BDI top
6,7
1,2,3
4,5
BDI bot
60
50
40
30
20
10
0
Force (kips)
-0.05
-0.04
-0.03
-0.02
-0.01
0
0.01
0.02
Strain (unitless)
20%
40%
60%
Failure

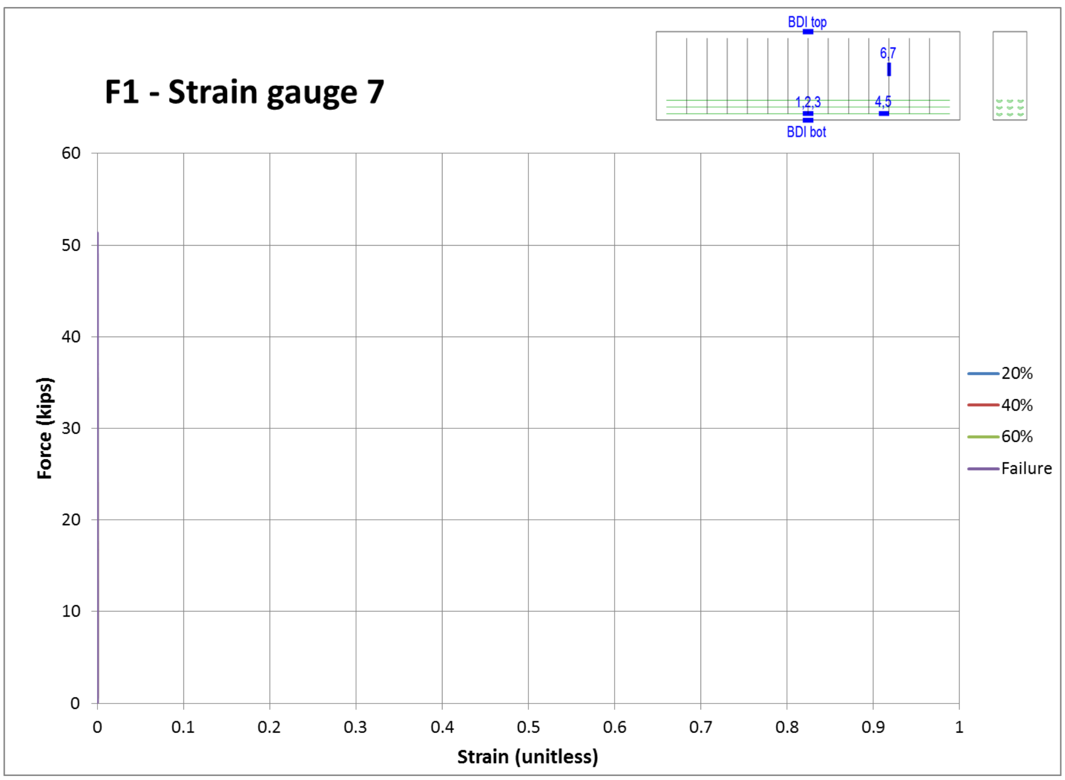

F1 - Strain gauge 2
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
60
50
40
30
20
10
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
Strain (unitless)
20%
40%
60%
Failure

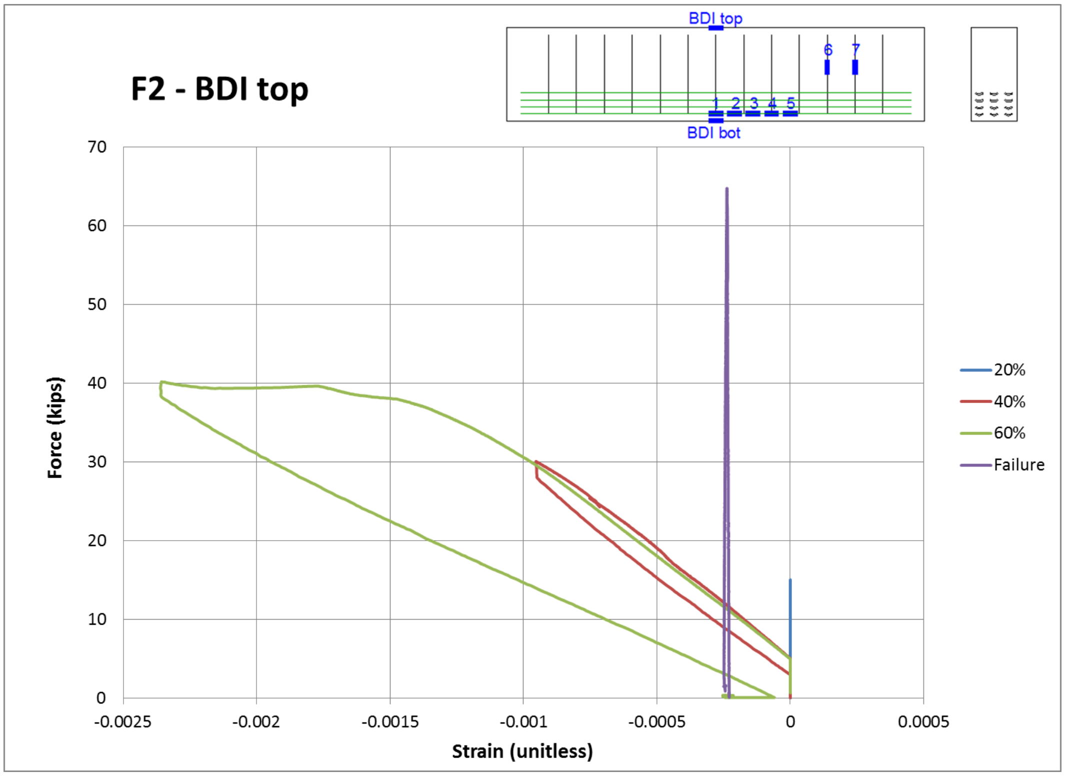

F1 - Strain gauge 3
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
60
50
40
30
20
10
0
-0.005
0
0.005
0.01
0.015
0.02
Strain (unitless)
20%
40%
60%
Failure

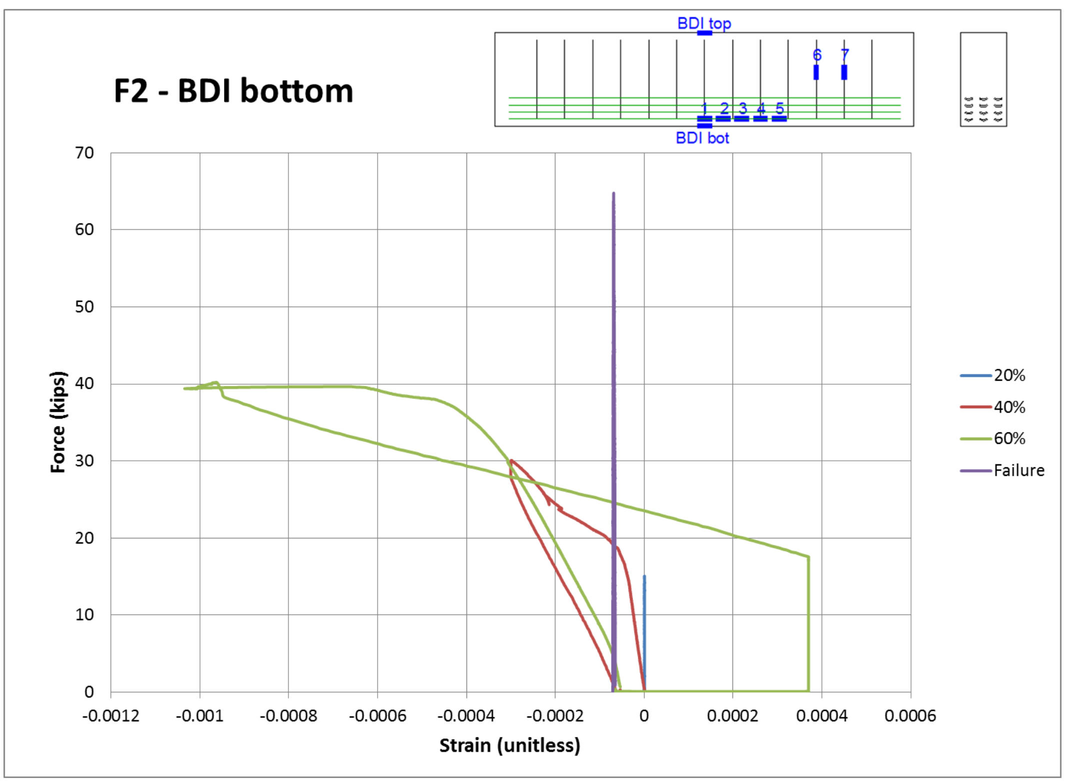

F1 - Strain gauge 4
BDI top
6,7
1,2,3
4,5
BDI bot
60
50
40
30
20
10
0
Force (kips)
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
0.007
0.008
0.009
Strain (unitless)
20%
40%
60%
Failure

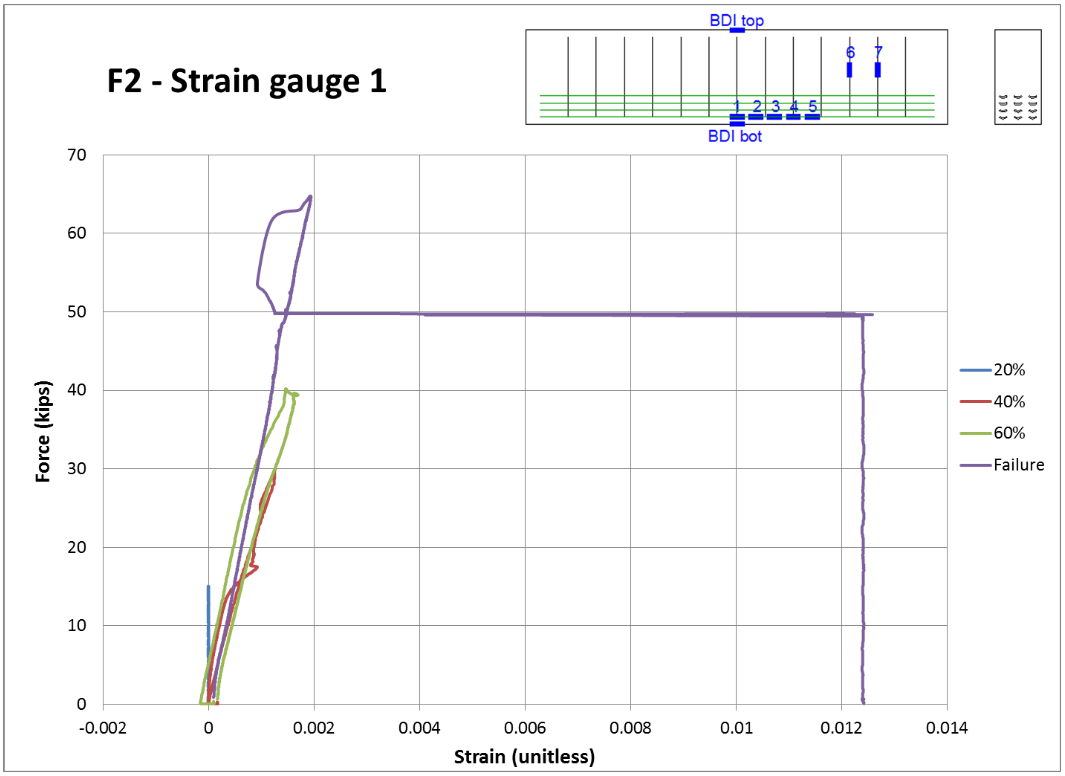

F1 - Strain gauge 5
BDI top
6,7
1,2,3
4,5
BDI bot
60
50
40
30
20
10
0
Force (kips)
-0.0007
-0.0006
-0.0005
-0.0004
-0.0003
-0.0002
-0.0001
0
0.0001
Strain (unitless)
20%
40%
60%
Failure

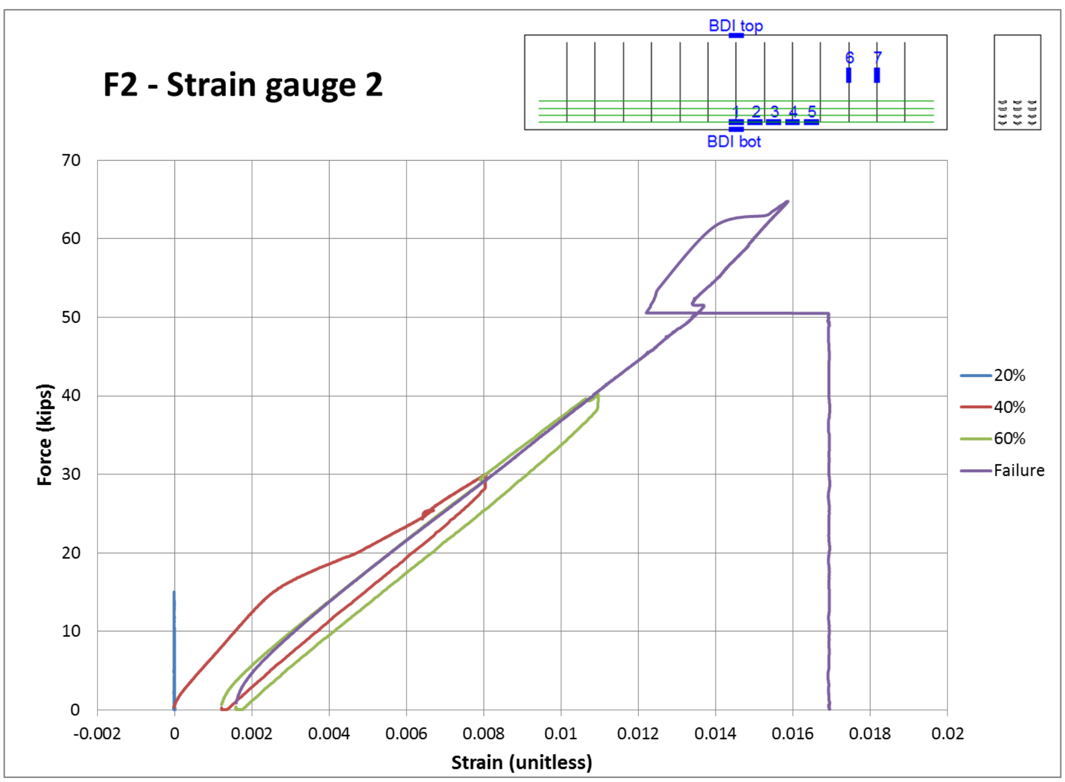

F1 - Strain gauge 6
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
60
50
40
30
20
10
0
-0.0005
0
0.0005
0.001
0.0015
0.002
0.0025
0.003
20%
40%
60%
Failure

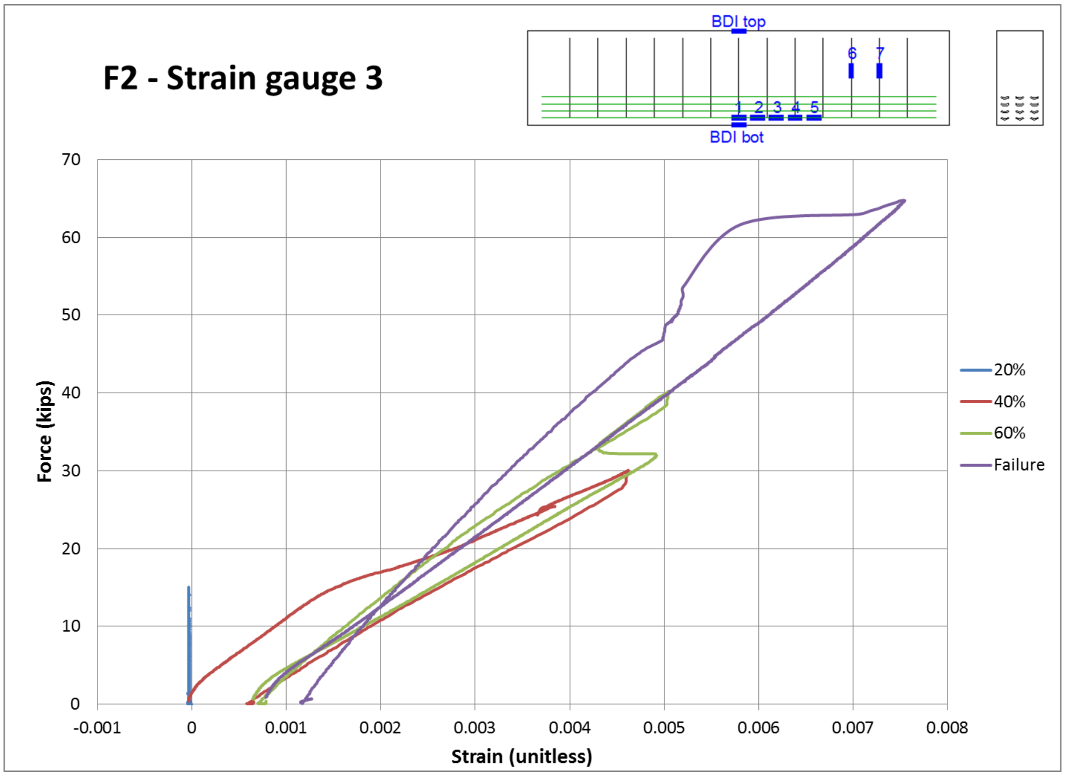

F1 - Strain gauge 7
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
60
50
40
30
20
10
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
20%
40%
60%
Failure

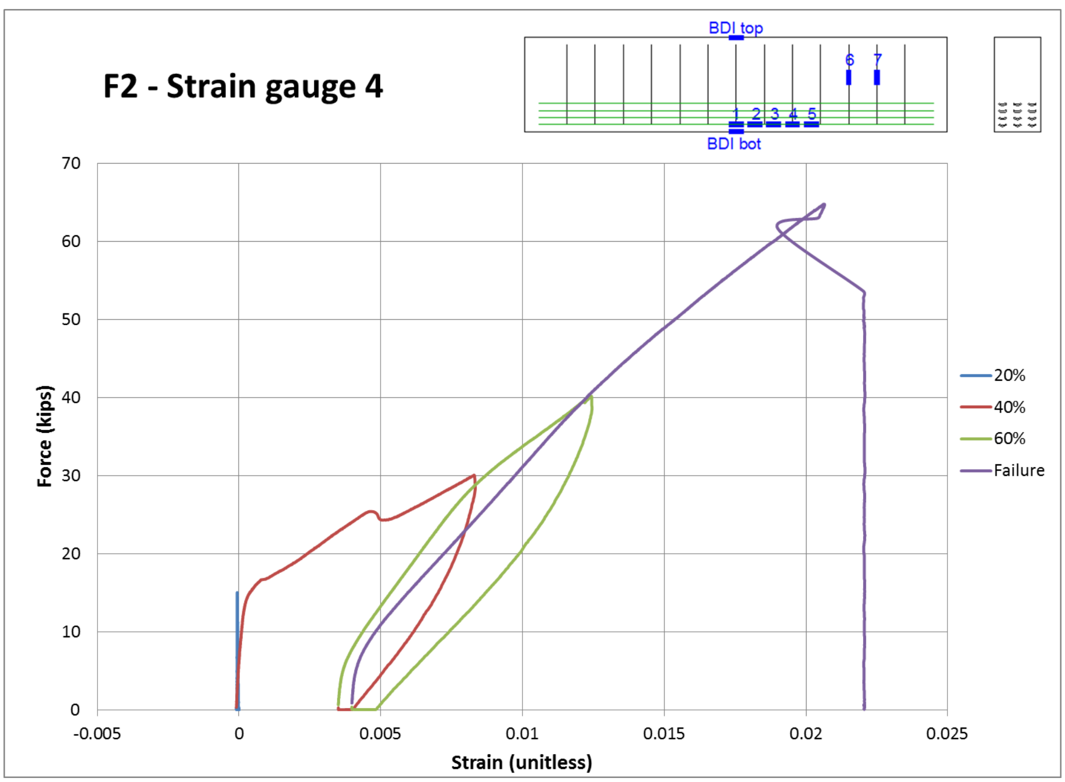

F2 - BDI top
BDI top
BDI bot
1 2 3 4 5
6 7
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

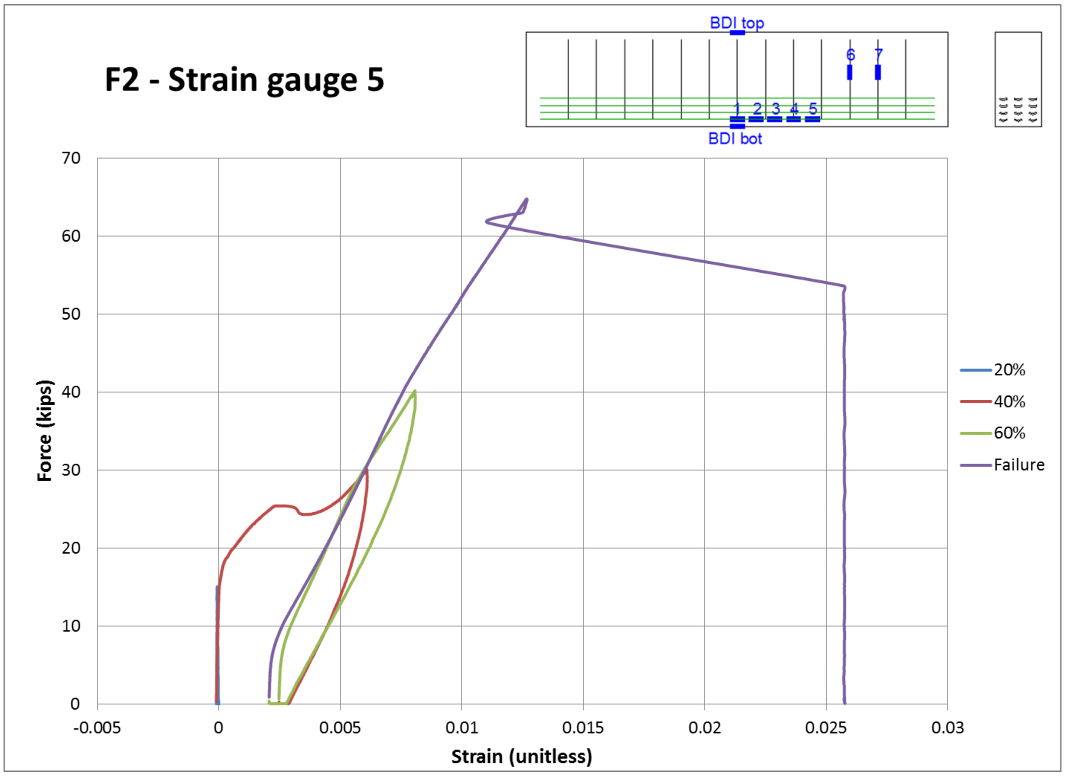

F2 - BDI bottom
BDI top
BDI bot
1 2 3 4 5
6 7
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

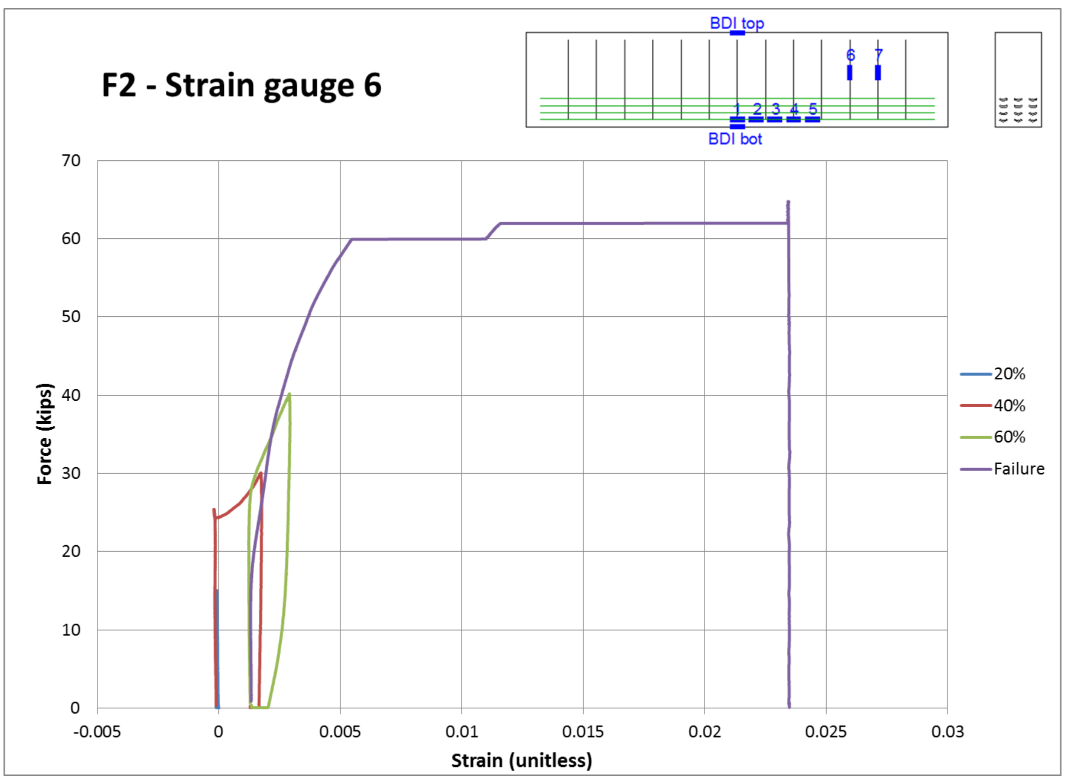

F2 - Strain gauge 1
BDI top
BDI bot
1 2 3 4 5
6 7
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
20%
40%
60%
Failure

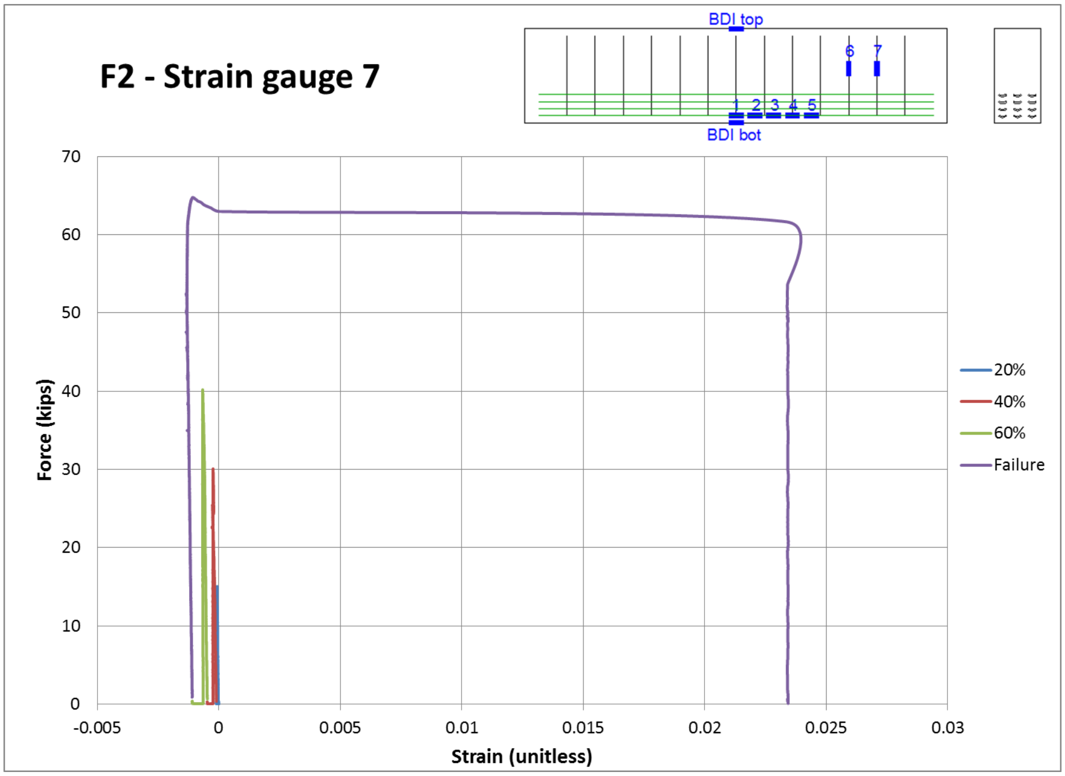

F2 - Strain gauge 2
BDI top
BDI bot
1 2 3 4 5
6 7
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02
20%
40%
60%
Failure

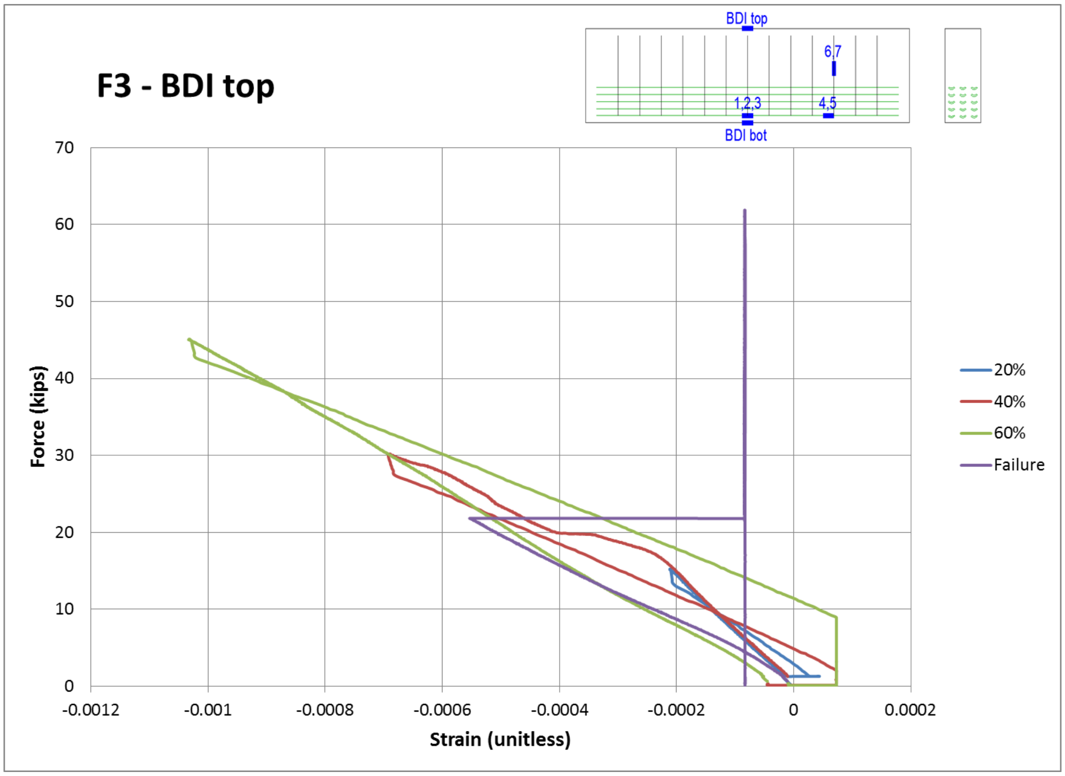

F2 - Strain gauge 3
BDI top
6
7
1
2
3
4
5
BDI bot
70
60
50
40
30
20
10
0
Force (kips)
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
0.007
0.008
Strain (unitless)
20%
40%
60%
Failure

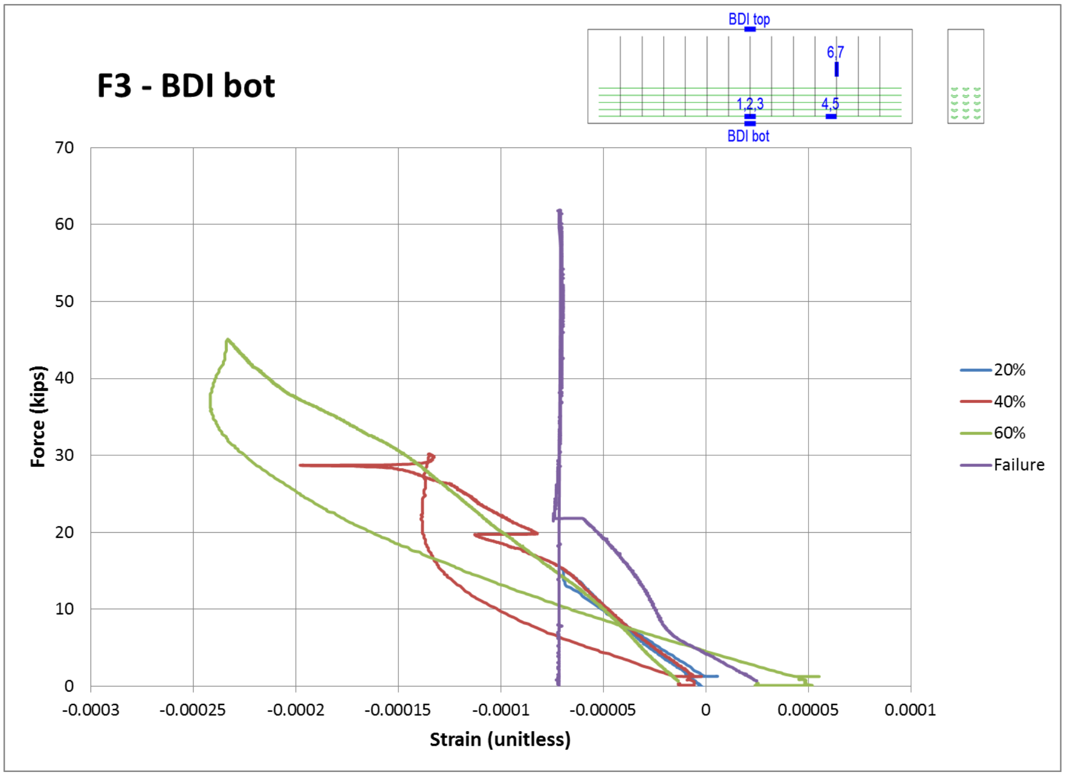

F2 - Strain gauge 4
BDI top
6
7
1
2
3
4
5
BDI bot
70
60
50
40
30
20
10
0
Force (kips)
-0.005
0
0.005
0.01
0.015
0.02
0.025
Strain (unitless)
20%
40%
60%
Failure

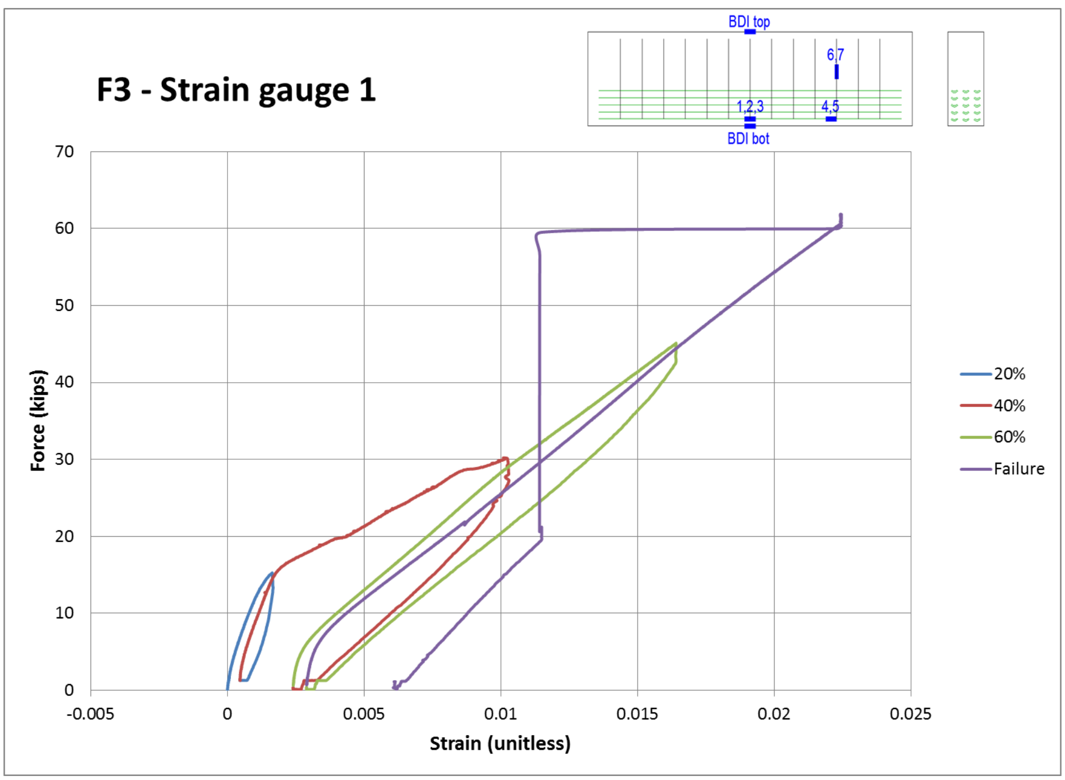

F2 - Strain gauge 5
BDI top
BDI bot
1 2 3 4 5
6 7
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
20%
40%
60%
Failure

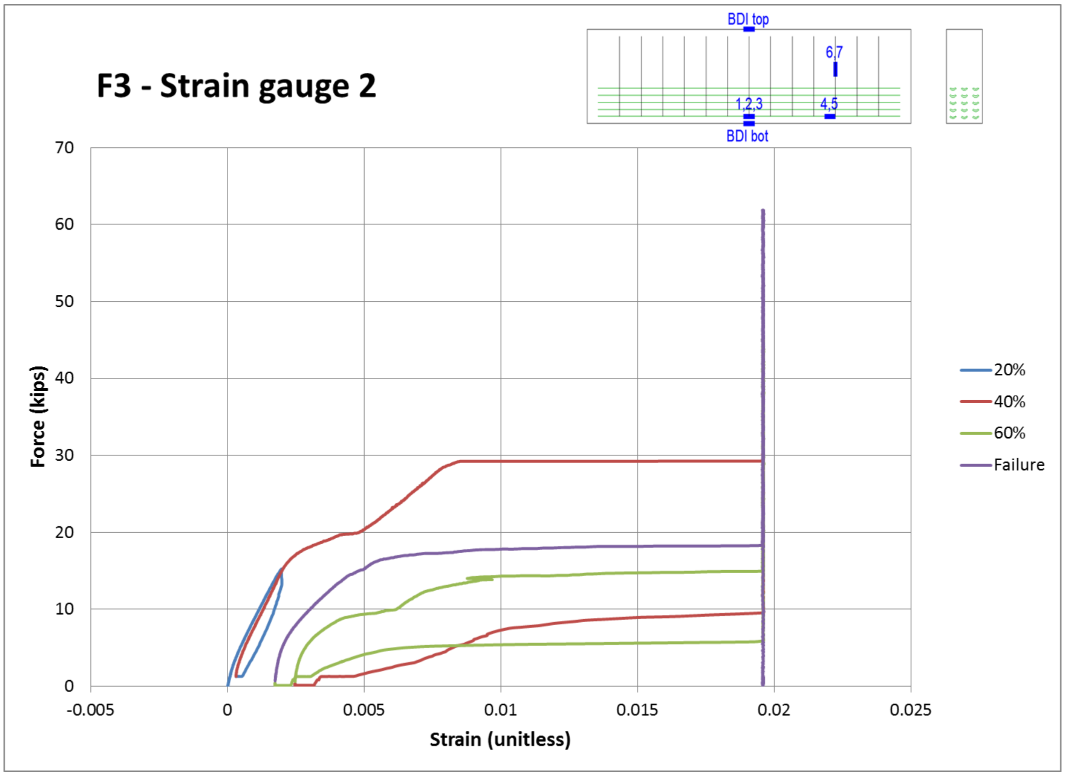

F2 - Strain gauge 6
BDI top
BDI bot
1 2 3 4 5
6 7
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
20%
40%
60%
Failure

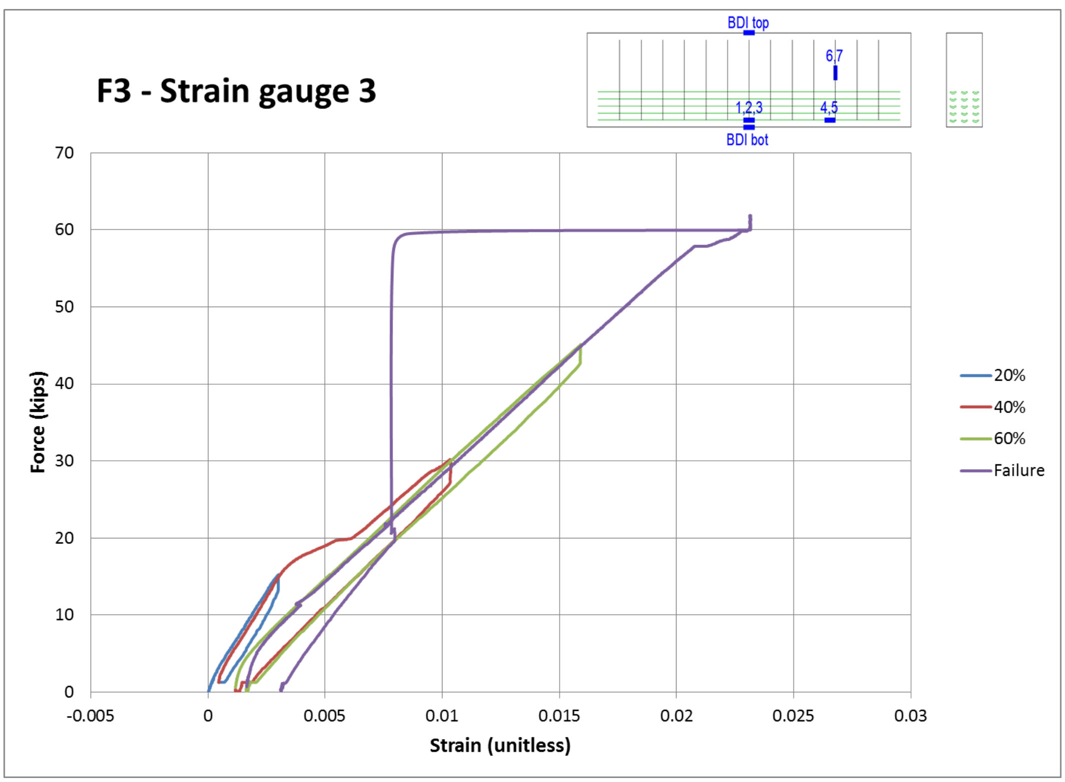

F2 - Strain gauge 7
BDI top
BDI bot
1
2
3
4
5
6
7
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

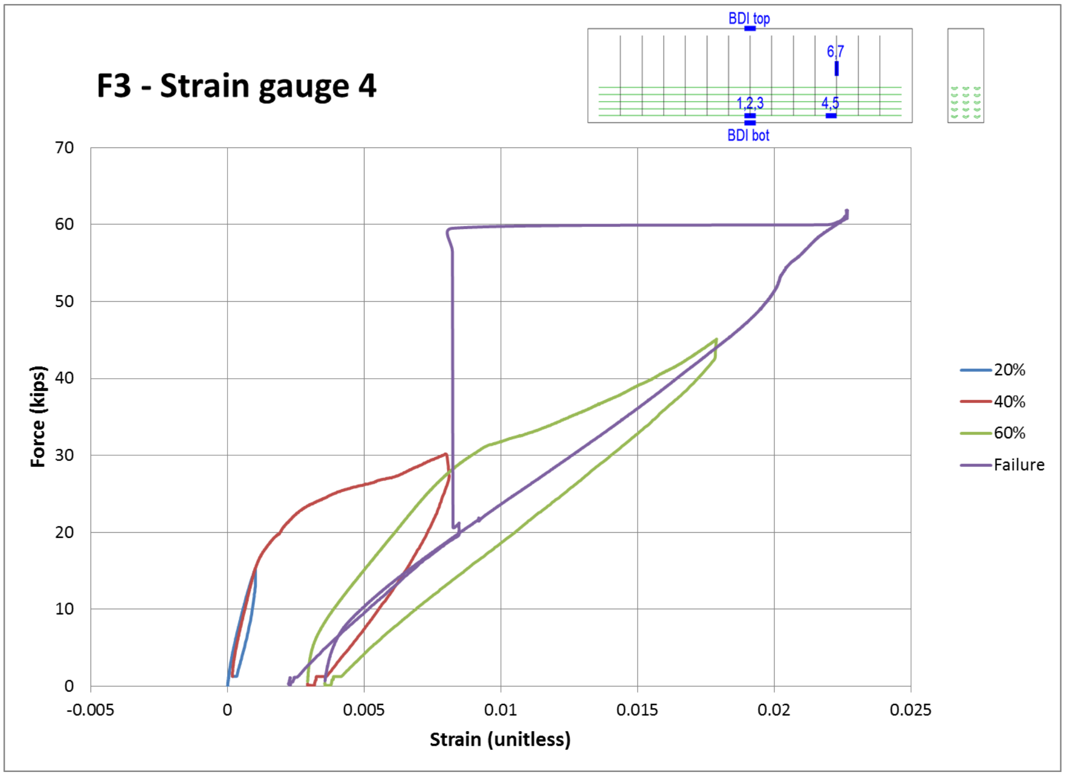

F3 - BDI top
BDI top
BDI bot
1,2,3
4,5
6,7
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

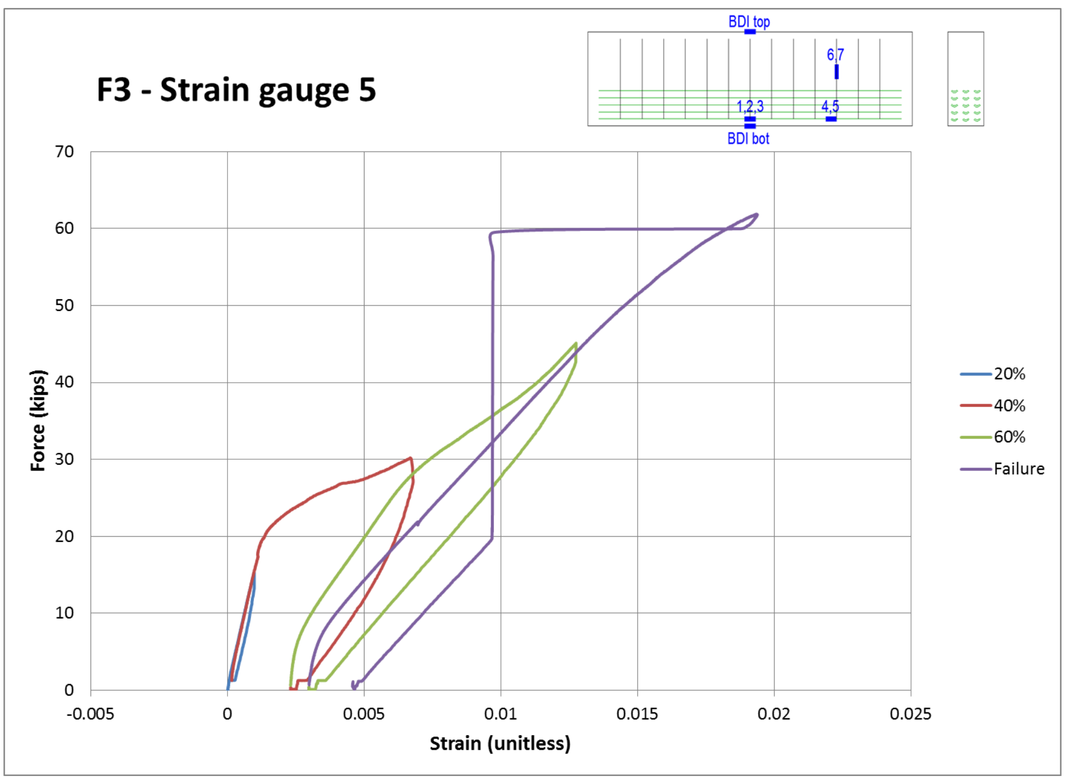

F3 - BDI bot
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

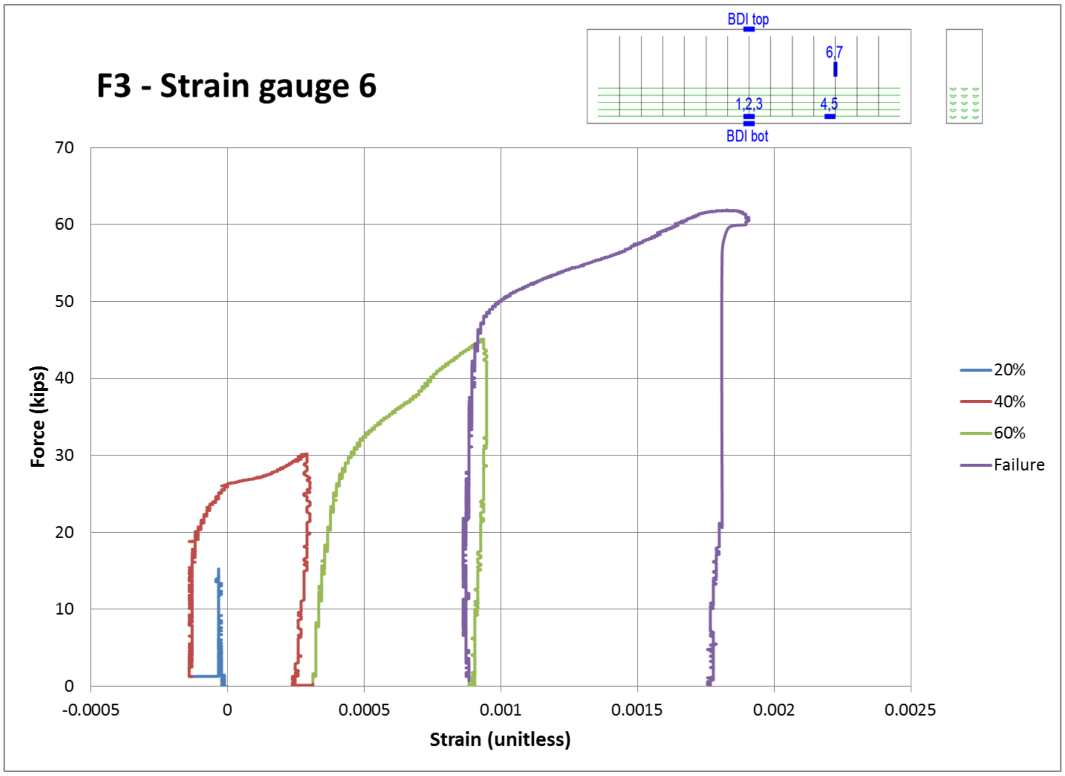

F3 - Strain gauge 1
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

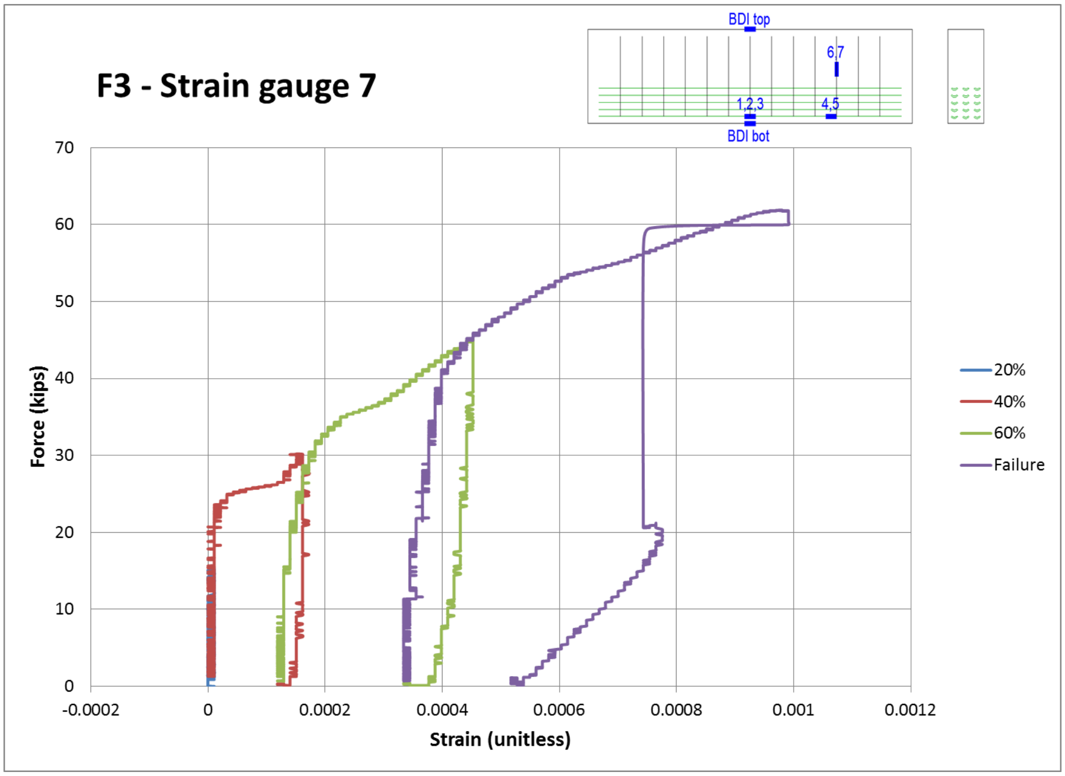

F3 - Strain gauge 2
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
20%
40%
60%
Failure

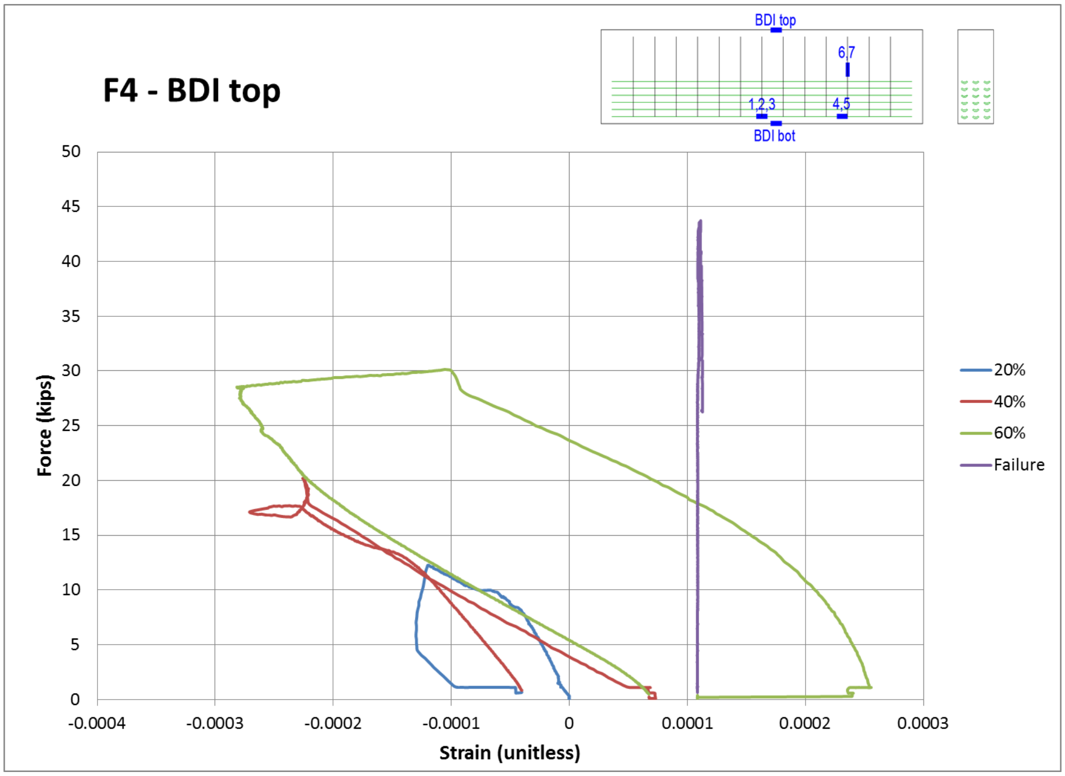

F3 - Strain gauge 3
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.005
0
0.005
0.01
0.015
0.02
0.025
0.03
20%
40%
60%
Failure

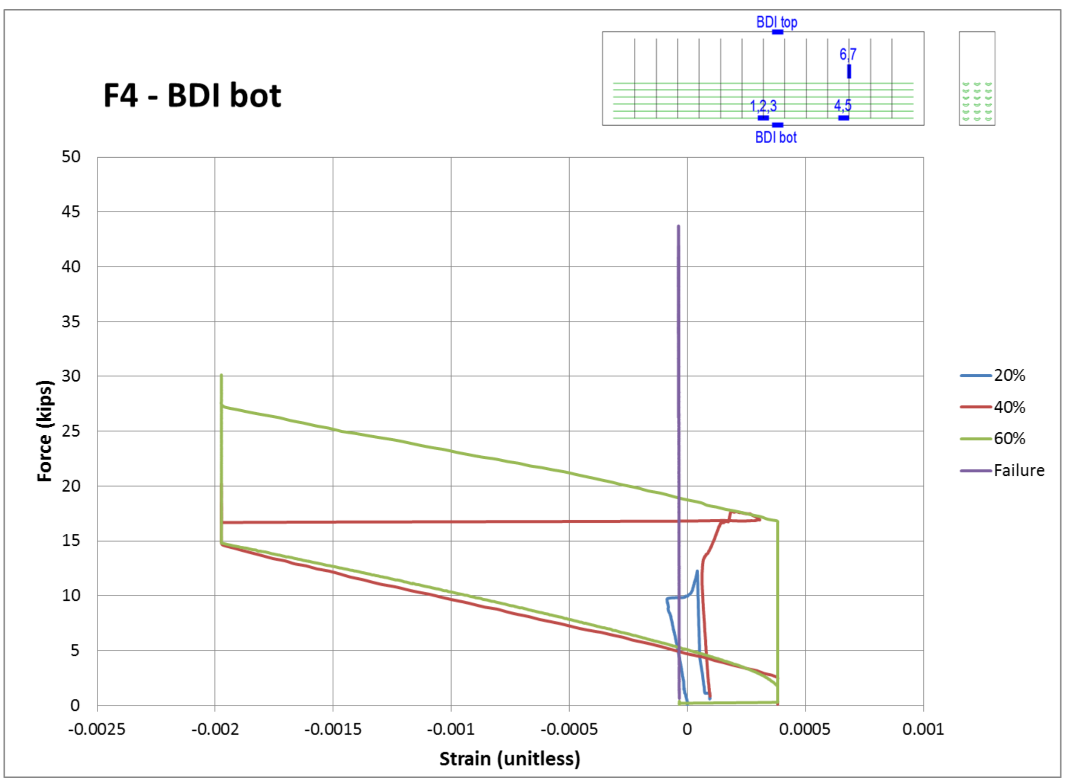

F3 - Strain gauge 4
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

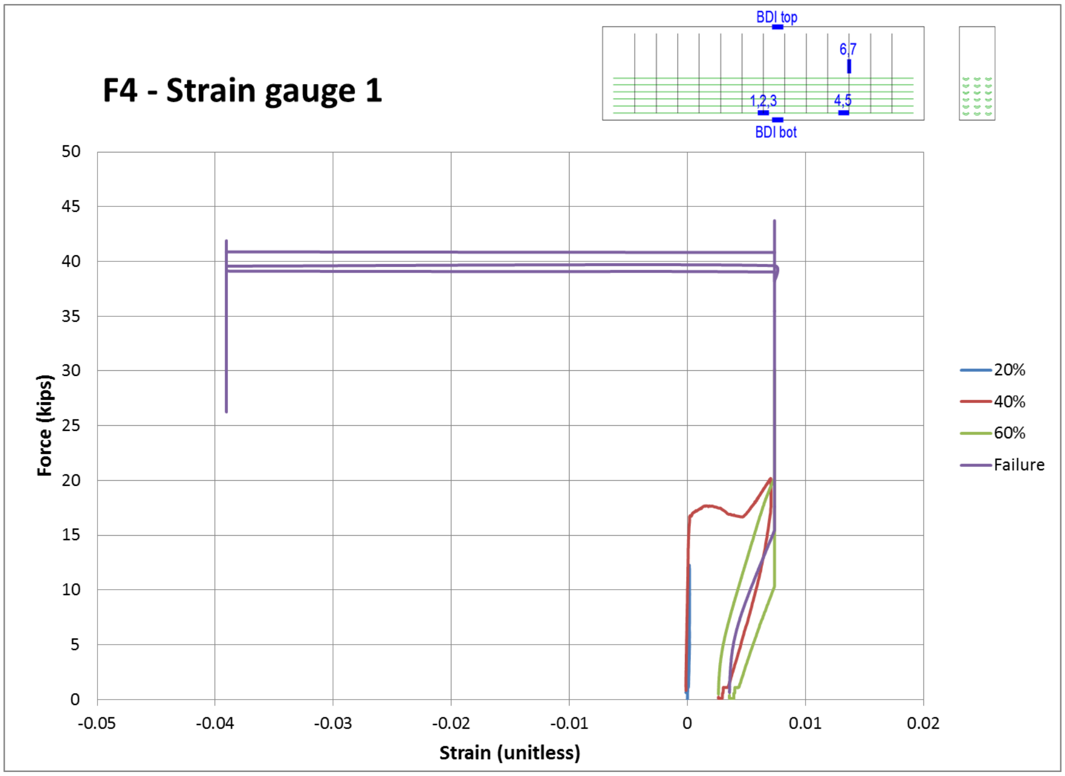

F3 - Strain gauge 5
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

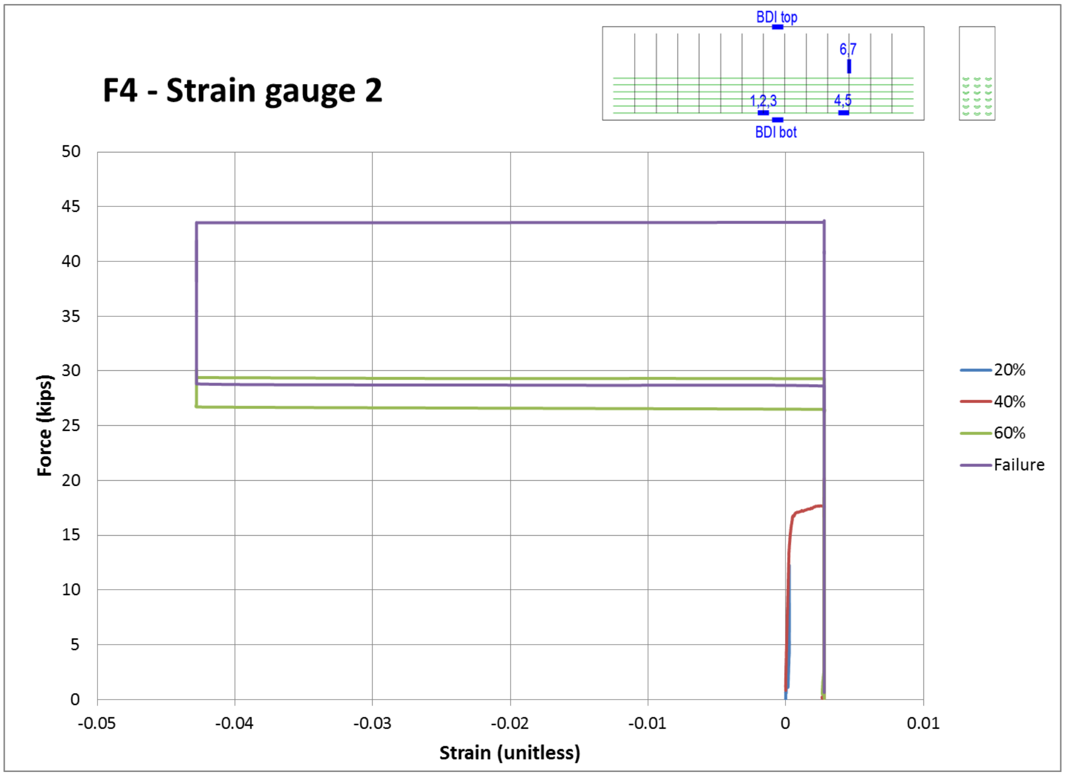

F3 - Strain gauge 6
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
70
60
50
40
30
20
10
0
-0.0005
0
0.0005
0.001
0.0015
0.002
0.0025
Strain (unitless)
20%
40%
60%
Failure

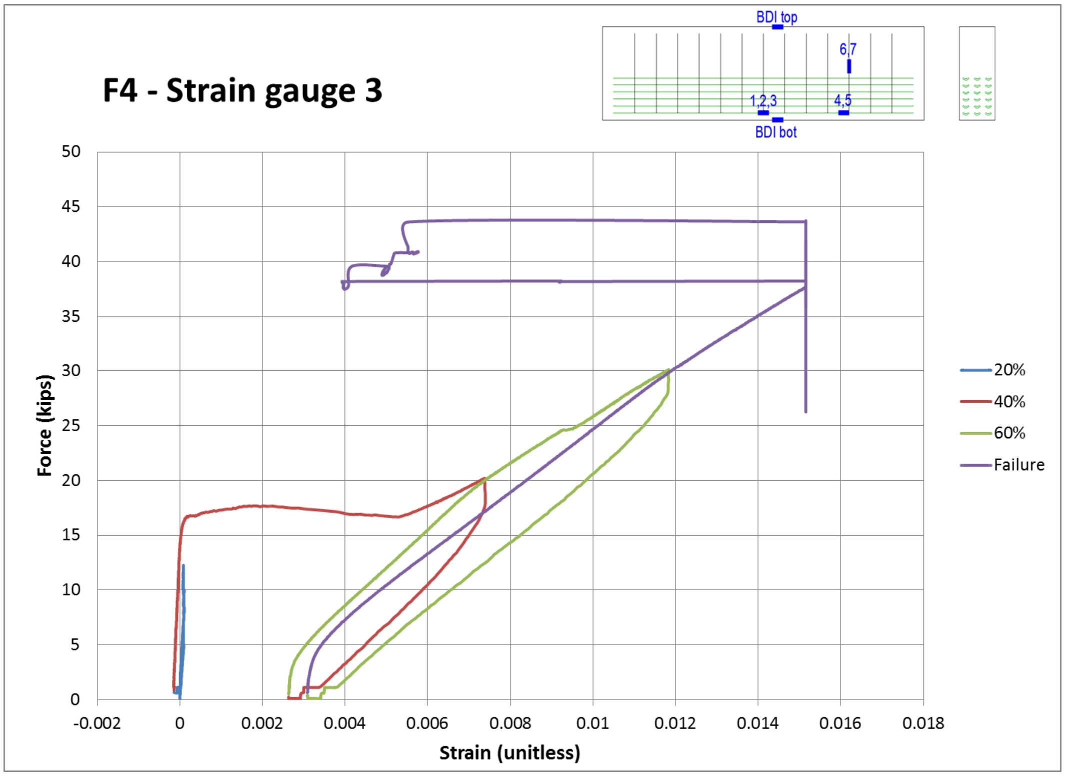

F3 - Strain gauge 7
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
70
60
50
40
30
20
10
0
-0.0002
0
0.0002
0.0004
0.0006
0.0008
0.001
0.0012
Strain (unitless)
20%
40%
60%
Failure


F4 - BDI top
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
50
45
40
35
30
25
20
15
10
5
0
-0.0004
-0.0003
-0.0002
-0.0001
0
0.0001
0.0002
0.0003
Strain (unitless)
20%
40%
60%
Failure

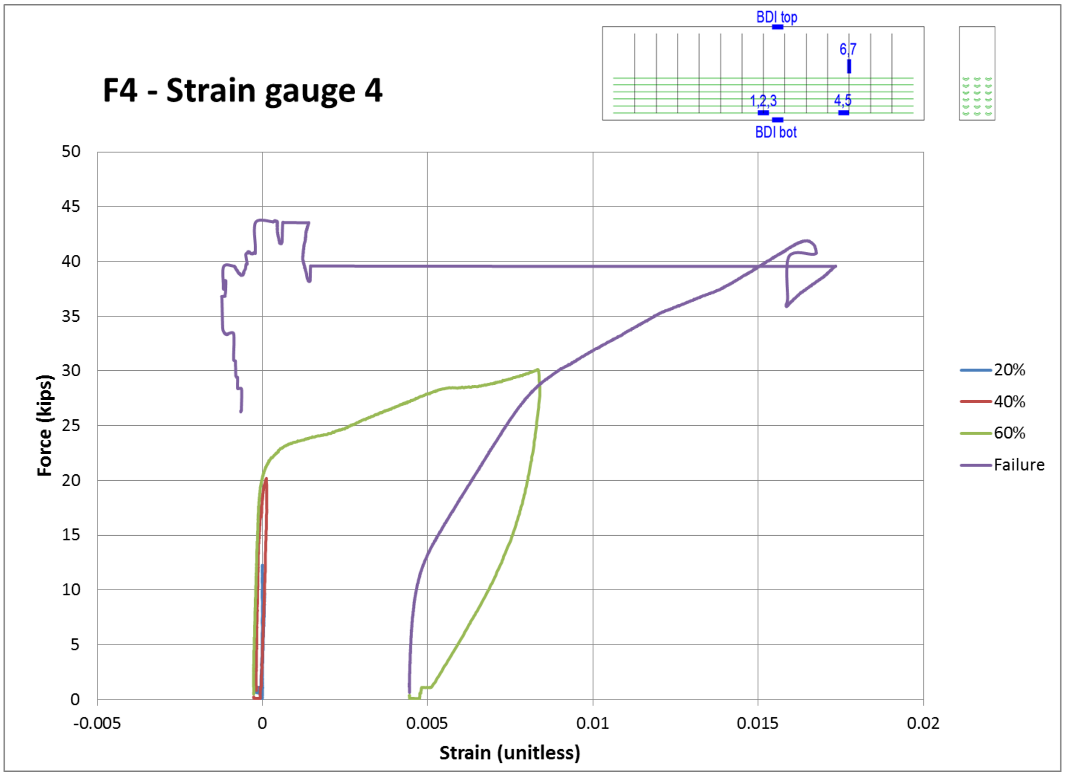

F4 - BDI bot
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
50
45
40
35
30
25
20
15
10
5
0
-0.0025
-0.002
-0.0015
-0.001
-0.0005
0
0.0005
0.001
Strain (unitless)
20%
40%
60%
Failure


F4 - Strain gauge 1
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

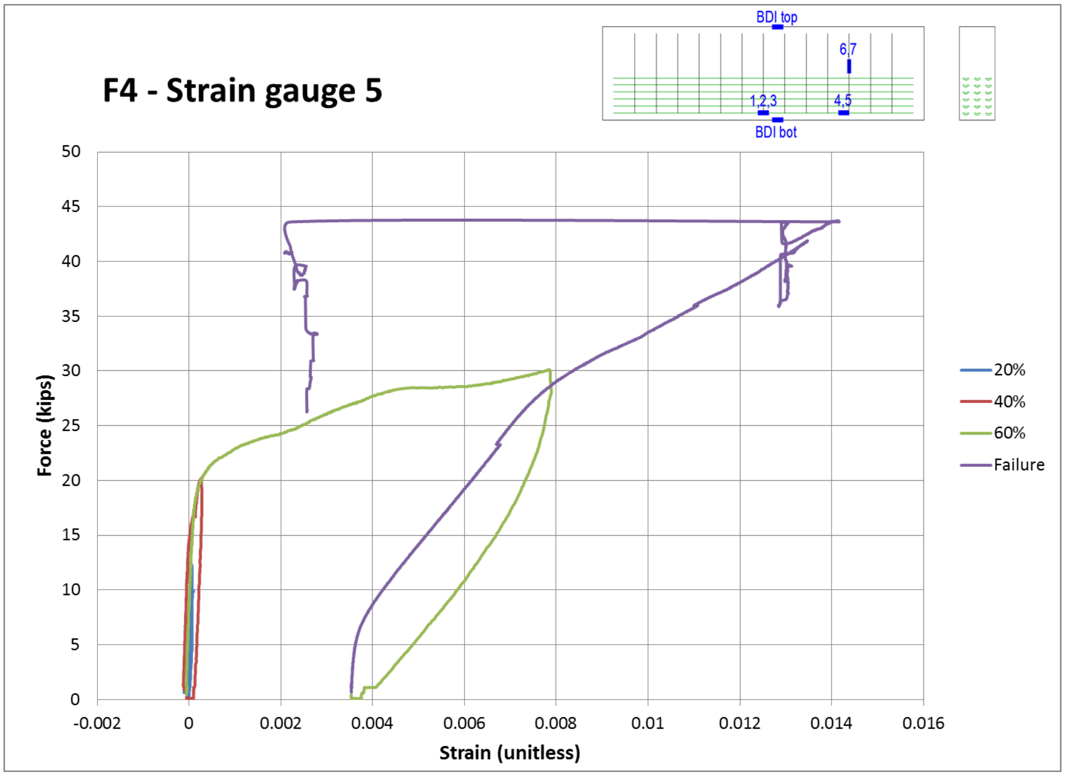

F4 - Strain gauge 2
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

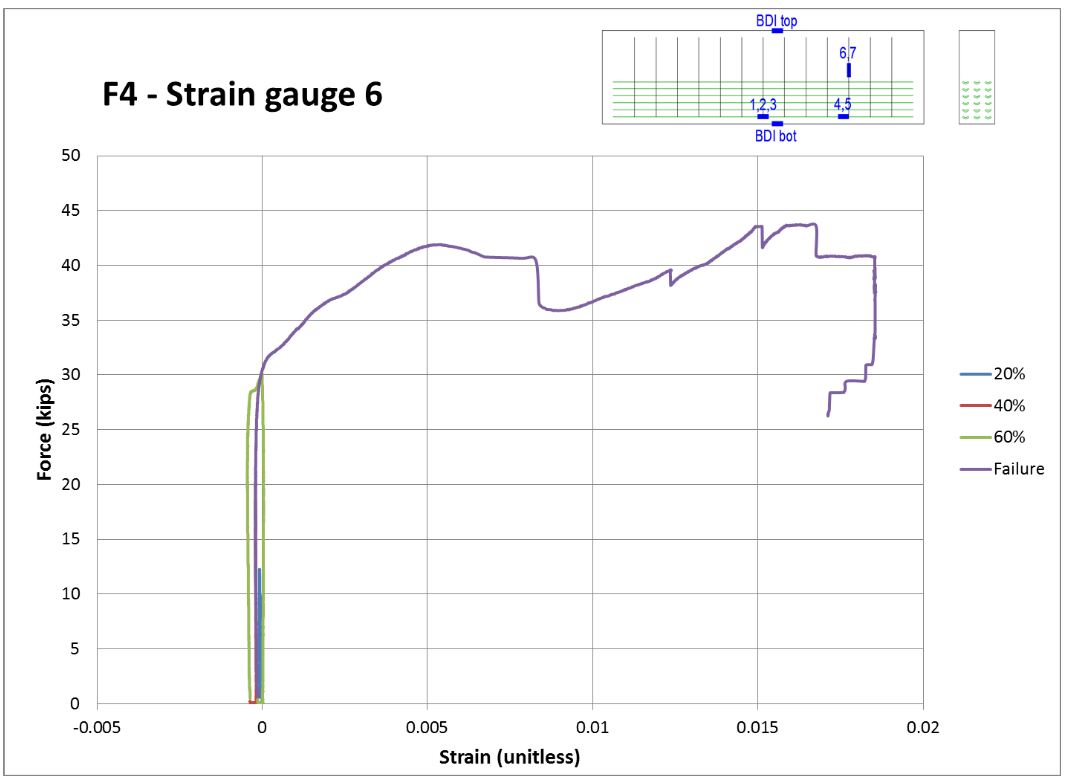


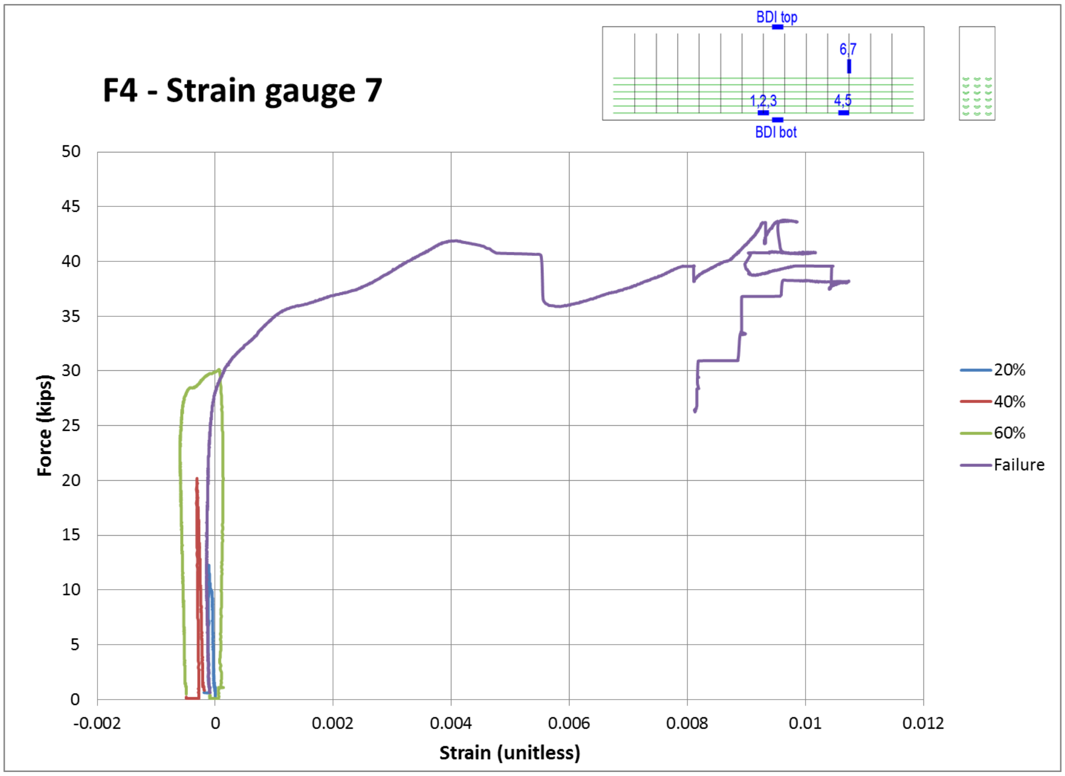
F4 - Strain gauge 3
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
50
45
40
35
30
25
20
15
10
5
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
Strain (unitless)
20%
40%
60%
Failure



F4 - Strain gauge 4
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
50
45
40
35
30
25
20
15
10
5
0
-0.005
0
0.005
0.01
0.015
0.02
Strain (unitless)
20%
40%
60%
Failure

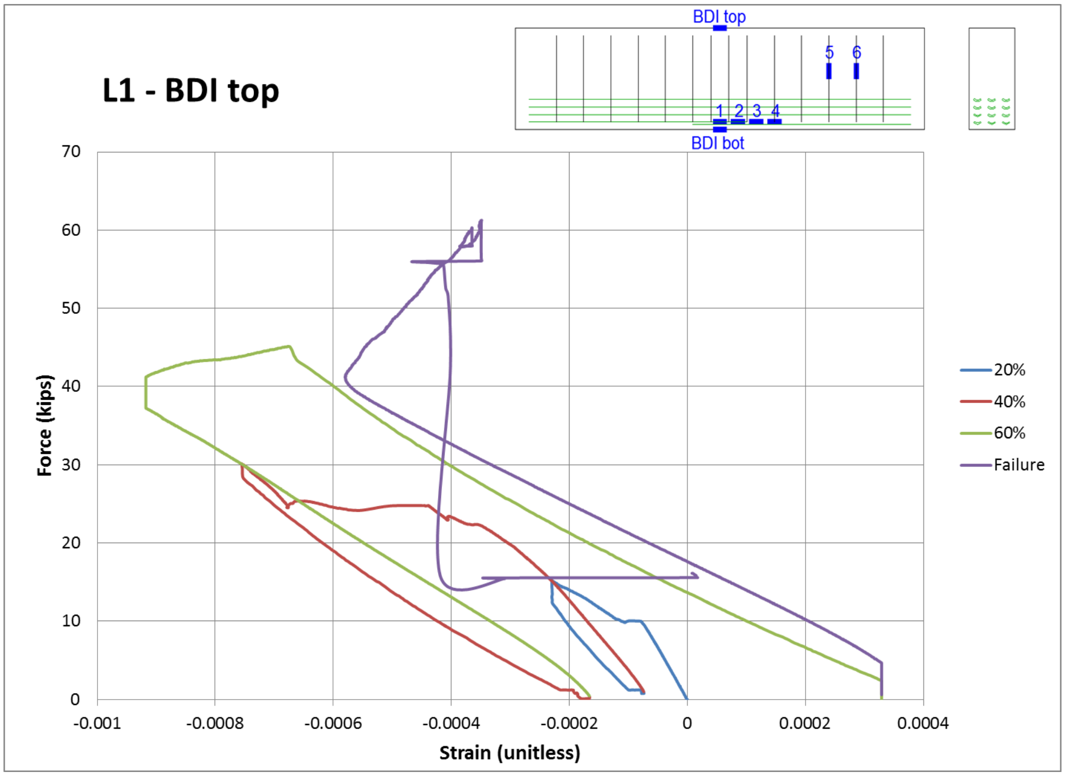

F4 - Strain gauge 5
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

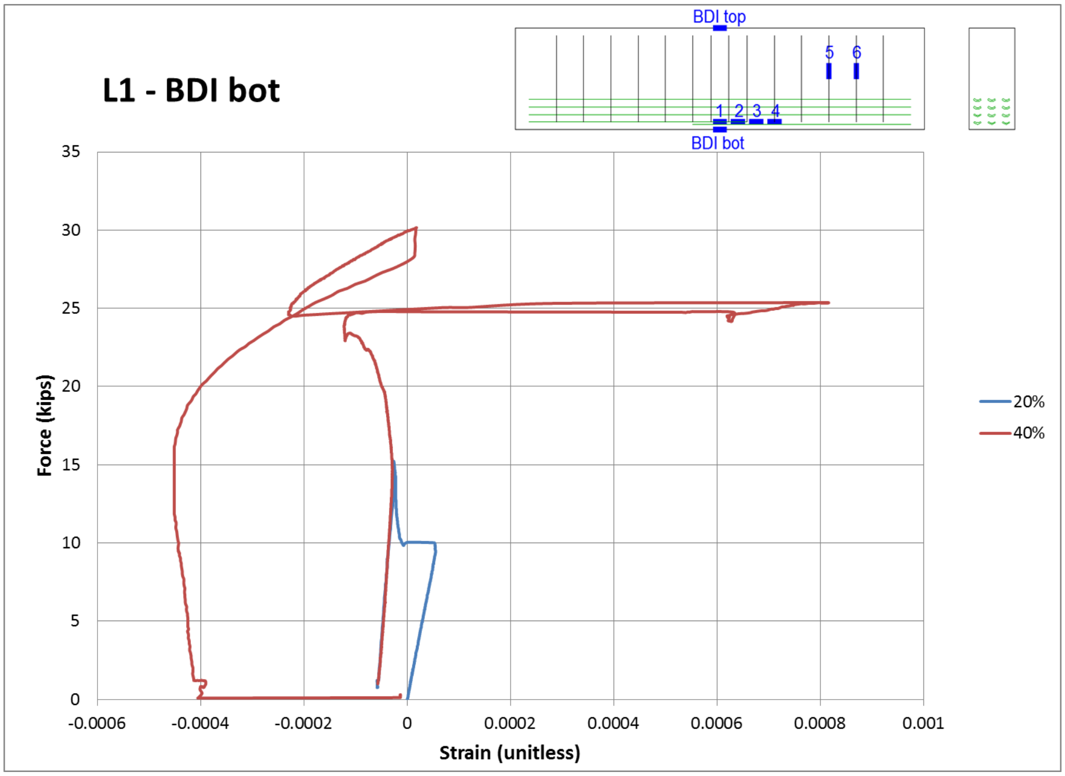

F4 - Strain gauge 6
BDI top
6,7
1,2,3
4,5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

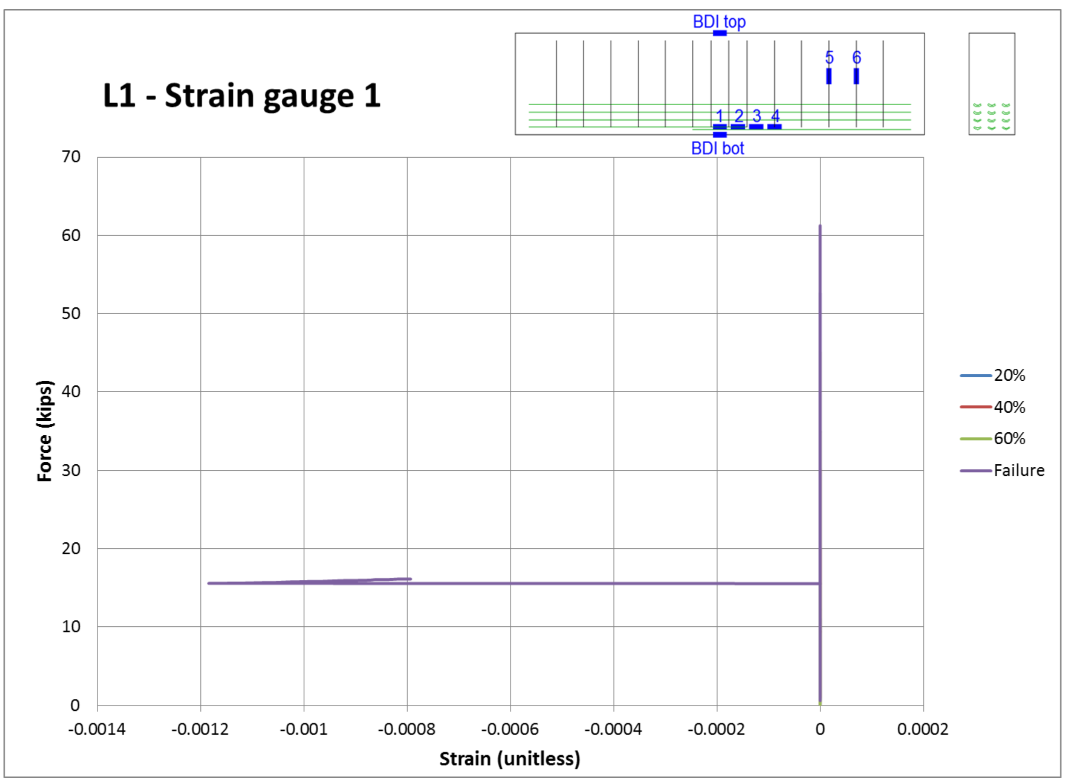


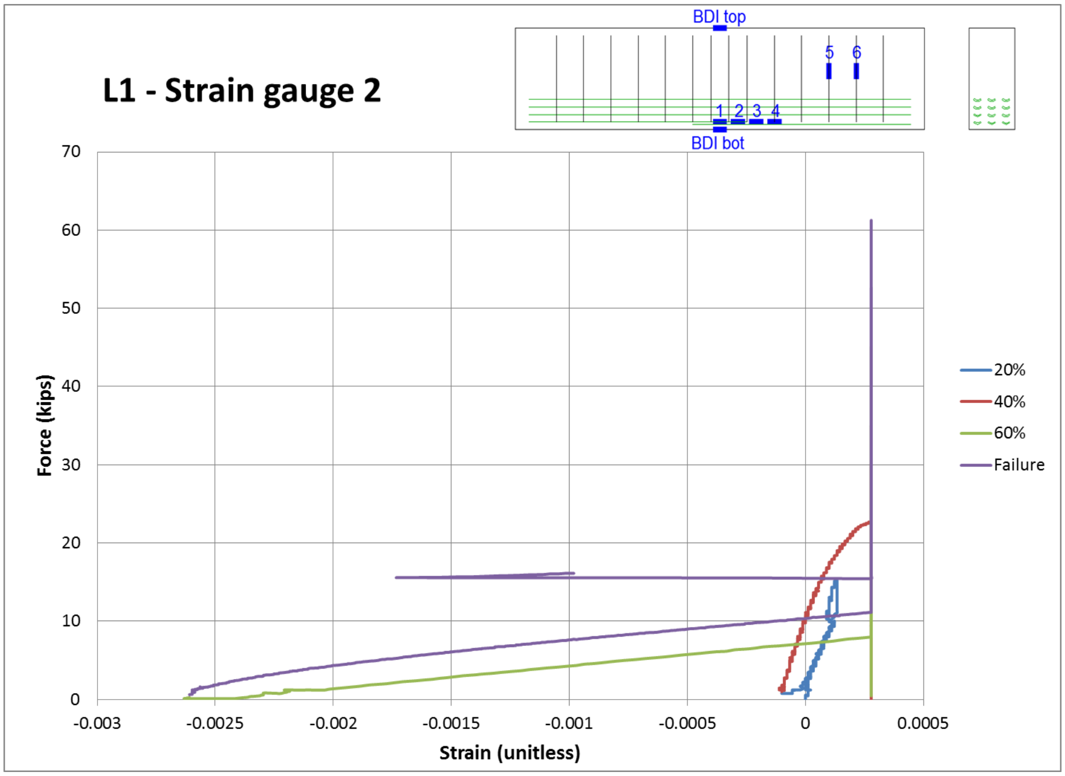
F4 - Strain gauge 7
BDI top
6,7
1,2,3
4,5
BDI bot
50
45
40
35
30
25
20
15
10
5
0
Force (kips)
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
Strain (unitless)
20%
40%
60%
Failure

Appendix G.3 Flexure lap-spliced beams

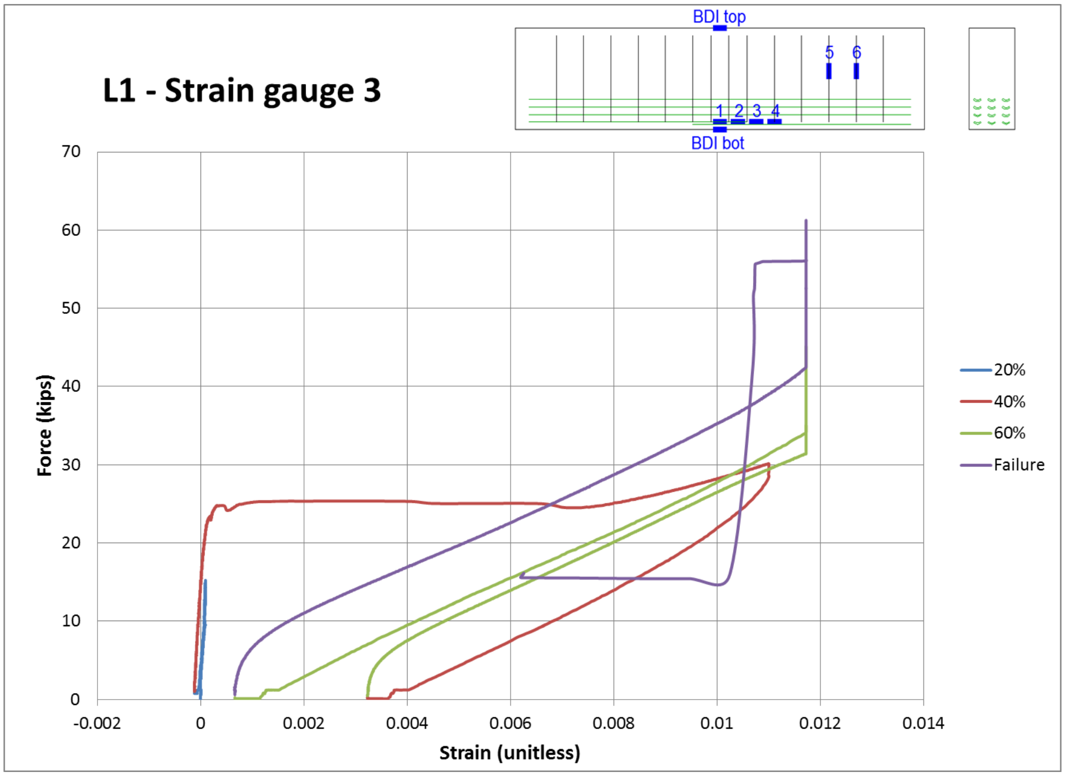



L1 - BDI bot
BDI top
5
6
1 2 3 4
BDI bot
35
30
25
20
15
10
5
0
Force (kips)
-0.0006
-0.0004
-0.0002
0
0.0002
0.0004
0.0006
0.0008
0.001
Strain (unitless)
20%
40%

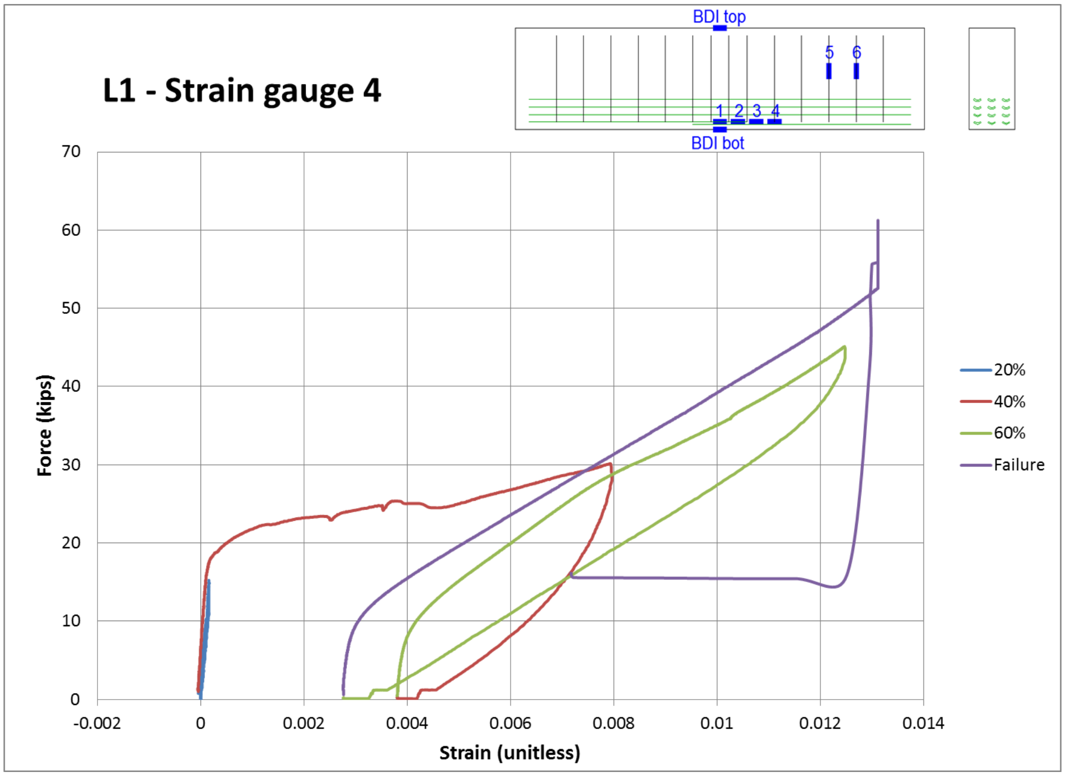

L1 - Strain gauge 1
BDI top
5
6
1 2 3 4
BDI bot
70
60
50
40
30
20
10
0
Force (kips)
-0.0014
-0.0012
-0.001
-0.0008
-0.0006
-0.0004
-0.0002
0
0.0002
Strain (unitless)
20%
40%
60%
Failure

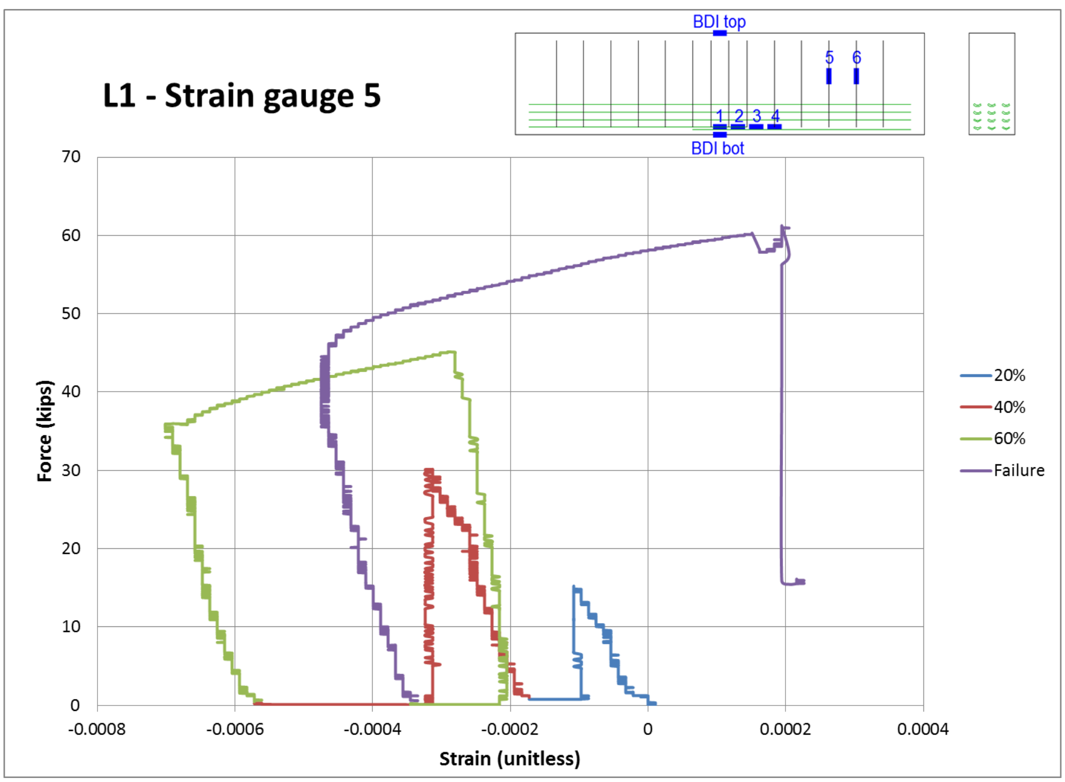


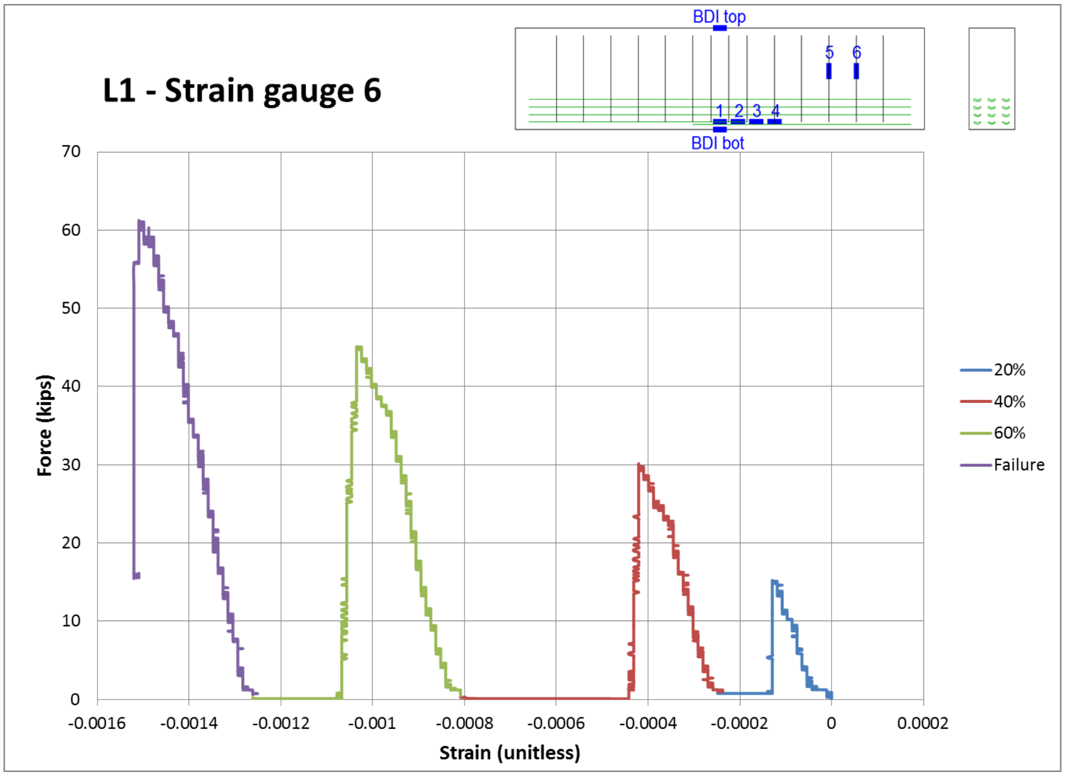
L1 - Strain gauge 2
BDI top
BDI bot
1
2
3
4
5
6
Force (kips)
Strain (unitless)
20%
40%
60%
Failure



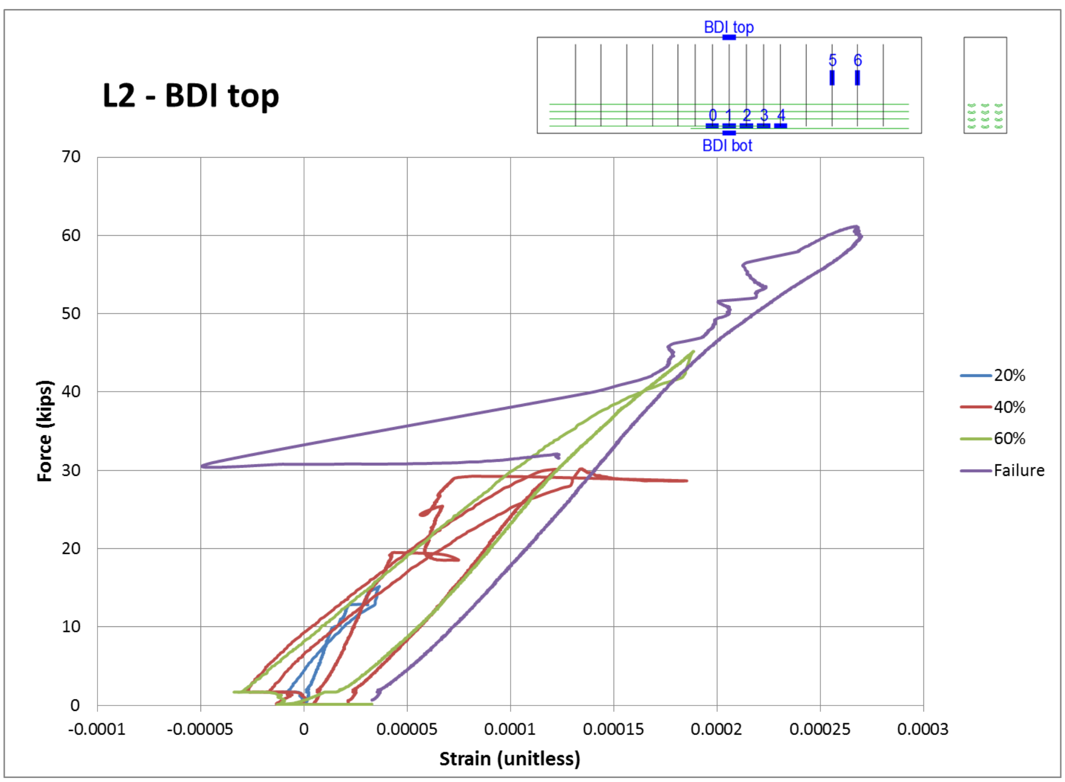
L1 - Strain gauge 3
BDI top
BDI bot
1
2
3
4
5
6
Force (kips)
Strain (unitless)
20%
40%
60%
Failure


L1 - Strain gauge 4
BDI top
5
6
1 2 3 4
BDI bot
Force (kips)
70
60
50
40
30
20
10
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
Strain (unitless)
20%
40%
60%
Failure

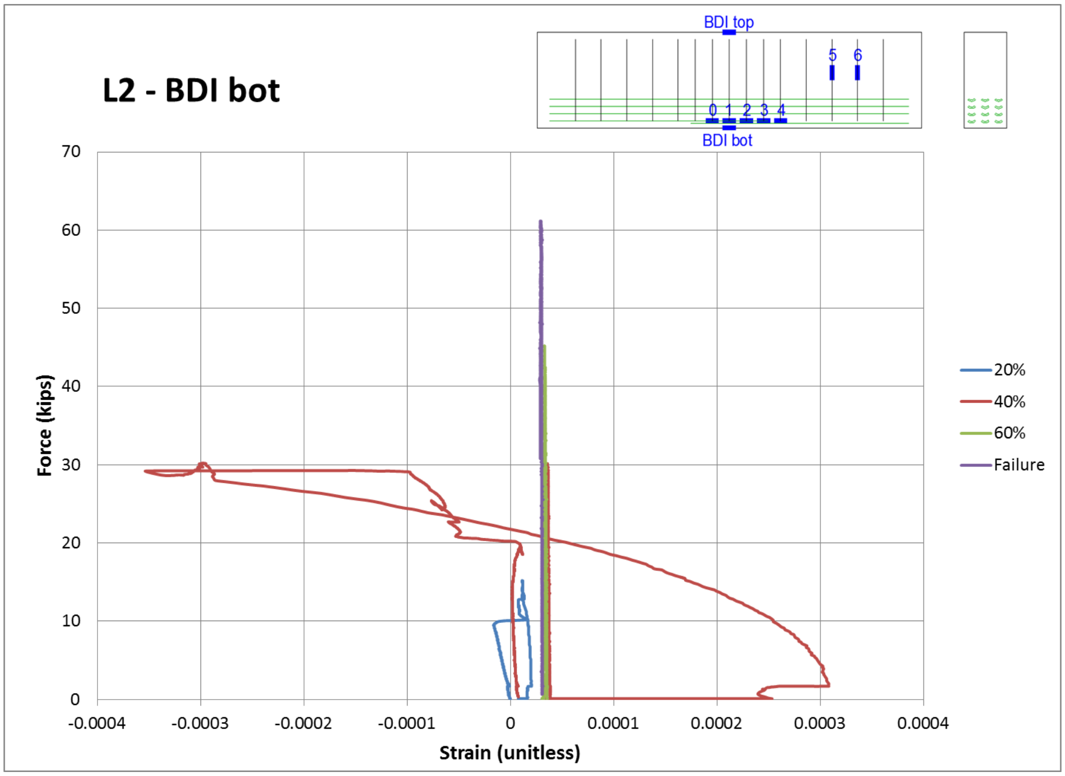

L1 - Strain gauge 5
BDI top
5
6
1 2 3 4
BDI bot
Force (kips)
70
60
50
40
30
20
10
0
-0.0008
-0.0006
-0.0004
-0.0002
0
0.0002
0.0004
Strain (unitless)
20%
40%
60%
Failure

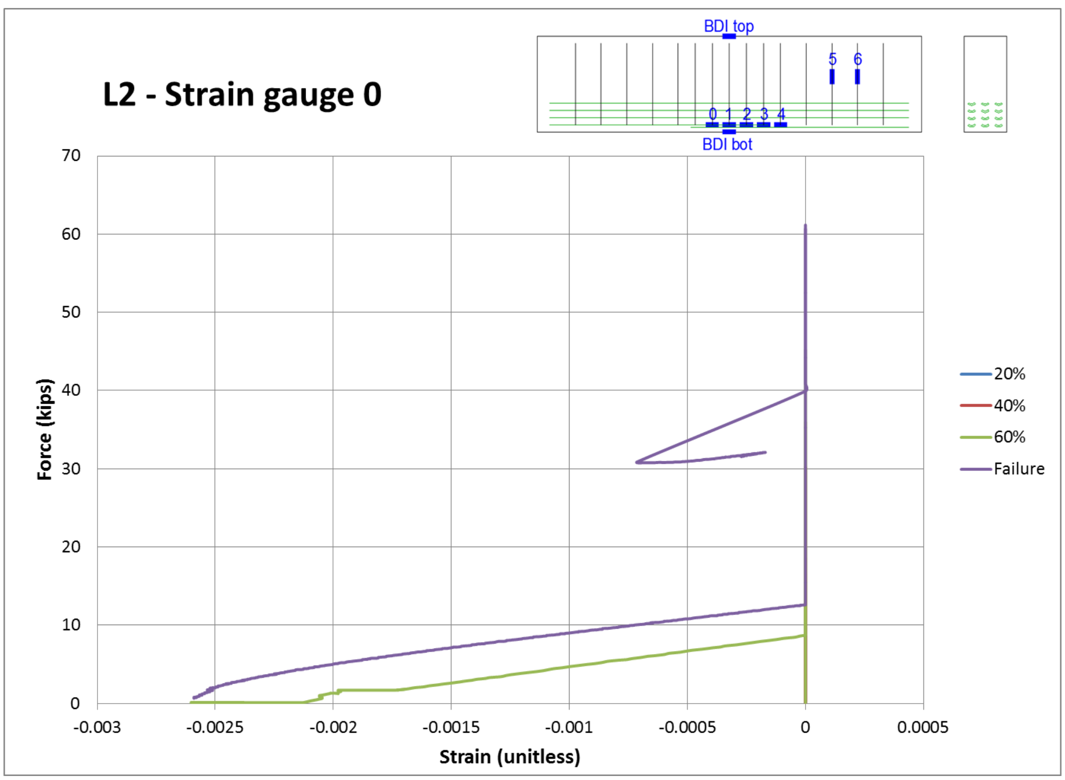


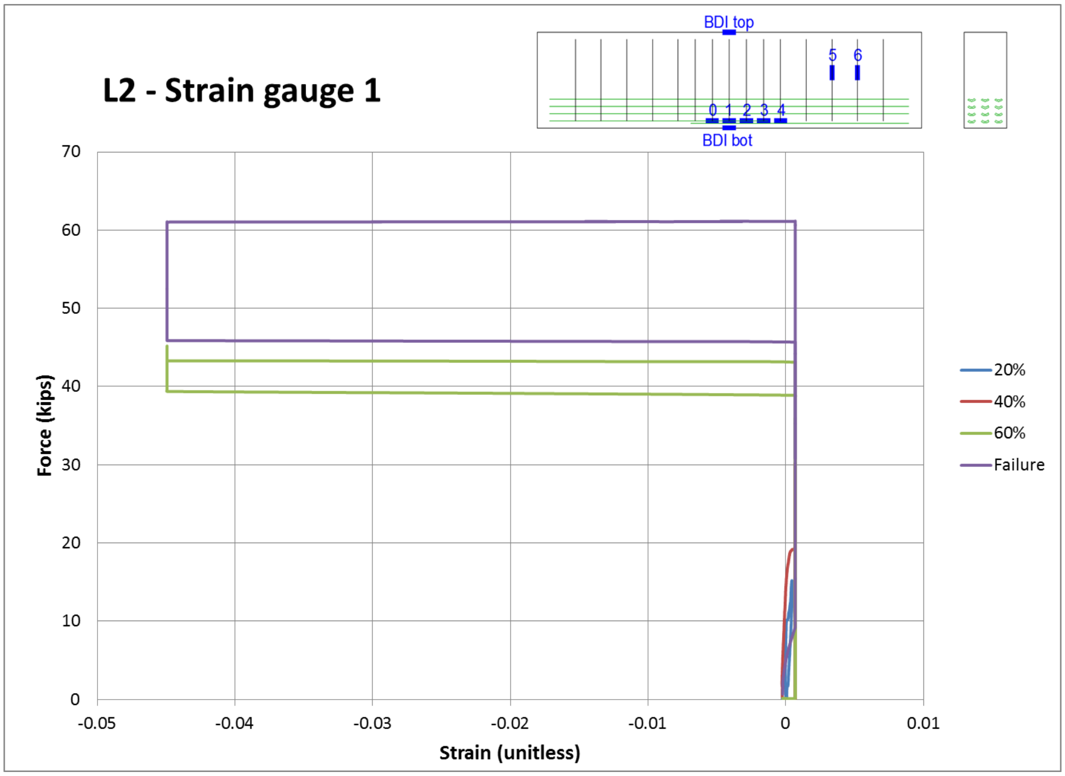
L1 - Strain gauge 6
BDI top
BDI bot
1 2 3 4
5 6
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.0016
-0.0014
-0.0012
-0.001
-0.0008
-0.0006
-0.0004
-0.0002
0
0.0002
20%
40%
60%
Failure



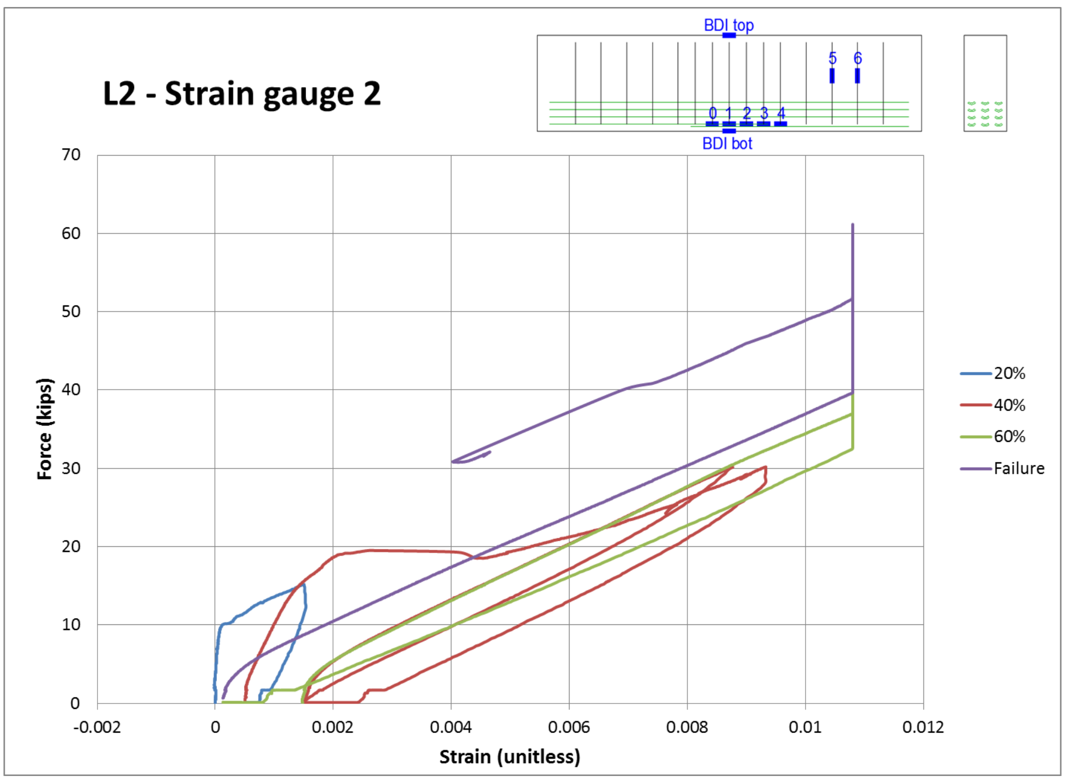
L2 - BDI top
BDI top
BDI bot
0 1 2 3 4
5 6
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.0001
-0.00005
0
0.00005
0.0001
0.00015
0.0002
0.00025
0.0003
20%
40%
60%
Failure


L2 - BDI bot
BDI top
BDI bot
0 1 2 3 4
5 6
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

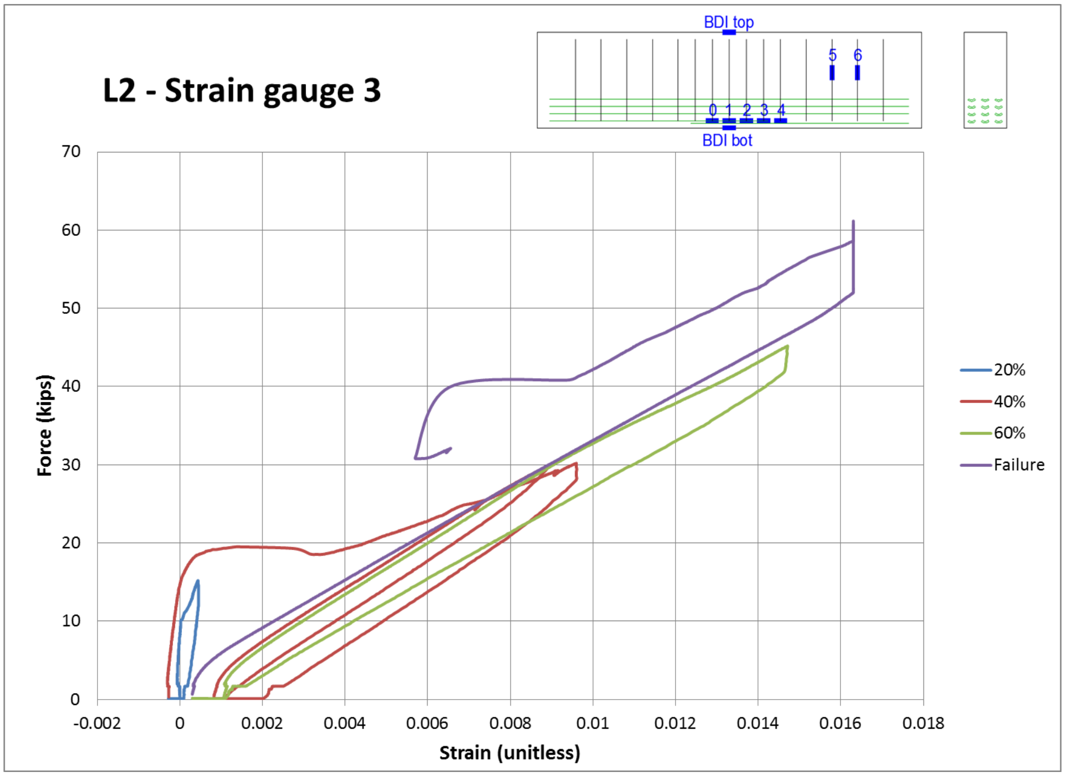

L2 - Strain gauge 0
BDI top
BDI bot
0 1 2 3 4
5 6
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

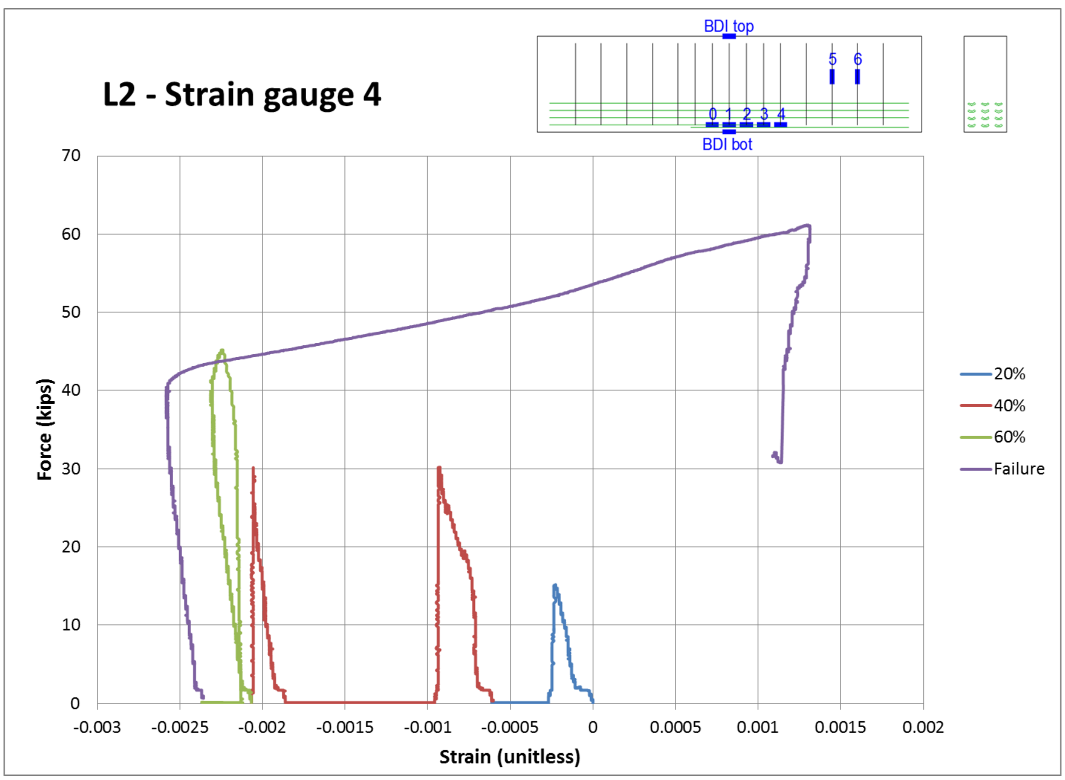


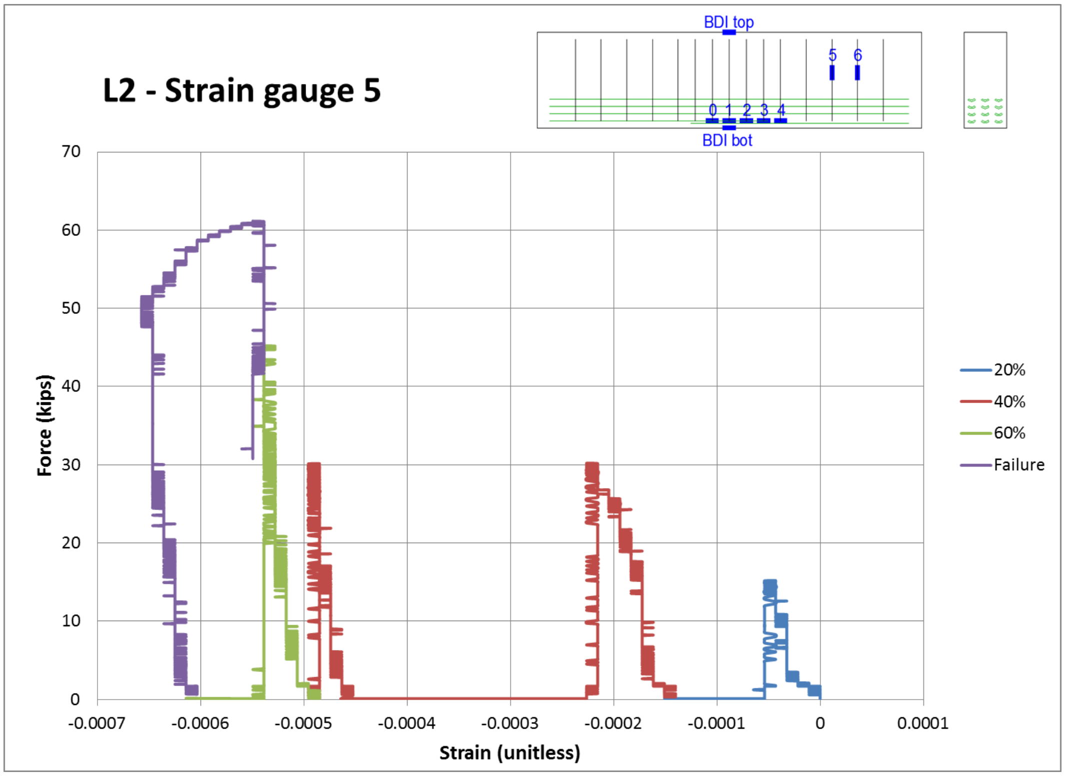
L2 - Strain gauge 1
BDI top
BDI bot
0 1 2 3 4
5
6
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.05
-0.04
-0.03
-0.02
-0.01
0
0.01
20%
40%
60%
Failure



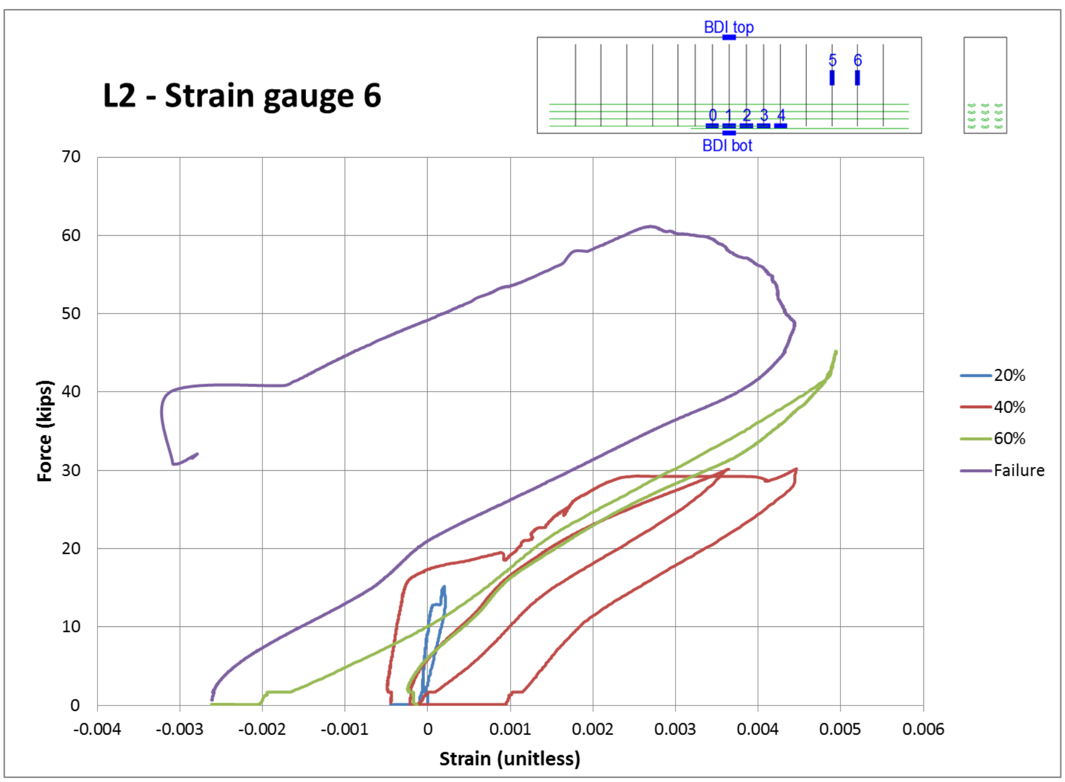
L2 - Strain gauge 2
BDI top
BDI bot
0 1 2 3 4
5
6
Force (kips)
Strain (unitless)
70
60
50
40
30
20
10
0
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
20%
40%
60%
Failure


L2 - Strain gauge 3
BDI top
5
6
0 1 2 3 4
BDI bot
70
60
50
40
30
20
10
0
Force (kips)
-0.002
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
Strain (unitless)
20%
40%
60%
Failure

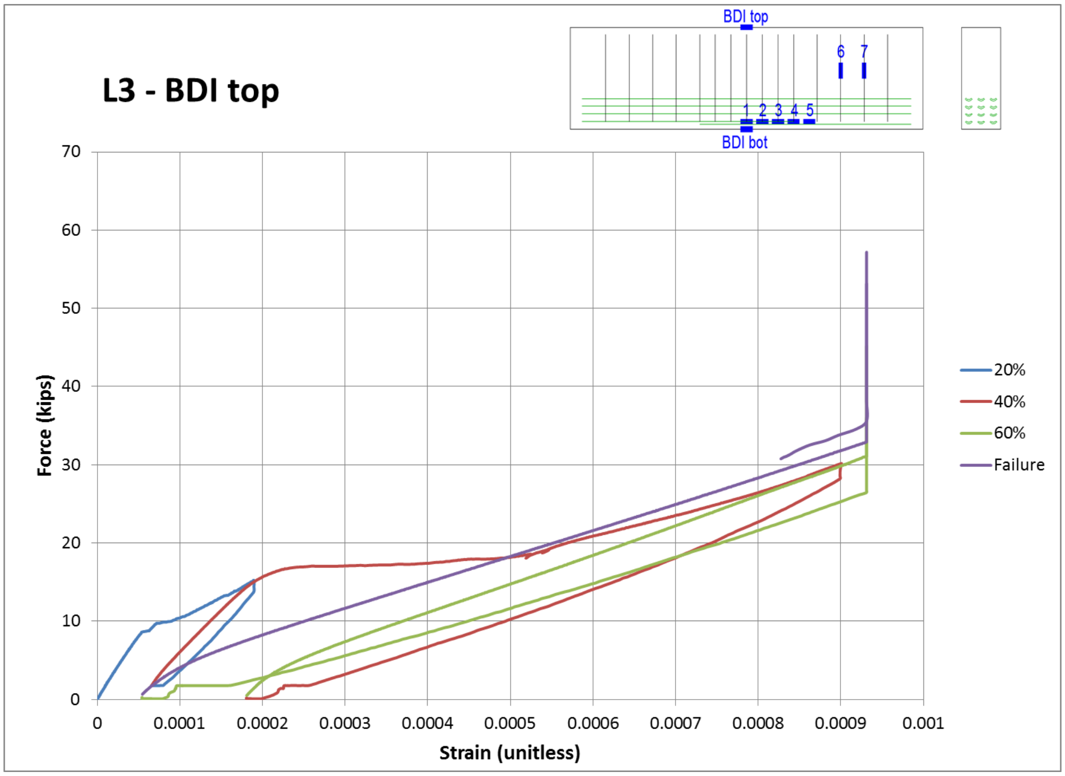

L2 - Strain gauge 4
BDI top
5
6
0 1 2 3 4
BDI bot
70
60
50
40
30
20
10
0
Force (kips)
-0.003
-0.0025
-0.002
-0.0015
-0.001
-0.0005
0
0.0005
0.001
0.0015
0.002
Strain (unitless)
20%
40%
60%
Failure

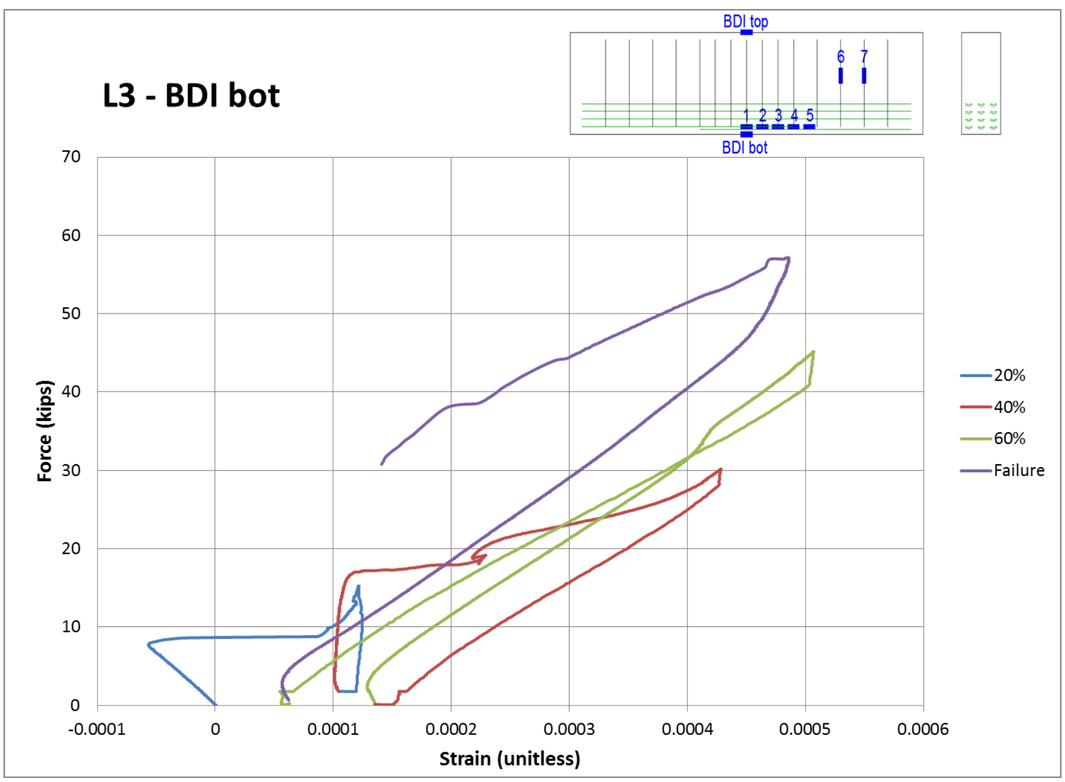


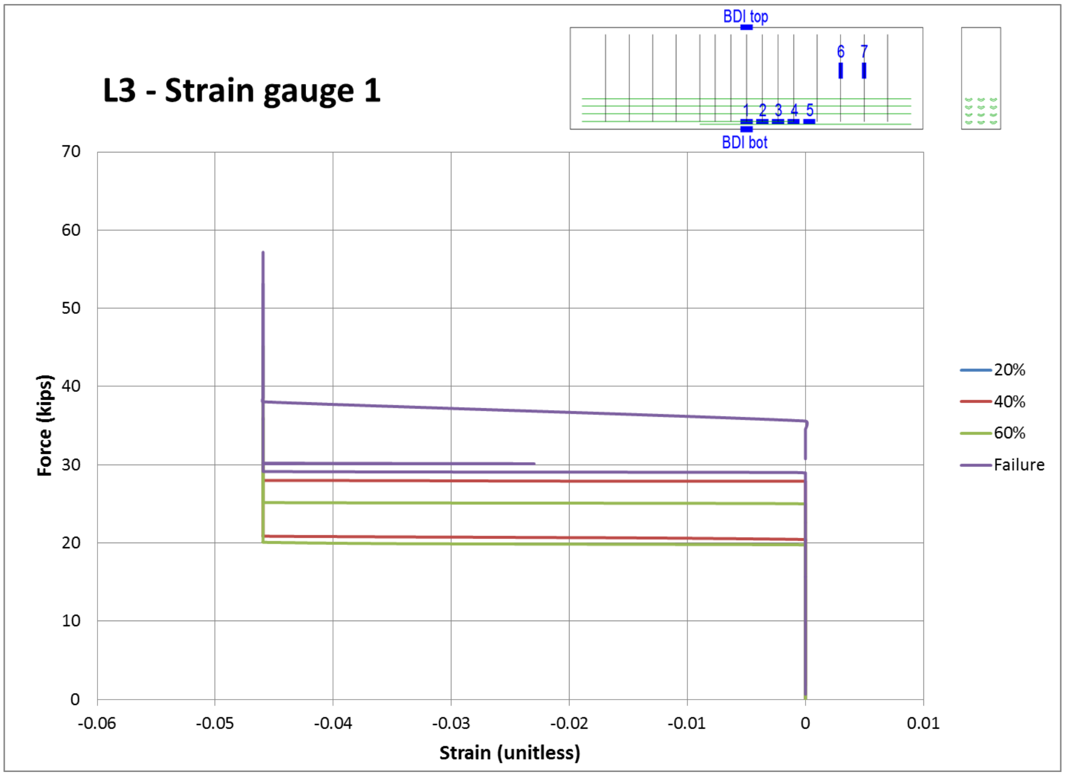
L2 - Strain gauge 5
BDI top
5
6
0 1 2 3 4
BDI bot
70
60
50
40
30
20
10
0
-0.0007
-0.0006
-0.0005
-0.0004
-0.0003
-0.0002
-0.0001
0
0.0001
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

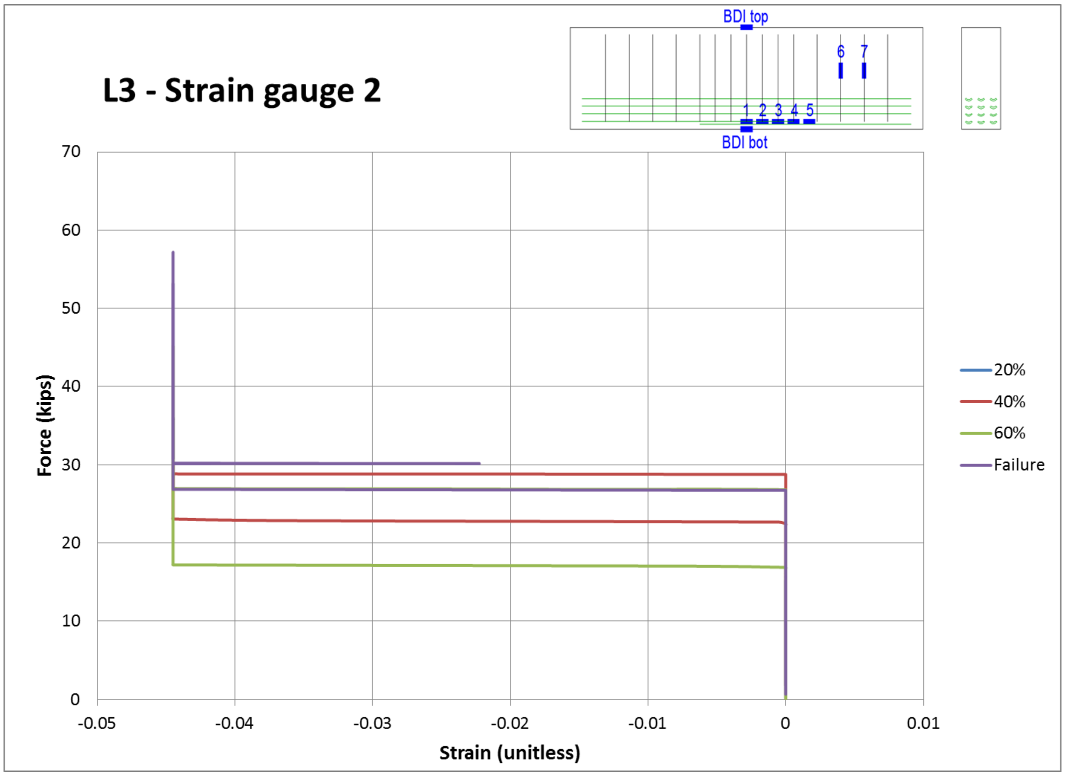


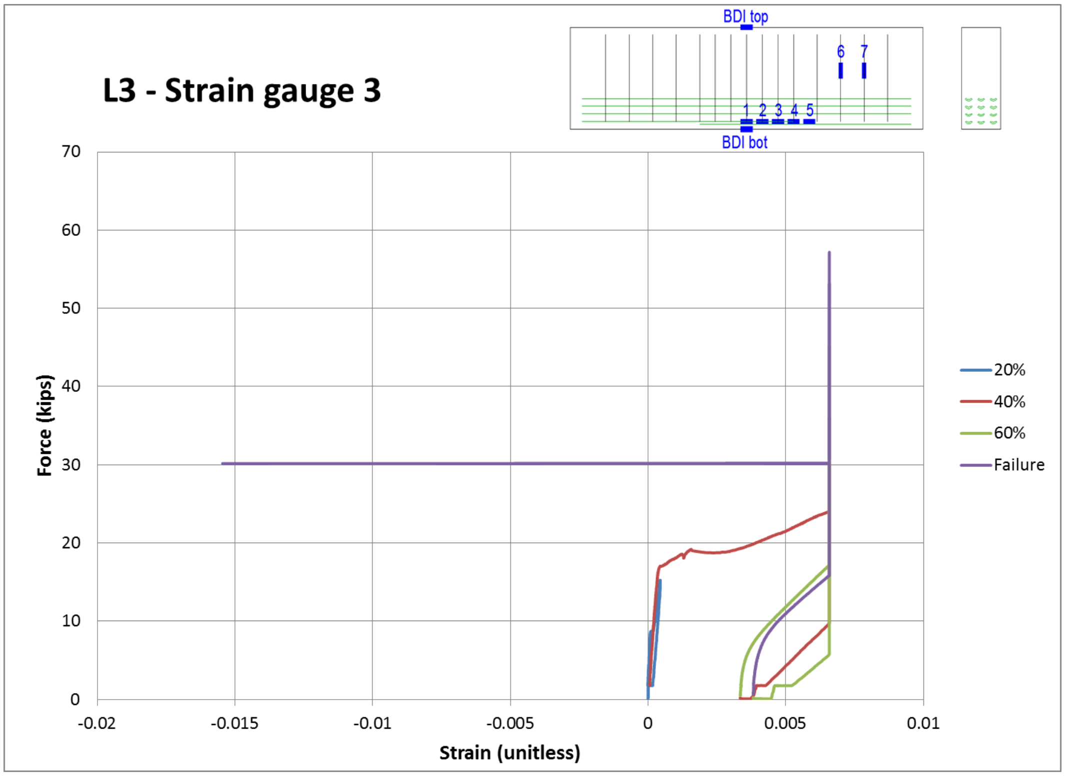
L2 - Strain gauge 6
BDI top
5
6
0 1 2 3 4
BDI bot
70
60
50
40
30
20
10
0
-0.004
-0.003
-0.002
-0.001
0
0.001
0.002
0.003
0.004
0.005
0.006
Force (kips)
Strain (unitless)
20%
40%
60%
Failure

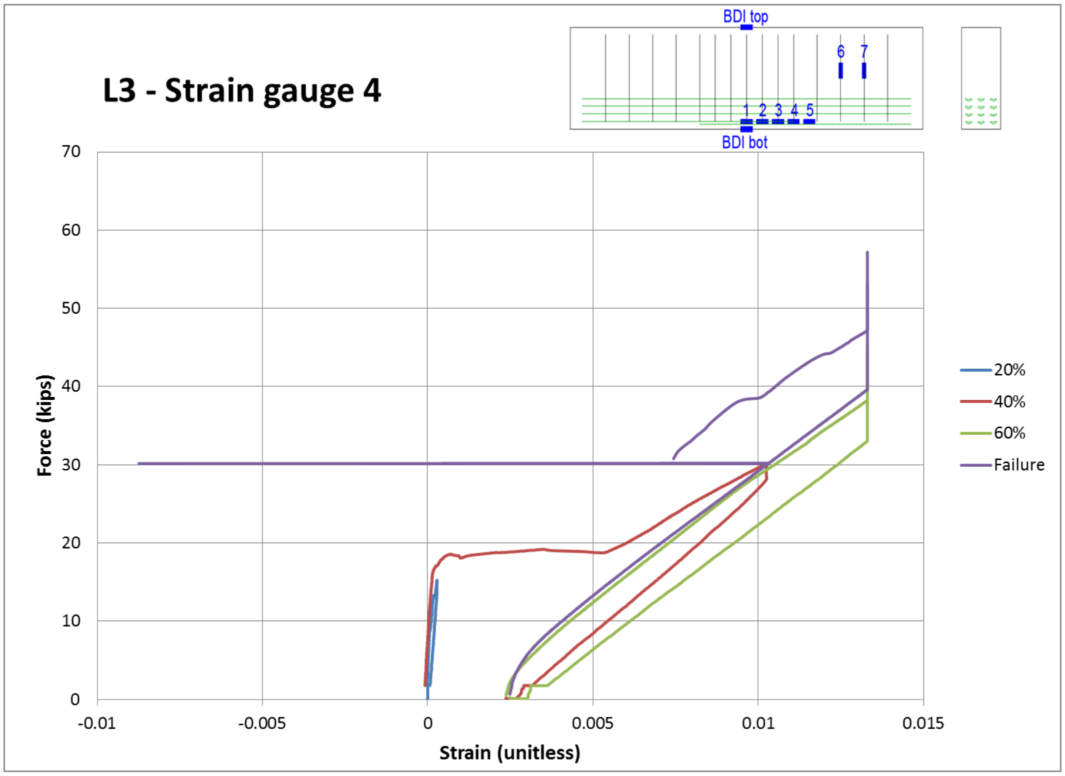

L3 - BDI top
BDI top
BDI bot
1 2 3 4 5
6 7
Force (kips)
Strain (unitless)
0
10
20
30
40
50
60
70
0
0.0001
0.0002
0.0003
0.0004
0.0005
0.0006
0.0007
0.0008
0.0009
0.001
20%
40%
60%
Failure

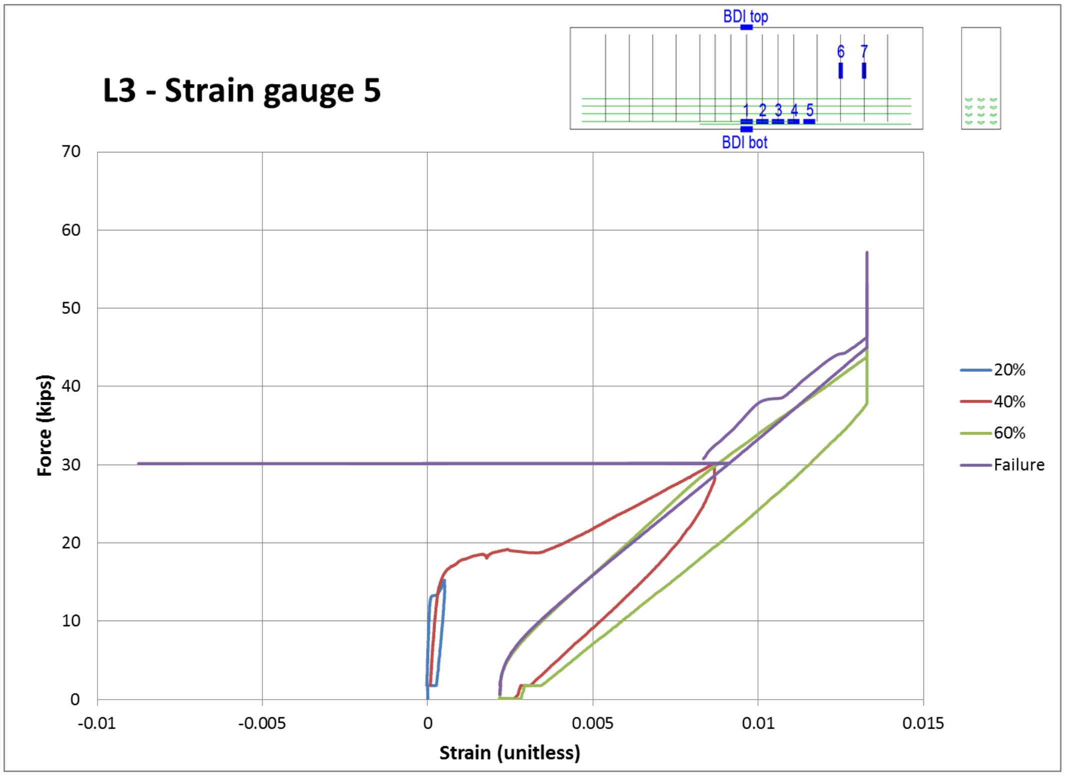

L3 - BDI bot
BDI top
BDI bot
1 2 3 4 5
6 7
Force (kips)
Strain (unitless)
0
10
20
30
40
50
60
70
-0.0001
0
0.0001
0.0002
0.0003
0.0004
0.0005
0.0006
20%
40%
60%
Failure

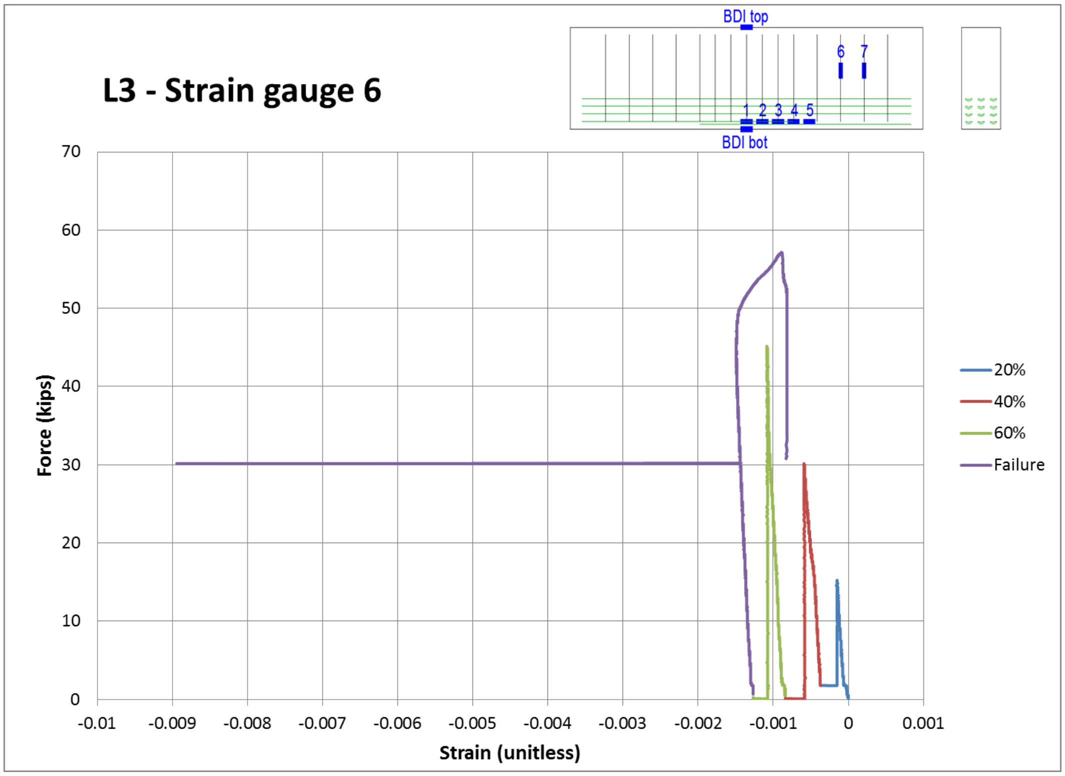

L3 - Strain gauge 1
BDI top
BDI bot
1
2
3
4
5
6
7
Force (kips)
Strain (unitless)
0
10
20
30
40
50
60
70
-0.06
-0.05
-0.04
-0.03
-0.02
-0.01
0
0.01
20%
40%
60%
Failure

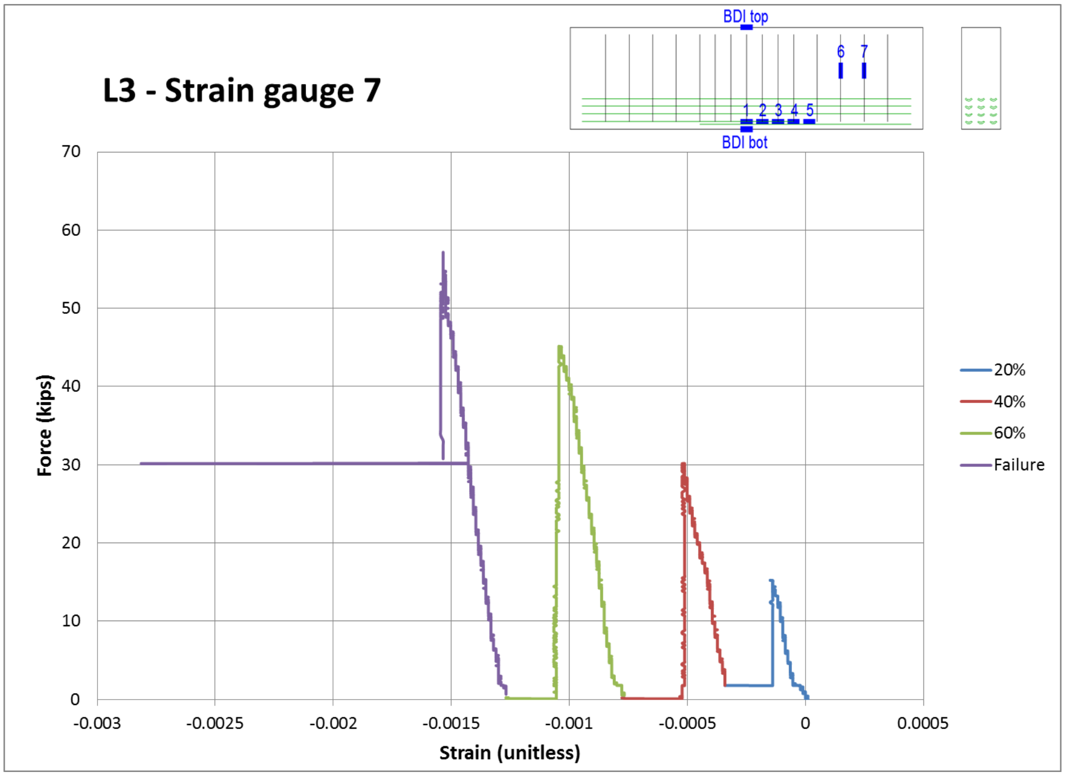

L3 - Strain gauge 2
BDI top
BDI bot
1
2
3
4
5
6
7
Force (kips)
Strain (unitless)
0
10
20
30
40
50
60
70
-0.05
-0.04
-0.03
-0.02
-0.01
0
0.01
20%
40%
60%
Failure


L3 - Strain gauge 3
BDI top
6
7
1 2 3 4 5
BDI bot
Force (kips)
70
60
50
40
30
20
10
0
-0.02
-0.015
-0.01
-0.005
0
0.005
0.01
Strain (unitless)
20%
40%
60%
Failure


L3 - Strain gauge 4
BDI top
6
7
1 2 3 4 5
BDI bot
Force (kips)
70
60
50
40
30
20
10
0
-0.01
-0.005
0
0.005
0.01
0.015
Strain (unitless)
20%
40%
60%
Failure


L3 - Strain gauge 5
BDI top
6
7
1
2
3
4
5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure


L3 - Strain gauge 6
BDI top
6
7
1
2
3
4
5
BDI bot
Force (kips)
Strain (unitless)
20%
40%
60%
Failure



L3 - Strain gauge 7
BDI top
6
7
1 2 3 4 5
BDI bot
70
60
50
40
30
20
10
0
Force (kips)
-0.003
-0.0025
-0.002
-0.0015
-0.001
-0.0005
0
0.0005
Strain (unitless)
20%
40%
60%
Failure

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