Use Authorization

In presenting this thesis in partial fulfillment of the requirements for an advanced degree at Idaho State University, I agree that the Library shall make it freely available for inspection. I further state that permission to download and/or print my thesis for scholarly purposes may be granted by the Dean of the Graduate School, Dean of my academic division, or by the University Librarian. It is understood that any copying or publication of this thesis for financial gain shall not be allowed without my written permission.

Signature ______________________________

Date __________________________________

MECHANICAL BEHAVIOR OF HIGH EARLY STRENGTH CONCRETE WITH POLYPROPYLENE FIBERS FOR FIELD-CAST CONNECTIONS OF BRIDGE PRECAST ELEMENTS

by

Maximilian Casanova

A thesis

submitted in partial fulfillment

of the requirements for the degree of

Master of Science in the Department of Civil and Environmental Engineering

Idaho State University

Summer 2018

Committee Approval

To the Graduate Faculty:

The members of the committee appointed to examine the thesis of MAXIMILIAN CASANOVA find it satisfactory and recommend that it be accepted.

Major Advisor

Dr. Arya Ebrahimpour

Committee Member

Dr. Mustafa Mashal

Graduate Faculty Representative

Dr. Anish Sebastian

Acknowledgments

Thank you to all those who helped me throughout this project. I could not have done it without the support of my family, colleagues, and professors.

I would like to express gratitude to my advisor, Dr. Ebrahimpour, for his guidance. I would like thank Chris Clauson who assisted me in the lab. Special thanks to the individuals who helped contribute to this project and the volunteers that aided during the “Pour day.” Thank you to Pocatello Ready Mix for their generous donation of materials to complete this project. I would like to thank Mike Horrocks, from Pocatello Ready Mix, for his guidance with the mix designs. I would like to thank my parents, Rick and Peggy Casanova, for their love and support.

This project would not have been possible without the financial support of Idaho Transportation Department and Federal Highway Administration.

Table of Contents

List of Figures

List of Tables

Thesis Abstract

Mechanical Behavior of High Early Strength Concrete with Polypropylene Fibers for Field-Cast Connections of Bridge Precast Elements

Idaho State University (2018)

Idaho Transportation Department is searching for an alternative connection detail to be used in Accelerated Bridge Construction precast deck elements in order to reduce costs and construction time versus the currently used Ultra High Performance Concrete (UHPC) connections. High early-strength (HES) concrete with polypropylene fibers was selected as the material to be studied for this research project. The cost of using HES concrete is comparable to conventional concrete, with estimated cost savings ranging from $50,000 to $100,000 for a typical highway bridge construction, over the use of UHPC. Six different HES concrete mixes were tested for compressive strength, splitting tensile strength, and shrinkage. The optimum mix was tested with precast concrete segments for interface bond strength and headed bar pullout strength. Precast panels with the non-contact lap splice closure pour detail were tested in three-point and four-point bending. The optimum mix, HES-D, had a compressive and splitting tensile strength of 8,864 psi and 785 psi, respectively. This mix also had lower long-term shrinkage (522 microstrain) compared to UHPC. Bond strength between precast and HES-D was 612 psi, which is comparable to that of UHPC (712 psi). The average headed bar pullout strength was 12.5 kips and flexural beams had an average ultimate moment capacity of 147 kip-inch.

Key Words: High-early strength concrete; Precast bridge connections; Alternative bridge concrete; Accelerated Bridge Construction

CHAPTER 1 INTRODUCTION

1.1 Background and Motivation

Bridges serve as an integral part of the infrastructure in the United States. There are approximately 600,000 bridges in the United States, with more than 12,000 over 100 years old (Federal Highway Administration 2018). Over 55,000 of these bridges are structurally deficient or functionally outdated. These bridges are in desperate need of rehabilitation or replacement. The approximate cost of replacing or retrofitting these bridges is estimated to be $32-$47 billion. In addition, the cost of new bridge construction is high.

In an effort to be more cost-efficient, without compromising standards or safety for bridge construction, various cost-reducing methods to replace or rehabilitate bridges have been, and are currently being, sought. Recently, Accelerated Bridge Construction (ABC), a project planning method that aims to reduce the construction time of bridges, has shown to be beneficial to meet bridge replacement needs. Other benefits of utilizing ABC are reducing traffic impacts to the community, maintaining bridge quality, and promoting construction safety. When implemented properly, ABC can produce higher quality bridges, faster and cheaper when compared to conventional bridge-construction methods (Culmo 2011).

An important component of highway bridge construction is the connections between bridge sections, also known as the concrete closure pour detail. One part of this connection involves pouring the closure concrete. Figure 1.1 shows an example of a closure pour between deck bulb-T girders. Concrete closure pour details are cast using a variety of mediums, including normal-weight concrete (NWC), grout, and ultra-high performance concrete (UHPC).

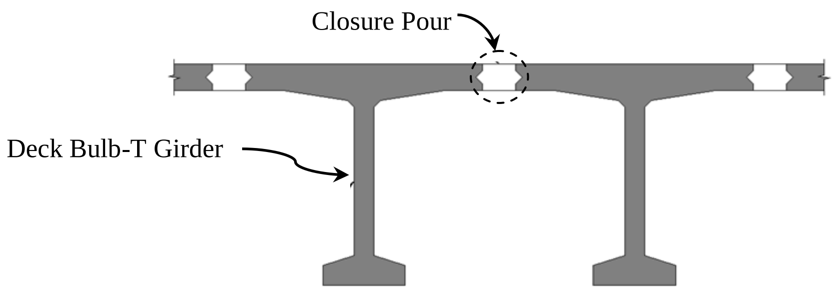


Figure 1.1 Example Closure Pour between Deck Bulb-T Girders

1.2 Problem Statement and Scope

Connection details, specifically the closure pours, for highway bridges have utilized UHPC because of its exceptional material properties. Although UHPC is becoming more widely used, the drawbacks include considerable cost and time. It also involves a labor-intensive construction process, utilizing a larger mixing crew, with portable mixers, who hand pour the concrete. In comparison, pouring normal-weight concrete only requires a relatively small construction crew using a mixing truck and pump to fill connections. ITD bridge engineers estimate cost savings of NWC compared to UHPC can be over $100,000 per project.

Taking into consideration cost and constructability, a more affordable solution is desired. Likewise, ITD is searching for an alternative connection detail for ABC in order to substantially reduce costs and construction time. A practical option is to explore the use of high early-strength (HES) concrete as opposed to UHPC construction.

1.3 Objectives

Bridge costs, specifically associated with connections, can be reduced by redesigning the connection detail, altering the construction process, and/or modifying the concrete mix design. This study will focus on modifying concrete mixes to determine a suitable and cost-effective alternative. Building on past literature, which investigated concrete material types and dosages, this study tests a specific un-researched concrete mix to use in bridge connection details. The proposed concrete has potential benefits in terms of cost and construction. This thesis intends to answer the following question: Is the material behavior and cost of high early-strength concrete with polypropylene fibers an effective alternative for field-cast connections of precast bridge elements in accelerated bridge construction?

Specific objectives of this research project are to:

1. Design a HES concrete mix class 50AF with addition of polypropylene fibers.
2. Determine the material properties of the closure pour material (compressive and tensile strength, shrinkage behavior, and bond strength).
3. Determine the pullout strength of the headed rebars.
4. Perform strength tests of beams with closure pour.

Laboratory experimentation is the primary method being used to complete the objectives for this research project. In order to determine an optimum mix for a cost-effective field-cast connection of precast bridge elements in ABC, a literature review about the various aspects of this study was completed. Next, applicable research methodologies were followed so that appropriate laboratory testing could be conducted.

1.4 Thesis Overview

This research is the culmination of a two-year project working in conjunction with ITD to design a cost-effective concrete mix that could be used as an alternative material for field-cast connections of precast elements in accelerated bridge construction. It is divided into six chapters.

1. Introduction: A brief overview of the project background and motivation as well as a description of the scope and objectives of this research project.

2. Literature Review: This chapter discusses the literature that was relevant to this project. This included related research, testing methods, materials, and sample preparation.
3. Methodology: Presented in this chapter are the testing methodologies that were used to carry out the experimental work. Appropriate testing standards were followed along with other non-standardized tests.
4. Description of Instruments: Descriptions of the devices and sensors that were used over the course of the project are discussed in this chapter. These instrumentation include strain gages, length sensors, load cells. Specimen instrumentation is also presented.
5. Results: This chapter presents the experimental results along with the analysis that was used to calculate rebar force or beam moment. Included are results for material properties (compression, split tensile, and shrinkage), interface bond strength, headed bar pullout tests, and large flexural beam tests. Trends in the data, average values, and comparisons between theoretical values are discussed.
6. Summary, Conclusions, and Future Work: The last chapter summarizes the experimental results that were presented in Chapter 5. Conclusions are discussed about the optimum mix and its benefits over UHPC. Future work involving computer modeling and bridge instrumentation are also discussed.

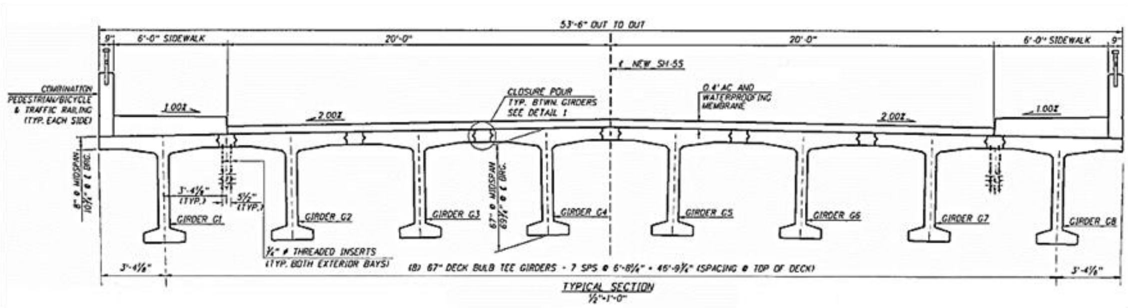
Also included are table of contents, figures, tables, and appendices. The appendices include material data sheets, experimental data, instrumentation information and procedures, and pictures.

CHAPTER 2 LITERATURE REVIEW

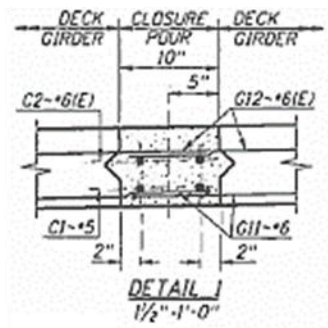
This literature review will examine prior research that is applicable to this thesis. Within this chapter are four main sections that focus on the key components of this research project followed by a summary. Section 1 gives an overview of the field-cast connections of prefabricated bridge elements (PBE). It includes components made with UHPC as well as alternative connection materials available, specifically HES concrete with fibers. Section 2 reviews literature relating to testing methods with a comprehensive breakdown of research done in areas of shrinkage and bond strength. Section 3 addresses the sample preparation of concrete specimens. This section reviews surface preparation as well as moisture at the interface. Section 4 examines the materials needed including bonding agent and polypropylene fibers. The chapter concludes with a summary of the review of literature as well as its application to this research.

2.1 Field-Cast Connections

Field-cast connections are a necessary component for ABC. These components connect the prefabricated bridge elements together. The most common materials utilized for field-cast connections are high-strength grouts and UHPC. Field-cast connections have been used for a variety of bridge components including cap beams, bridge girders (i.e. deck bulb-T), and deck panels, among others. This research is limited to longitudinal connections between deck bulb-T girders. Depicted in Figure 2.1 are a typical cross-section of a prestressed concrete bridge and a view of the closure pour connection detail. The connection detail shows the interlacing rebars and closure pour concrete that connects the two girders. As the connection detail get more complex, rebar can start to limit the amount of space inside the closure, this causes congestion and can make construction and pouring more difficult. Two examples of connection details are shown in Figure 2.2. Ways to alleviate the rebar congestion problem are to design better connection details as well as to use materials suited to the application. Material flowability is important for consolidation and placement within the connection. Ideal materials for closure pours are flowable and self-consolidating.



(a) Typical Cross-Section View



(b) Closure Pour Detail

Figure 2.1 Drawing of SH-55 over Payette River Bridge in Cascade, Idaho. (NTS) (Drawings from the Idaho Transportation Department, Bridge Section).

(a) U-Bar (b) Interlaced Straight Bar

Figure 2.2 Sample Closure Connection Details (Haber et al. 2016)

Grout is used as a material for field-cast connections due to its good flowability. Non-shrink cementitious grout (NSCG) was developed to mitigate the shrinkage cracking seen in grout. Although grout has been used extensively, other materials have been used to reduce initial bridge construction costs or long-term costs. As a viable alternative in PBE connections, UHPC has increased in popularity with its application to ABC and is further discussed in the next section.

2.1.1 Ultra-High Performance Concrete – UHPC

Through research, standard tests have shown that UHPC is highly suited for applications in bridge construction. UHPC typically has compressive strength of 24 ksi, split tensile strength of 1.3 ksi, and long-term drying shrinkage of 550 microstrain (Graybeal 2014). These values far exceed typical NWC values of 4 ksi and 0.4 ksi, for compressive and tensile strengths, respectively. The high strength values, low shrinkage, and good durability of the UHPC make it an ideal material to use for connecting precast concrete bridge components. Although UHPC is gaining popularity, it is still not as readily available as NWC, and it is a proprietary product. The material cost of UHPC is high, ranging between $2,000 and $4,000 per cubic yard (De la Varga and Graybeal, 2016; Graybeal 2014). According to ITD the installation cost of UHPC is approximately $15,000 per cubic yard. Given the literature, UHPC will be used as a comparison to evaluate the proposed concrete mix in terms of performance and overall costs. A number of studies conducted by Graybeal (2010) have investigated the performance of UHPC in PBE connections. In the same study Graybeal examined the performance of large-scale panels with typical closure pour connections shown in Figure 2.2. In that study, the researchers subjected the panels to cyclic and static structural loading. Another study determined that UHPC outperformed grout in PBE connections (Haber et al. 2016). Researchers found similar results when comparing multiple UHPC concretes and NSCG (Haber and Graybeal 2016). Despite the advantages of UHPC, there still remains a cost-prohibitive element to its use.

2.1.2 Alternative Materials

A proposed alternative to the costly UHPC is HES concrete with fibers. According to ITD bridge engineers the cost of using HES concrete is comparable to conventional concrete ($600-$700 per cubic yard). The estimated cost saving of using HES over UHPC can range from $50,000 to $100,000. Although most research involving prefabricated bridge element connections focuses on the use of grout and UHPC materials for field-cast connections, there is insufficient research involving a more conventional HES concrete mix. The only significant research found was a study conducted for the Virginia Department of Transportation (VDOT) by Hoomes et al. (2017) which evaluated high-performance fiber-reinforced concrete (HPFRC) for bridge deck connections and joints, including closure pours. Their research was focused on the cracking and crack opening which occurs between link slabs in bridges. An example of a UHPC link slab connection detail is depicted in Figure 2.3.

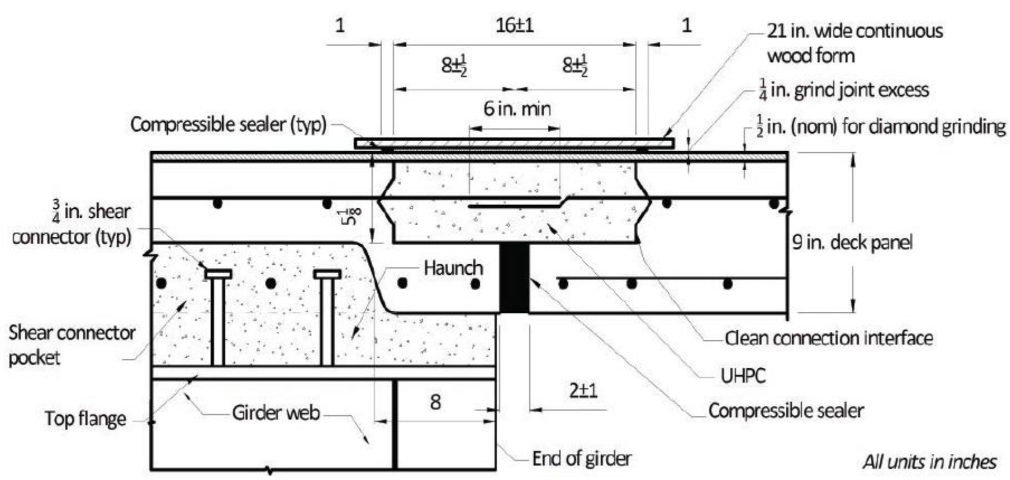


Figure 2.3 UHPC Link Slab Connection Detail on SR962G Bridge in Owego, NY (Graybeal et al. 2014)

Hoomes et al. determined the most advantageous mix based on performance and respective costs. Performance parameters included: bond strength, flexural toughness, deflection hardening, shrinkage, and fresh concrete properties. The study concluded that Hybrid Fiber Reinforced Concrete (HyFRC-G), which contained polypropylene and PVA, performed to specifications. The total fiber content for this mixture was 2% by volume (Hoomes et al. 2017). The current research project used much lower fiber contents, as will be discussed in Section 2.4, but their research presented applicable conclusions for the use of HES concrete. The higher dosage of fibers reduced workability and needed increased amounts of high-range water-reducing admixture, which caused segregation of aggregates. Although it was not the highest performing mix, it was the most economical and user-friendly. VDOT has shown an interest for alternative materials, specifically (HES) concrete, for use in bridges. VDOT has used HES concrete in connecting prefabricated deck components in several bridge projects.

2.2 Testing Methods

2.2.1 Shrinkage

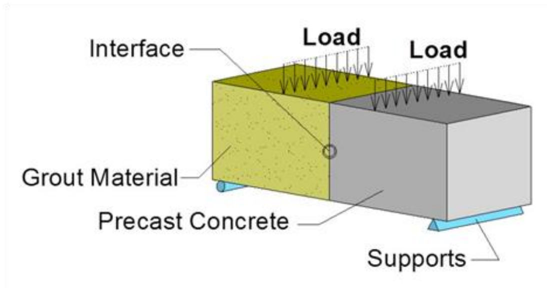
Shrinkage in concrete is the reduction in volume due to loss of water. This occurs at the early age, in its plastic state prior to hardening, and over long term, after the concrete hardens. Early-age shrinkage is also known as plastic shrinkage and the long-term shrinkage is mainly due to drying shrinkage (Mamlouk and Zaniewski 2017). Plastic shrinkage can be controlled by preventing the loss of water, from evaporation of surface moisture or absorption of concrete forms, until the concrete has set. Excessive volume change can cause cracks, known as shrinkage cracks, in the concrete which increases permeability. Minimizing shrinkage is important particularly in connections for PBE. Shrinkage reducing admixtures and fibers significantly decrease cracking.

The standard test method for shrinkage is ASTM C157, *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete* (ASTM 2017c). A prismatic beam of 3 in. x 3 in. cross-section and 11.25 in. length with gage studs embedded into the ends is used for this method. This test measures the length change after the concrete has hardened and cured for a set amount of time. Most often the drying shrinkage is reported. Graybeal (2014) reported the long-term shrinkage for UHPC at 550 microstrain, which was used as a comparison value for this study.

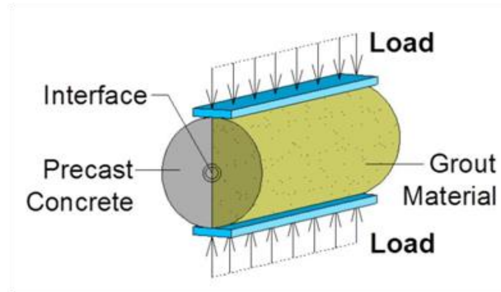
2.2.2 Bond

This section examines the methods used to evaluate bond strength and the different factors that contribute to bond behavior. Durability of connections utilized in PBE is critical to the overall performance of the bridge. Poor bond between precast concrete and field-cast concrete could result in cracks at the interface, allowing water to penetrate into the deck and cause damage. Some of the research discussed here investigates specific applications and may not directly apply to the current study but they will still be used to evaluate bond performance.

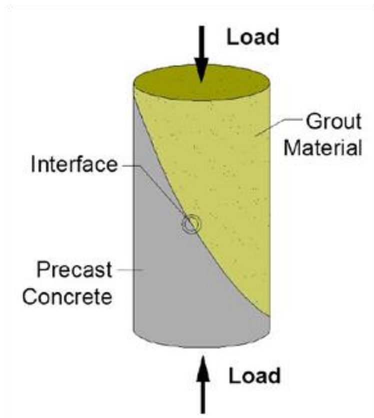
Common tests for determining bond strength are flexural beam, splitting cylinder, slant-shear, and direct tension pull-off (Bentz et al. 2017; De la Varga et al. 2017; Emmons 1994; Haber and Graybeal 2016; Silfwerbrand 2003; Swenty and Graybeal 2017; Yildirim et al. 2015). These tests are based on ASTM standard test methods. Bond strength tests are shown in Figure 2.4. All test specimens use segments of previously cast concrete (e.g. precast or another concrete base) and connection material (e.g. grout, UHPC, or new concrete). These tests are intended to determine the bond behavior of the interface between the two materials.



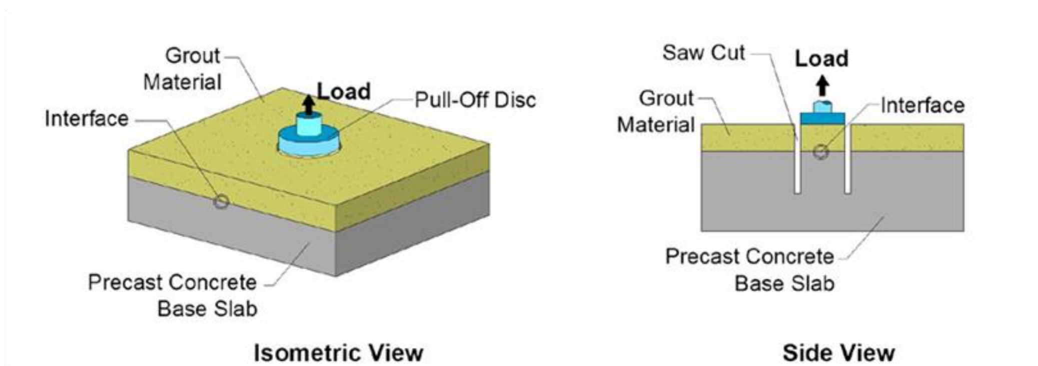
(a) Flexural beam test



(b) Splitting cylinder test



(c) Slant-shear compression test



(d) Direct tension pull-off test

Figure 2.4 Tests for Characterizing Bond between Precast Concrete and Field-Cast Connection Grouts (De la Varga et al. 2017)

Each test method puts the interface into a different state of stress (e.g. flexure, tensile, or shear). The flexural beam test (Figure 2.4a) is based on ASTM C78, *Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)* (ASTM 2018).

This test is performed on a 6 in. x 6 in. x 21 in. composite beam with one half containing the base material and the other half containing the closure material. Third-point loading is applied and puts the bottom portion of interface into a state of flexural tension. Splitting cylinder (Figure 2.4b) is based on ASTM C496, *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens* (ASTM 2017d). This test uses a 4 in. x 8 in. or 6 in. x 12 in. composite cylinder with halves consisting of the base material and the closure material. A load is applied along the length of the cylinder putting the interface into indirect tension. Slant-shear test (Figure 2.4c) is based on ASTM C882, *Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear* (ASTM 2013b). This test uses a slant cylinder in which the halves are cast such that a slant face is produced to apply a combination of shear and compression at the interface. The direct tension pull-off test (Figure 2.4d) follows ASTM C1583, *Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-Off Method)* (ASTM 2013c). Specimens consist of a base concrete slab and a 2 in. thick overlay of the connection material. A steel disc is then glued to the top surface and a partially-cored hole is drilled approximately 1 in. into the base concrete. This test puts the interface into direct tension. Bond strength can be difficult to determine because there are multiple testing methods that can be used, therefore a test method should be selected based on the anticipated stresses in the field. Due to the number of variations that can contribute to bond strength, some researchers investigating bond performance have employed several of the test methods of Figure 2.4 in their studies (Bentz et al. 2017; De la Varga et al. 2017; Julio et al. 2004). The optimal test method for determining bond strength is the flexural beam test, ASTM C78, because it best simulates loading in the field.

Some research has concluded that these tests may not be representative of the true bond strength due to factors such as: precast substrate surface preparation, pre-wetting substrate, surface cleanliness, differential shrinkage, and differential stiffness (Bentz et al. 2017; De la Varga et al. 2017; Santos and Julio 2011). Some of these factors will be discussed in the next section.

2.3 Precast Concrete Interface Surface Preparation

This section will present literature that is geared toward improving bond strength. Among the numerous factors contributing to bond strength between precast concrete and closure material, two of the most critical are surface roughness and moisture at the interface.

2.3.1 Surface preparation

Substrate refers to the base concrete material. Researchers agree that substrate surface preparation contributes significantly to bond strength (De la Varga et al. 2016; De la Varga et al. 2017; Garbacz et al. 2004; Julio et al. 2004; Santos and Julio 2011; Tayeh et al. 2013). One component to surface preparation involves roughening of the substrate surface which increases the contact area for the new concrete to bond. In general, research shows that the greater the surface roughness, the higher the bond strength (De la Varga et al. 2017). Surface preparation methods vary; including among others, wire brushing, jack hammering, pressure washing, sand blasting, and exposing the aggregate of the precast concrete (De la Varga et al. 2016; Tayeh et al. 2013). Bond strength, effort needed to achieve surface roughness (i.e. the ease of implementation), and practical considerations for field application were factors used to compare results. It was determined that exposed aggregate surface preparation had the best rating based on the previous work.

2.3.2 Moisture at the interface

Presence of moisture also affects bond strength (De la Varga et al. 2017a; De la Varga et al. 2017b; Bentz et al. 2017; Julio et al. 2005; Emmons 1994). This refers to the surface moisture prior to casting the connection concrete. Research has revealed opposing conclusions about the need for moisture at the substrate, which is often attributed to the testing method used. Bentz et al. (2017) conducted tests on sets of slant-shear and direct tension specimens that were prepared with different substrate finishes and moisture conditions. In the study, Bentz et al. employed neutron and X-ray radiography to examine the dynamic microstructural rearrangements occurring at the interface during curing (i.e. identifying the water movement and densification between the two materials). The two bond tests, slant-shear and direct tension, produced different results. Slant-shear tests resulted in higher bond strength compared to direct pull-off tests when the substrate was dry. Conversely, when the substrate was saturated surface dry (SSD), direct pull-off tests had higher bond strengths compared to slant-shear (Bentz et al.). For the dry substrate case, the flow of water from the repair material (RM) to the substrate causes densification of the layer and this may be the cause of higher slant-shear results. For the moist condition, the excess water provided for better hydration and consolidation for the RM resulting in higher direct pull-off values.

2.4 Materials

2.4.1 Bonding agent

A bonding agent is a liquid compound, latex or epoxy based, applied to a concrete substrate to promote good bonding to new or repaired concrete. Literature involving use of bonding agents was intended for concrete repair and overlay material applications. Emmons (1994) suggests not using a bonding agent because it can produce a vapor barrier that could result in failure. Another experimental study consisted of preparing sets of slant-shear specimens with each pair treated with a different substrate finish (e.g. as-cast, wire-brushing, partially chipped, and sand-blasted) (Julio et al. 2005). The research group concluded that a bonding agent is not necessary, provided the surface roughness is adequate. The sand-blasted specimen without a bonding agent performed better than the specimen with a bonding agent. From the literature, use of bonding agents may not be beneficial, particularly because the applications for bonding agents were mainly for repair or overlays. These types of applications induce different stresses (i.e. shear and direct tension) at the interface than those stresses (flexural tension) related to this project.

Several commercially available bonding agents were compared for use with the current study. There are two types of bonding agents, re-emulsifiable and non-re-emulsifiable that act differently in the presence of water. In this study, the researchers chose to only compare re-emulsifiable bonding agents because of their property to re-wet after initial application, a preferable method in the field. Bonding agents from four different companies were evaluated and compared based on the data provided by each company for their products. The comparisons of these products were problematic since there are multiple testing methods used to determine bond strength of a bonding agent. These testing methods included ASTM C1059, *Standard Specification for Latex Agents for Bonding Fresh to Hardened Concrete* (ASTM 2013a), and ASTM C1042, *Standard Test Method for Bond Strength of Latex Systems Used With Concrete By Slant Shear* (ASTM 1999). Bond strengths for each bonding agent were also compared. Table 2.1 lists the bonding agents that were compared and the strength data from the data sheets provided by the company. No strength information was given for MasterProtect P110. Products from Sika and US Mix Company were much lower than that of Euclid. This study chose the bonding agent Tammsweld produced by Euclid Chemical Company because it had the highest bond strength of 4,600 psi.

Table 2.1 Bonding Agent Comparison

Company	Product	Strength	ASTM
BASF	MasterProtect P110	n/a	n/a
Euclid Chemical Company	Tammsweld	4,600 psi	C1042
Sika	SikaLiquid Weld	1,300 psi	C1059
US Mix Company	Multi-55	14-day strength, Type I: 1,700 psi Type II: 1,300 psi	C1042

2.4.2 Polypropylene Fibers

There are a number of material properties to consider for concrete; however, one of the most important one for this study is fibers. A fiber is a strand of material that is mixed with the concrete to improve performance (Patel et al. 2012). Fibers are made of steel, glass, synthetic, and naturally occurring substances. Depending on the application of the concrete mix, a particular fiber might be more suitable. The most commonly used fibers are steel and synthetic. Steel fibers would be added to a mix to increase tensile capacity for use in structural applications (Graybeal 2014), whereas a synthetic fiber, such as polypropylene, may be added to control cracking (Ahmed et al. 2006; Banthia and Gupta 2006; Madhavi et al. 2014; Serdar et al. 2015). Fibers are typically used for secondary reinforcing (i.e. for crack control), as opposed to conventional methods such as wire mesh. This study is focused on the use of polypropylene fibers.

Polypropylene fibers have been a topic of interest in recent years due to their versatility. Most notable properties of polypropylene fibers are its low specific gravity (S.G. = 0.91), high tensile capacity (80 ksi to 101 ksi), and high acid and salt resistance. In addition, they are nonabsorbent, noncorrosive, and chemically inert, meaning not reacting with concrete admixtures (Banthia and Gupta 2006; Kakooei et al. 2012; Serdar et al. 2015). Fiber lengths typically range from 0.25-2.5 inches, and can be either monofilament, consisting of single strands, or fibrillated, a bundle of strands. Polypropylene fibers can be added into any concrete mix, are readily accessible, and inexpensive, making them a suitable option for many applications, including bridge construction. Fiber dosage needs to be considered when determining the appropriate mix.

Research has shown an optimum range of fiber dosage for concrete. Due to the variability of concrete mixes and types of fibers used in each study the general findings and conclusion will be presented. Tests conducted by Ahmed et al. (2006) examined fiber dosage rates of 1, 2, and 3 lb/yd^3 (fiber dosage is in pounds of fiber per cubic yard of concrete). Compressive strength and splitting tensile strength increased with increasing fiber content up to 2 lb/yd^3, and a decrease in strength at 3 lb/yd^3 (Ahmed et al. 2006). They also showed a reduction in shrinkage cracking of 83% and 85% with the addition of 2 lb/yd^3 and 3 lb/yd^3, respectively. Experimental work conducted by Kakooei et al. (2012) also observed an increase in compressive strength with a fiber volume of 1.5-2 kg/m^3 (2.5-3.4 lb/yd^3).

Besides the fiber dosage, the actual fiber type can factor into the performance. A study by Banthia and Gupta (2006) concluded that the geometry of the fibers is important for optimizing shrinkage reduction. Fiber dosage rates used were 0.1% (1.5 lb/yd^3), 0.2% (3.1 lb/yd^3), and 0.3% (4.6 lb/yd^3) by volume. They tested four different fiber products and had a control test for a baseline to determine shrinkage crack reduction. The researchers concluded that longer, finer-sized fibers were better at reducing crack widths, and fibrillated fibers were better at controlling shrinkage cracking than monofilament fibers (Banthia and Gupta 2006).

Manufacturers provide recommended dosages which range between 1-1.5 lb/yd^3 (Madhavi et al. 2014). At higher fiber dosages, workability is reduced but this can be remedied with the use of a plasticizer. Using polypropylene fibers benefits the concrete in its plastic state and after hardening. Fibers hold the mix together while the concrete is still fresh, or plastic, reducing the possibility of segregation and bleeding. Segregation is the tendency of the heavier coarse aggregates to move toward the bottom of the concrete. Bleeding is a form of segregation - water in the mix rises to the surface because it is lighter than the other constituents. Fibers also reduce the shrinkage cracking. In addition, fibers improve the properties of hardened concrete, including: reducing drying shrinkage, increasing resistance to abrasion and freeze-thaw, increasing impact resistance, and restraining cracking.

2.5 Summary

The literature review examined numerous studies and offered valuable insight into this current study. Based on the literature reviewed for this thesis, the following key points are found:

1. High performance fiber reinforced concrete has been used as an alternate material to UHPC in connecting certain bridge precast components by the Virginia Department of Transportation.
2. ASTM C-157 is an adequate method for determining the drying shrinkage of concrete specimens.
3. The optimal test method for determining bond strength is the flexural beam test, ASTM C-78, because it best simulates loading in the field.
4. Based on the performance and ease of application the optimum substrate surface preparation is the exposed aggregate finish.

5. Saturated surface dry moisture condition at the interface is suggested to provide the best performance.
6. Bonding agents are primarily used for applications involving repair or overlays.
7. Polypropylene fibers proved to be a suitable option based on performance, availability, and cost.

Based on this summary, the objective of this study becomes significant, especially in terms of finding a cost-effective alternative for field-cast connections of precast elements in accelerated bridge construction.

CHAPTER 3 METHODOLOGY

3.1 Introduction

The optimum HES concrete mix was chosen based on compressive strength, splitting tensile strength, and shrinkage. Interface bond strength tests were conducted using segments of precast and optimum HES concrete.

3.2 Mix Designs

This project consisted of designing a control mix and five alternate mixes. The control mix was determined by using the ACI absolute volume method. Cement for this mix was Type II. The cement was supplied by Ash Grove Cement Company. Fly ash Type F was used as a cement substitute (secondary cementitious material).

To obtain HES concrete, MasterSet AC 534 accelerating admixture was included. Air entrainer MasterAir AE 200 was used to control air content. High-range water reducer (HRWR) MasterGlenium 1466 was used. Other parameters to create the remaining mixes were shrinkage reducing admixture (SRA), bonding admixture (BA), and polypropylene fibers. Bonding admixture was AKKRO-7T and was supplied by Euclid Chemical Company. Shrinkage reducing admixture was MasterLife SRA 035. Polypropylene fibers were Fibermesh 150 as shown in Figure 3.1. Table 3.1 provides a list of the products used in the concrete mixes for this research project. Material specifications for constituents can be found in Appendix A.

Figure 3.1 Polypropylene Fibers (Fibermesh 150)

Table 3.1 Product List

	Product	**Company**
Accelerating admixture	MasterSet AC 534	BASF
Air entraining admixture	MaterAir AE 200	BASF
High-range water-reducing admixture	MasterGlenium 1466	BASF
Shrinkage reducing admixture	MasterLife 035	BASF
Bonding admixture	AKKRO-7T	Euclid
Bonding agent	Tammsweld	Euclid
Polypropylene Fiber	Fibermesh 150	Fibermesh

Table 3.2 shows the mix design variables which were developed in consultation with the ITD Technical Advisory Committee. Trial batches mixed in the laboratory were also helpful with determining admixture and fiber dosages. During trial batches, workability was significantly decreased with a fiber dosage of 3.0 lb/yd^3. The trial batches using a fiber dosage of 1.5 lb/yd^3 proved more workable. To avoid workability issues in later mixes, fiber dosages of 0.75 lb/yd^3 and 1.5 lb/yd^3 were used for this research. Accelerator dosage of 70 fl oz/cwt was determined from trial batches for the control mix to obtain the minimum 1-day compressive strength of

3,000 psi, as specified by ITD. This dosage was used for all mixes. Air entrainer was adjusted for each mix to meet the air content requirement. Mixes with bonding admixture (BA) did not require additional air entrainment. Recommended manufacturer dosages for SRA and BA were used. SRA dosage used was 1 gal/yd^3. For BA the recommended dosage was to mix one part AKKRO-7T with three parts water (1:3). All admixture dosages were verified by the local ready mix company's owner who has served as a consultant for ISU student projects, and provided many of the materials for this research. A control mix was developed following American Concrete Institute (ACI) absolute volume method. The mix design also followed the specifications required by ITD's *Standard Specifications for Highway Construction* (2012). Refer to Appendix B for the mix design procedure and the appropriate ITD design parameters. The precast concrete mix design used for this research was supplied by a local precast producer.

Table 3.2. Mix Design Variables

Mix	Fibers	SRA	BA
A (Control)	-	-	-
B	0.75 lb/yd^3	-	-
C	1.5 lb/yd^3	-	-
D	1.5 lb/yd^3	✓	-
E	0.75 lb/yd^3	✓	✓
F	-	✓	✓

Note: Fibers are polypropylene fibers; SRA = shrinkage reducing admixture; BA = bonding admixture

A great deal of time was spent conducting trial batches and tests to determine the control mix for this project. The control mix proportions are shown in Table 3.3. Water adjustments to the mix design were made to account for the moisture content and absorptions of the coarse and fine aggregates. Water adjustments for admixtures were also considered as per the recommendation of ITD Materials Engineer (Clint Hoop). Accelerator admixture is composed of 46.5% liquids, so the equivalent amount of water was taken out. Equivalent amounts of water were taken out with the addition of SRA or BA. The contributions of air entraining and water reducing admixtures to the adjusted water were minimal compared to the others and were ignored. The first water value shown in Table 3.3 is the required mix water adjusted for the moisture and absorption of the aggregates. The second value "Water used" is the amount of water adjusted for aggregates and admixtures. This value was the actual amount of mix water used during batching. Table 3.4 shows the summary of the admixture dosages used for each mix. More details about water adjustments and all the mix design proportions can be found in Appendix B.

Table 3.3 Control Mix Proportions

Water to cement ratio (w/c)	0.36	
Fine aggregate		
Moisture content	5.7	*%*
Absorption	2.1	*%*
Coarse aggregate		
Moisture content	1.9	*%*
Absorption	1.3	*%*
Water (*adjusted for moisture contents and absorption of fine and coarse aggregates*)	176	*lb/yd*^{3}
Water used (*adjusted for admixtures*)	156	*lb/yd*^{3}
Cement	52	*lb/yd*^{3}
Fly ash	132	*lb/yd*^{3}
Fine aggregate	1564	*lb/yd*^{3}
Coarse aggregate	1454	*lb/yd*^{3}
Admixture	Dosage	
Accelerator (AC)	70	*fl oz/cwt*
Air entrainer (AE)	7	*fl oz/cwt*
Superplasticizer (HRWR)	8	*fl oz/cwt*
Shrinkage reducing admixture (SRA)	0	*gal/yd*^{3}
Bonding admixture (BA)	0	
Fibers	0	*lb/yd*^{3}

Note: cwt = hundred weight of cementitious material

Table 3.4 Mix Design Admixture Dosage Summary

Admixture	Units	A	B	C	D	E	F
(AC)	fl oz/cwt	70	70	70	70	70	70
(AE)	fl oz/cwt	7	10	10	10	-	-
(HRWR)	fl oz/cwt	8	8	8	8	5	6
(SRA)	gal/yd^3	-	-	-	1	1	1
(BA)		-	-	-	-	1:3	1:3
Fibers	lb/yd^3	-	0.75	1.5	1.5	0.75	-

Note: cwt = hundred weight of cementitious material

3.3 Aggregate Analysis

The aggregate for this project was supplied by Pocatello Ready Mix. Sampling of aggregates was conducted following ASTM D75, *Standard Practice for Sampling Aggregates* (ASTM 2014a). Aggregate was collected from the stockpile as seen in Figure 3.2 and were kept sealed in five gallon buckets to preserve their moisture content during storage. The moisture content of aggregates was obtained following ASTM C566 (ASTM 2013d). Aggregate absorption was determined by following ASTM C127 (ASTM 2015b) and ASTM C128 (ASTM 2015c), for coarse and fine aggregates, respectively. Sieve analysis was also conducted according to ASTM C136, *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates* (ASTM 2014b). Moisture and absorption values were used to determine water adjustments for the mix design. Aggregate gradations met the requirements as specified by the Idaho Transportation Department (2012). Aggregate analysis data can be found in Appendix C.

Figure 3.2 Collecting Aggregate from Stockpile

3.4 Sample Casting

Concrete samples were cast in accordance with ASTM C192, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory* (ASTM 2016). Slump and air content tests were performed before casting samples. Slump and air content were determined in accordance with ASTM C143 and ASTM C231, respectively. Capping of concrete cylinders followed ASTM C617, *Standard Practice for Capping Cylindrical Concrete Specimens* (ASTM 2015a). Concrete batching was mainly done in the laboratory with a barrel mixer as shown in Figure 3.3. However, a mixing truck was used when a large quantity of precast concrete was needed for headed bar pullout and flexural beam specimens. Appropriate laboratory procedures for mixing concrete were followed.

The following procedure was used for mixing concrete:

1. Add the air entraining admixture to the mixing water. (This aids in the dispersion of the admixture.)
2. Mix approximately one half of the coarse aggregate, fine aggregate, and water. Mix for approximately three minutes to ensure the aggregates are well graded.
3. Add about one half of the cement and fly ash while the mixer is running.

4. Once the constituents are thoroughly mixed, add the remaining coarse aggregate, fine aggregate, cement, and fly ash. Also, add the remaining water and additional admixtures. Mix for approximately three minutes.
5. Add the fibers and continue to mix for 3-5 minutes. Depending on the batch size more mixing time may be needed.

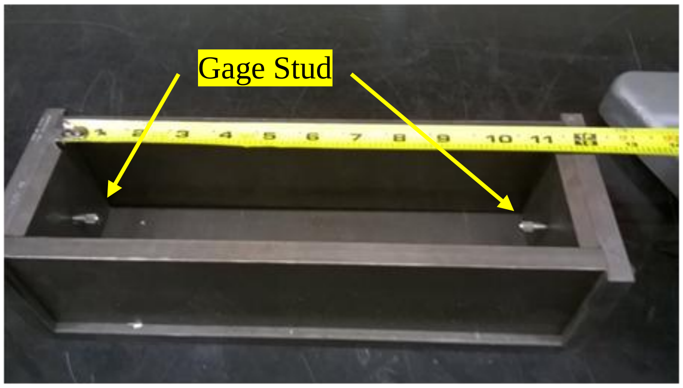
3.4.1 Cylinder and Shrinkage Prism Casting

Cylinders were cast for determining compressive and splitting tensile strengths. Samples were 4-inch diameter by 8-inch tall and cast using standard plastic molds. Length change prisms were cast in molds with dimensions of 3 in. x 3 in. x 11.25 in. and included anchor holes in the ends to secure gage studs. Gage studs were screwed into the ends of the mold to be cast into the specimens. Concrete molds are shown in Figure 3.4.

After 24 hours the samples were removed from their molds and placed in a water bath to cure. A water tank was constructed for curing due to the large quantity of samples that needed to be tested (Figure 3.5). Concrete cylinders and shrinkage prisms were moist cured in lime-saturated water for 28 days, after which the samples were removed from the water and prepared for testing.

Figure 3.3 Barrel Mixer

(a) Cylinder mold



(b) Length change prism mold

Figure 3.4 Concrete Molds

Figure 3.5 Water Tank for Moist Curing of Samples

3.4.2 Interface Beam Casting

To determine the interface bond strength between precast concrete and HES concrete, a modified ASTM C78 developed by De la Varga, Haber, and Graybeal (2016) was utilized. The modified standard uses a 6 in. x 6 in. x 21 in. composite beam (Figure 3.6) loaded in third-point bending. Specimens are made by first casting the precast segments. The precast interface was prepared next, followed by the pouring of the connection concrete. Exposed aggregate surface finish was the chosen method for precast concrete interface surface preparation. Exposing the aggregate was accomplished through the use of a concrete surface retarder, Formula F manufactured by Euclid Chemical Company. This product delays the setting of concrete so that the surface can be washed or scrubbed away to reveal the underlying coarse aggregate. Based on the literature and the recommendations from ITD, the precast interface surface moisture condition utilized was saturated surface dry (SSD).

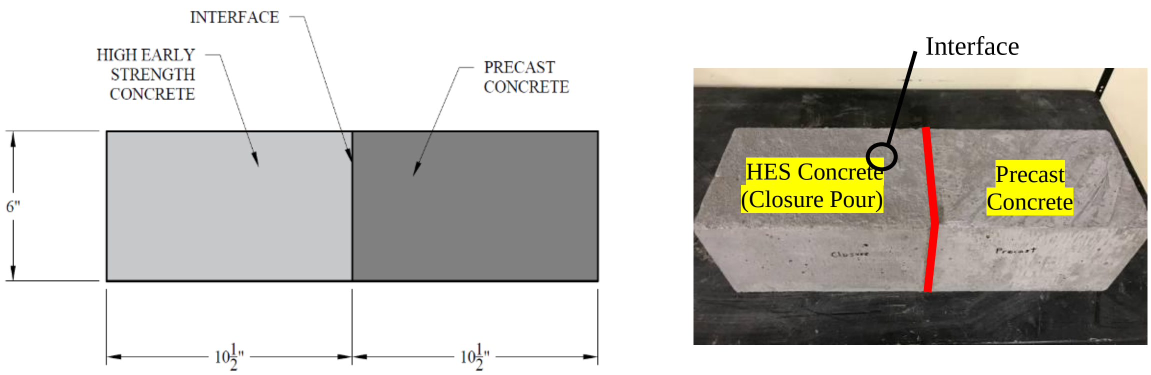


Figure 3.6 Interface Bond Specimen

The wood mold seen in Figure 3.7 shows a divider in the middle that was painted with a layer of concrete surface retarder. The molds were lubricated with WD-40 to prevent concrete from sticking to the wood and promote easy mold removal, except where the concrete surface retarder was applied. The molds were filled in two lifts of equal volume and rodded 32 times for each lift. After each lift the concrete was consolidated by tapping the sides of the mold 10 to 15 times with a mallet. The surface was troweled to achieve a smooth finish once the second lift is consolidated. The samples were covered with plastic to prevent water from evaporating and were allowed to cure for 24 hours. After curing, the precast segments were removed from the molds and the surface, in contact with the concrete retarder, was washed away with water using a garden hose sprayer to produce an exposed aggregate (EA) finish as shown in Figure 3.8.

(a) Interface beam mold (b) Concrete retarder on form

(c) Precast section poured

Figure 3.7 Interface Bond Mold

Wet burlap was placed around the precast concrete segments and then covered in plastic. The burlap was monitored and kept moist with a sprayer throughout the curing process. The precast sections were removed at 28 days. After removal, these sections were placed back into the molds. The molds were lubricated with WD-40 oil spray. The EA surface on the precast portion was sprayed with water to create a saturated surface dry (SSD) moisture condition. Casting the HES concrete followed the same steps as the precast concrete and a similar curing method was used after specimens were removed from the molds.

A second set of beam specimens were used to determine the effect of applying bonding agent at the interface. Casting and curing procedures were the same as the previous set except for the precast interface surface preparation. The precast sections were removed from curing one day before casting the HES closure concrete. The EA surfaces were dried with a heat gun then the bonding agent (BG), Tammsweld, was applied with a paint brush and allowed to dry before being placed back in the molds. For casting the HES concrete, water was not sprayed over the bonding agent. Refer to Appendix M for pictures of the casting process.

Figure 3.8 Exposed Aggregate Surface Preparation

3.4.3 Headed Bar Pullout Specimen Casting

Headed bar pullout tests were used to simulate the lower portion of a closure pour deck connection between Deck Bulb-T girders as shown in Figure 3.9. The specimen consists of precast concrete sections and the closure concrete reinforced with three headed rebars. Lenton Terminators supplied by Pentair were chosen for use in this research to provide the headed rebar.

Figure 3.10 shows a typical headed rebar which consists of a tapered thread and an oversized coupling that is screwed to the end. Rebars were supplied by Harris Rebar. The sequence of beam construction is shown in Figure 3.11.

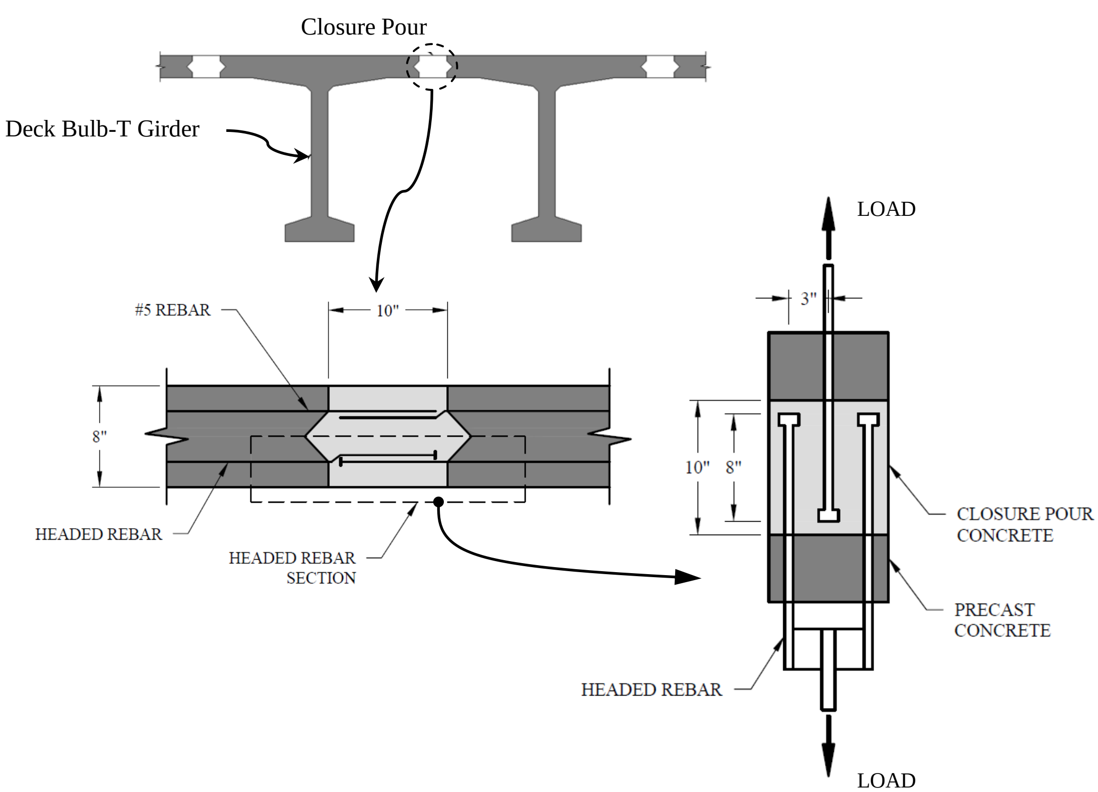
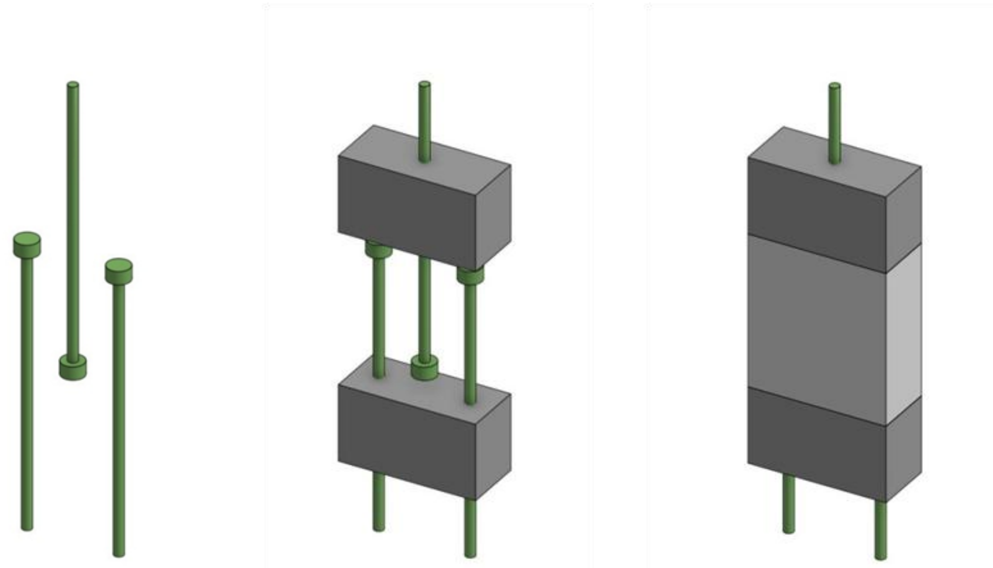


Figure 3.9 Headed Bar Pullout Specimen Schematic

(a) Tapered threaded rebar and Lenton Terminator

(b) Screwed together

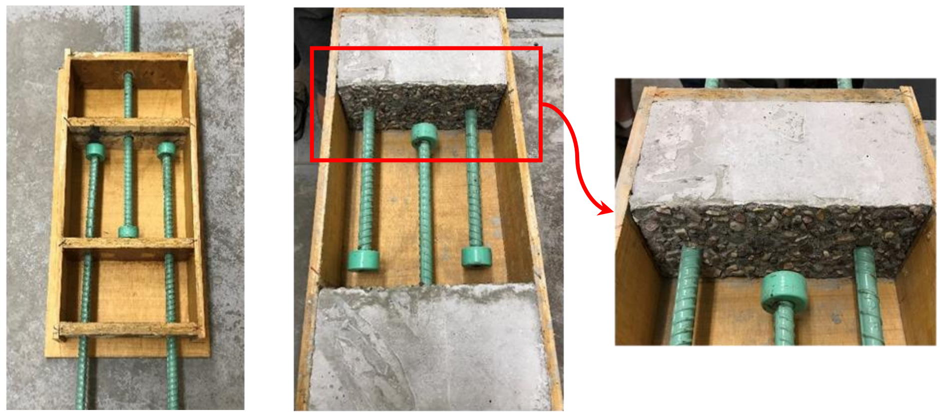
Figure 3.10 Headed Rebar



(a) Rebars (b) Precast concrete segments cast (c) Closure pour concrete cast

Figure 3.11 Headed Bar Pullout Specimen Fabrication Sequence

Precast segments of the headed bar pullout were prepared similar to the interface bond sample with the exposed aggregate surface finish. Molds were constructed from plywood. Holes were drilled for placing the rebar. Concrete surface retarder was applied to the inside face of the two precast segments. Strain gages were installed on rebar before casting concrete. Rebars were set in the molds so that the strain gages were oriented facing up and down. Precast sections were poured in two lifts and rodded 25 times for each lift. After the precast concrete cured, the closure concrete was poured. Steel plates and threaded rods were welded to the ends of the sample. These were used to attach the specimen to the United tensile testing machine. Prior to testing, the front faces of the specimens were painted white and 1 in. x 1 in. grid lines were marked with a pencil. The paint helped identify cracks. Refer to Appendix M for details regarding the casting and fabrication of the headed bar pullout specimens. Figure 3.12 shows the progression of specimen fabrication; beginning with the rebar placement, then casting the precast concrete, and finally casting the closure concrete. Actual fabrication can be seen in Figure 3.12. The photos show the rebar being placed in the wood mold, then the exposed aggregate surface finish of the precast interface. The last photo is a specimen after casting the closure concrete and attaching the end fixtures.



(a) Mold (b) Precast segments cast (c) Exposed aggregate finish

(d) Fully cast specimen

Figure 3.12 Headed Bar Pullout Specimen Fabrication

3.4.4 Flexural Beam Casting

Large beam samples were used to determine the flexural strength under three-point and four-point bending. Beams were constructed by connecting two separate precast segments with the closure concrete. The overall dimensions of the beam were 78 in. x 12 in. x 8 in. and contained reinforcement as seen in Figure 3.13. Casting procedure for large beam specimens was similar to headed bar pullout specimens. Strain gages were installed on rebar before casting concrete and were oriented so strain gages were facing up and down. The sequence of casting is shown in Figure 3.14. Rebars were first set up with epoxy coated rebars at the top and headed rebars at the bottom. Next, the precast concrete was poured. The last step was to connect the two precast segments by pouring the closure concrete.

Due to the large volume of precast concrete needed, a mixing truck was used for the larger precast segments. Precast concrete for headed bar pullout and flexural beam specimens were cast at the same time. The concrete surface retarder was applied to the molds. Rebars were placed inside the molds and set the correct distance. A group of ISU student volunteers helped pour all the specimens within an hour. Figure 3.15 shows pictures from the "Pour Day" which the volunteers were involved with. After casting the concrete, the rebars were checked to ensure they were correctly positioned. Eye bolts were embedded in samples to be used for lifting. Precast concrete cylinders to be used for material property tests were cast at the same time. Refer to Appendix M for more photos of specimen fabrication.

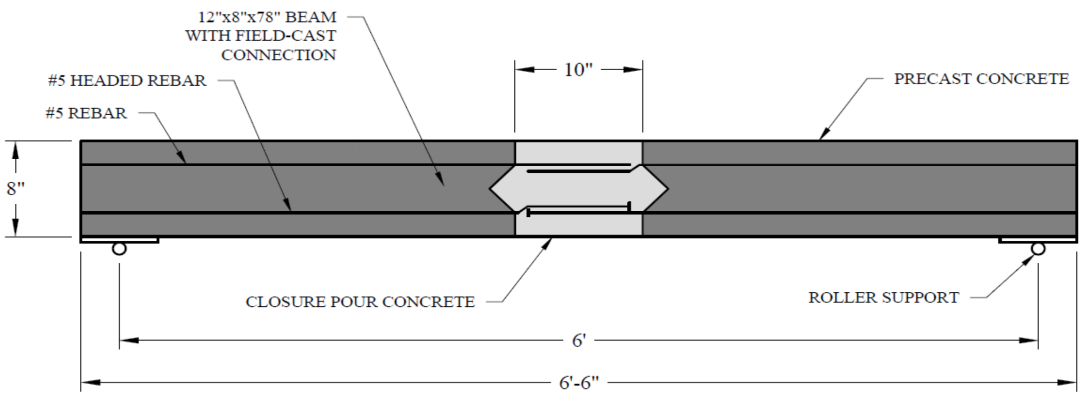
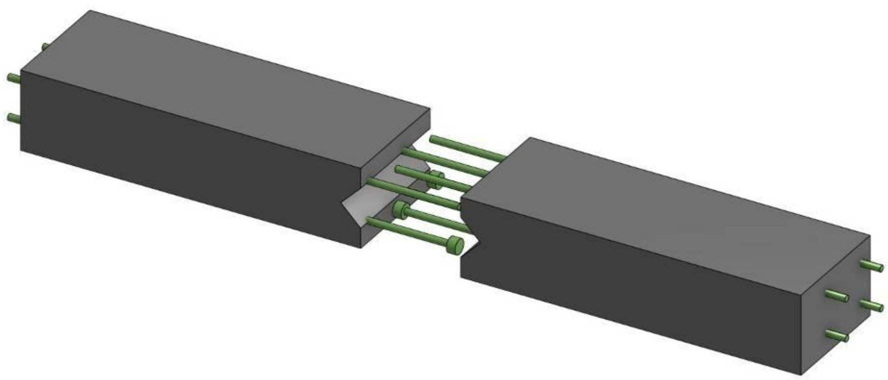
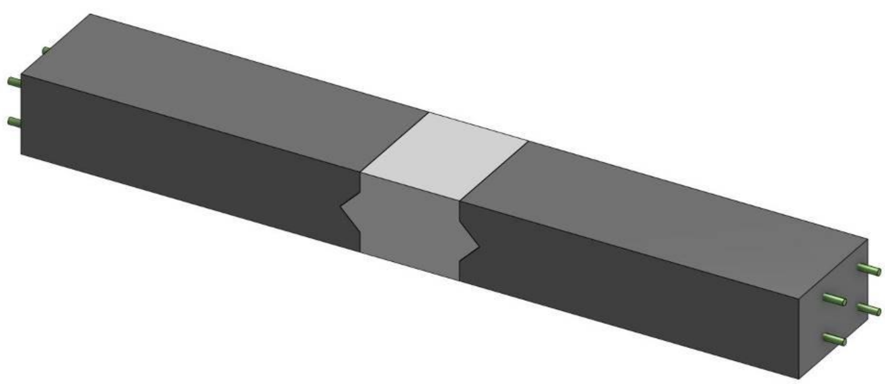


Figure 3.13 Large Beam Schematic

(a) Rebars



(b) Precast concrete segments cast



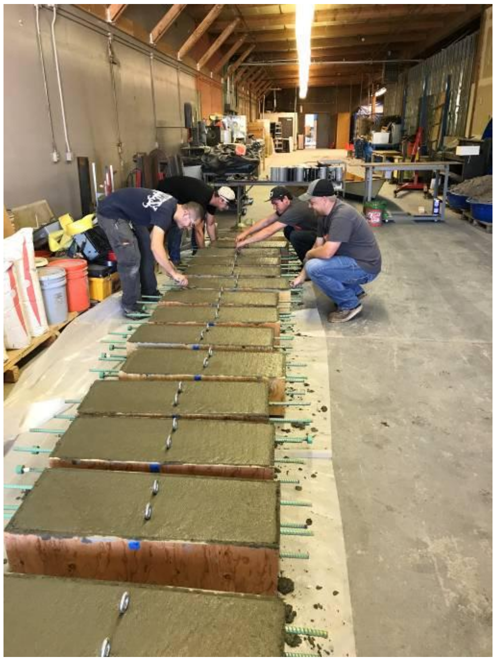
(c) Closure concrete cast

Figure 3.14 Large Beam Specimen Fabrication Sequence

(a) Pouring large beams

(b) Pouring pullout beams

(c) Rodding large beam



(d) Embedding eye bolt anchors

Figure 3.15 Pictures from "Pour Day" (Casting of Precast Concrete Segments)

3.5 Testing

This section presents the various tests that were carried out as part of the experimental work for this project. These tests include concrete compressive strength, splitting tensile, length change, modulus of elasticity, and Poisson's ratio which were used to determine concrete material properties. Additional tests include interface bond, headed bar pullout, and flexural beam. These tests are described in the following sections.

3.5.1 Compression Test

Concrete compressive strength tests were conducted according to ASTM C39, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens* (ASTM 2018). Specimens were capped according to ASTM C617 (ASTM 2015a) with a sulfur based compound to provide uniform load distribution. Cylinders were used to determine the 1-day and the 28-day compressive strengths. Compression tests were conducted using the Gilson Compression Testing Machine shown in Figure 3.16. A loading rate of approximately 440 lb/sec was applied until failure. The peak load was recorded from the digital readout of the machine. Figure 3.17 shows a compressive specimen prior to testing.

Figure 3.16 Gilson Compression Machine

Figure 3.17 Compression Testing Set-Up

3.5.2 Splitting Tensile Test

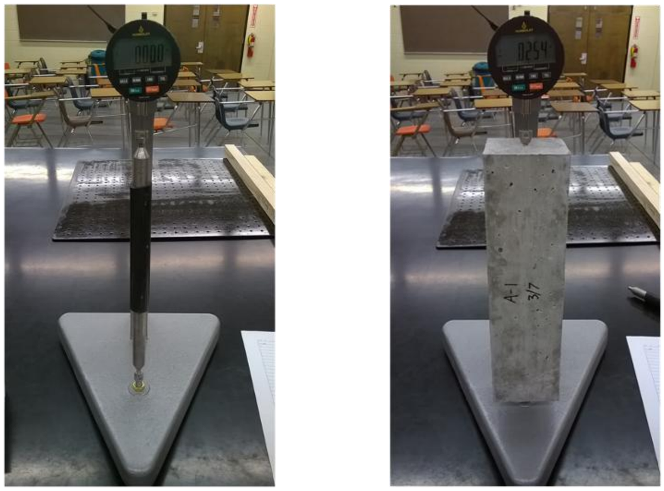
Splitting tensile strength tests followed ASTM C496, *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens* (ASTM 2017d). Either 4 in. x 8 in. or 6 in. x 12 in. concrete cylinders are used for this test. Wood strips were used to provide uniform loading along the length of the cylinder. A loading rate of approximately 126 lb/sec was applied until failure. A typical splitting tensile test is shown in Figure 3.18.

Figure 3.18 Splitting Tensile Testing Set-Up

3.5.3 Length Change Test

Length change testing followed ASTM C157, *Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete* (ASTM 2017c). This test method uses 3 in. x 3 in. x 11.25 in. concrete prisms with gage studs embedded at the ends to measure changes in length over time. An initial measurement was taken after removing samples from the molds 24 hours after casting. Samples were then moist cured. At 28 days, samples were removed and another measurement was taken. Samples were then placed on a rack at least one inch apart to allow for uniform drying as shown in Figure 3.19. Specimens continued to dry during which time measurements were periodically recorded. Measurements were taken after curing at 4, 7, 14, 28, 42, and 56 days, and then every 4 weeks for a total of 336 days following curing. A comparator, shown in Figure 3.20, was used to measure the difference in length between the reference bar and the specimen. A reference bar is first placed in the comparator and the digital indicator is zeroed. A specimen is then placed into the comparator and the measurement is recorded. The change in length is then calculated according to the ASTM.

Figure 3.19 Shrinkage Specimens Air Drying



(a) Length Comparator (b) Measuring Sample

Figure 3.20 Length Change Test

3.5.4 Modulus of Elasticity and Poisson's Ratio Tests

Modulus of elasticity and Poisson's ratio tests were conducted on cylinders that were made while casting the precast and closure concretes for the headed bar pullout and large beam specimens. Prior to conducting compressive strength tests, concrete cylinders were first used to determine modulus of elasticity and Poisson's ratio according to ASTM C469 (ASTM 2014c). This test uses a compressometer/extensometer (Figure 3.21) to measure strain of a concrete specimen subjected to compression loading as seen in Figure 3.22. Refer to Appendix K for the detailed testing procedure.

Figure 3.21 Compressometer/Extensometer

Figure 3.22 Compressometer/Extensometer Test Set-Up

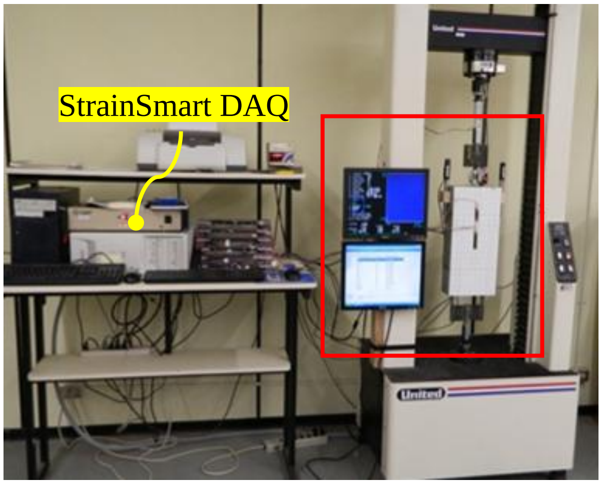
3.5.5 Interface Bond Test

Similar bond tests performed by De la Varga et al. (2016) and Haber and Graybeal (2016) were used for this study because this test best replicates the stresses at the interface in the field. Interface bond samples were tested in third-point loading in accordance with ASTM C78. Figure 3.23 shows the Gilson Testing Machine and flexural beam testing apparatus that were used to break the samples. A constant loading rate of 30 lb/sec was applied until beam failure occurred. After testing concluded, the maximum load was recorded and cross-sectional measurements were taken to calculate bond strength.

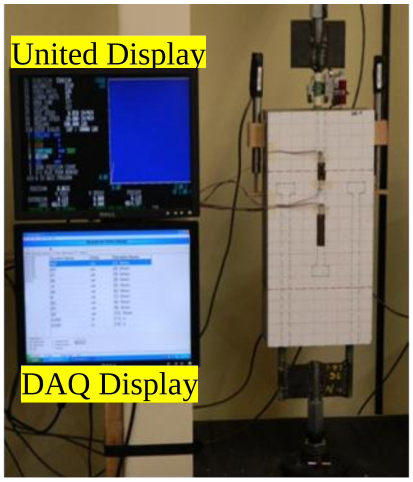
Figure 3.23 Interface Bond Test Set-Up

3.5.6 Headed Bar Pullout Test

Samples were tested in tension using the United Testing Machine shown in Figure 3.24. Monitors were set up next to the test specimen so that a camera could view the specimen and the measured values. Specimens were loaded at a constant rate of 0.01 in./min until failure occurred. Load, displacement, and strain values were recorded with each test. Six headed bar pullout tests were conducted. Instrumentation of the specimens will be discussed in Chapter 4.



(a) Set-up



(b) Sample

Figure 3.24 Headed Bar Pullout Test Set-Up

3.5.7 Flexural Beam Test

A total of six beams were cast. Three beams were tested in three-point bending and three were tested in four-point bending. Beams were tested in the Tinius Olsen Testing Machine shown in Figure 3.26. Testing was conducted such that the closure pour concrete age was 28 days. The span length for all beams was 72 inches.

Figure 3.26 shows the diagram of the flexural beam test set-up. Before loading the connecting bolts for the steel beam to the steel column supports were removed. The Tinius Olsen contains a hydraulic pump underneath the loading platform so when the pump is activated the platform is lifted up into the upper plate. This means the steel beam is unsupported at the ends and is only support by the loading platform.

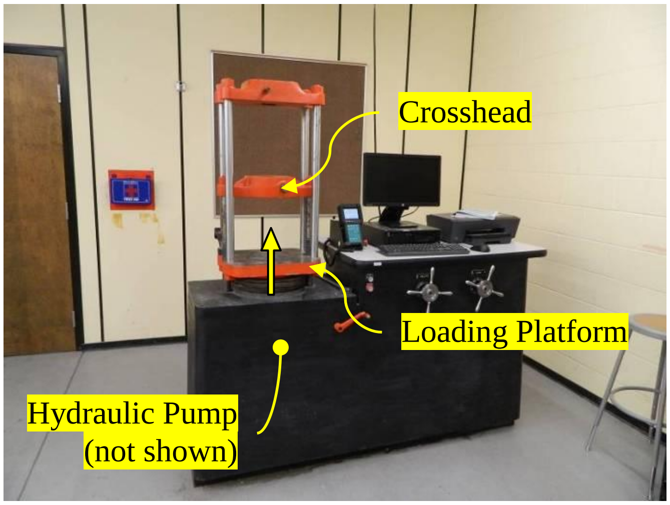


Figure 3.25 Tinius Olsen Testing Machine

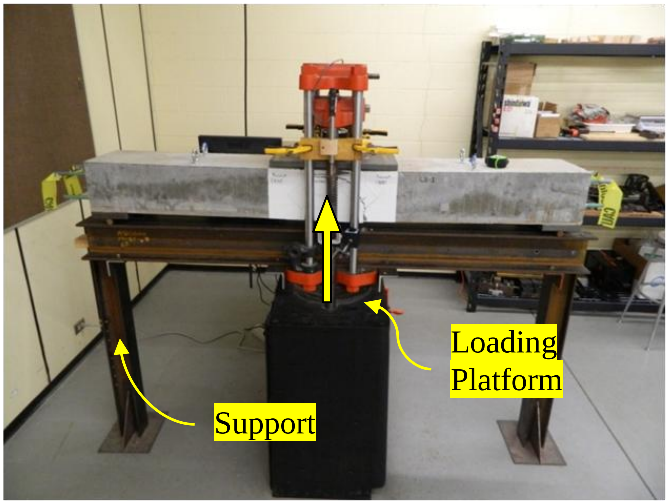


Figure 3.26 Beam Test Set-Up

Three-point flexural testing consisted of applying the load to a one-inch thick plate in the center of the beam. The plate was 10 in. x 20 in., which simulates a truck tire footprint as prescribed by American Association of State Highway and Transportation Officials (AASHTO).

Figure 3.27 shows the set-up for three-point load testing. The steel plate, load cell, and rubber pad were set up as shown in Figure 3.28.

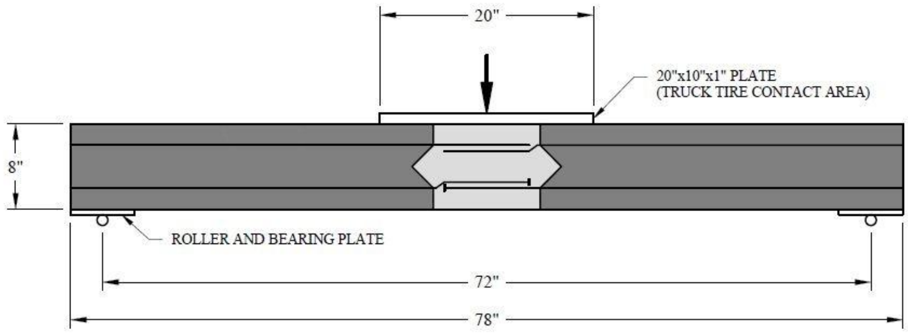


Figure 3.27 Three-Point Flexural Test Diagram

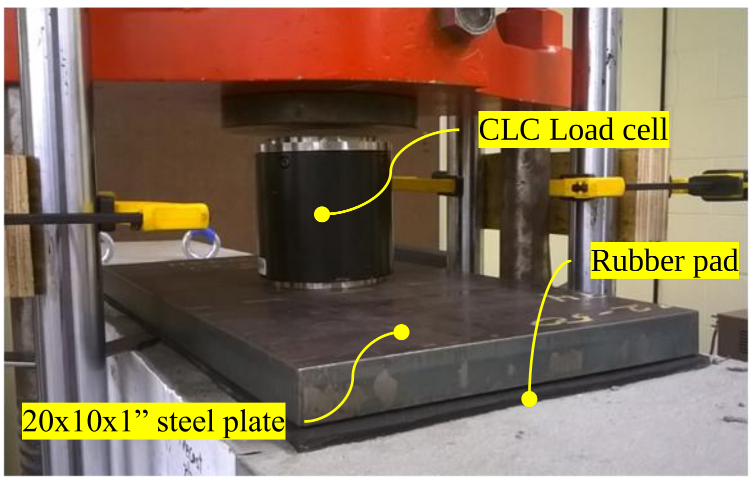


Figure 3.28 Three-Point Loading Set-Up

Four-point loading in Figure 3.29 shows a spreader beam distributing the load through a spreader beam (HSS 8 x 4 x 3/8). The load cell set-up for four-point bending is seen in Figure 3.30. Loading for all beam tests were applied at a rate of 0.1 in./min until failure. Instrumentation of the beam specimens will be discussed in Chapter 4. More pictures of tests setups can be found in Appendix M.

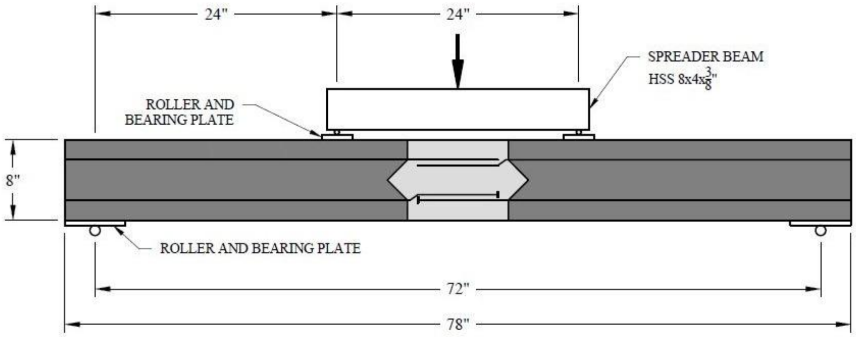


Figure 3.29 Four-Point Flexural Test Diagram

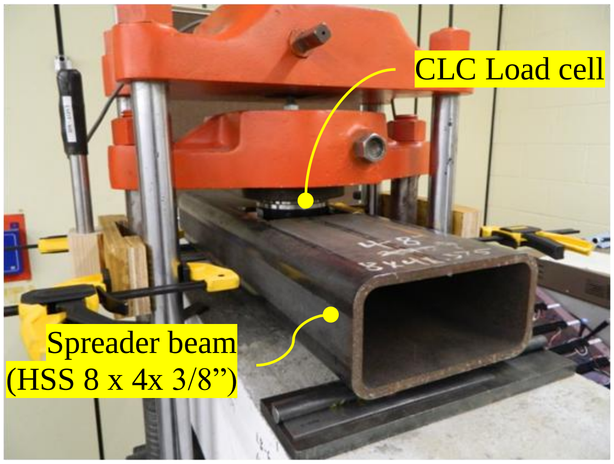


Figure 3.30 Four-Point Loading Set-Up

CHAPTER 4 DESCRIPTION OF INSTRUMENTATION

4.1 Introduction

The following section describes the instrumentation used to measure specimens during testing. Instrumentation includes sections on strain gages, linear variable differential transformers (LVDT), and load cells. In addition, instrumentation for headed bar pullout tests and flexural beam tests is discussed. A description of the Vishay StrainSmart System 6000 Data Acquisition System, which was used to collect the test data, will also be given. The data acquisition system (DAQ) uses cards for different measuring devices. This project utilized strain gage and LVDT sensors.

4.2 Strain Gages

A strain gage is a sensor that changes in resistance due to deformation (e.g. compression or extension) of the gage when an external force is applied. The change in resistance can then be converted to stress, force, pressure, or some other unit of measure. Foil strain gages consist of a grid made of a metallic resistive foil as shown in Figure 4.1. Sizes are determined by its gage length which is defined as the length of the foil grid. Three sizes of strain gages were used for this project: one quarter inch, one half inch, and two inch (Figure 4.2).

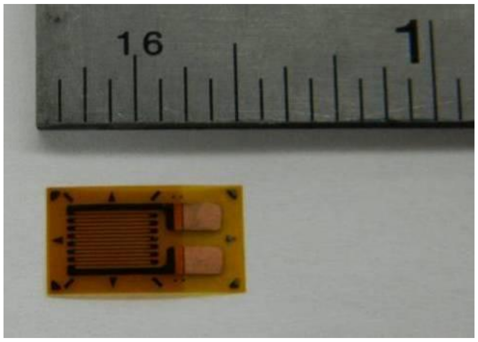
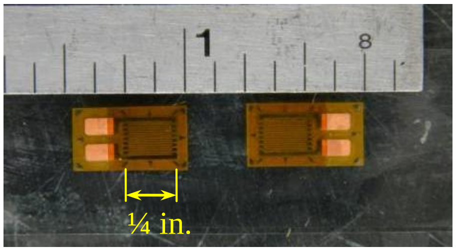
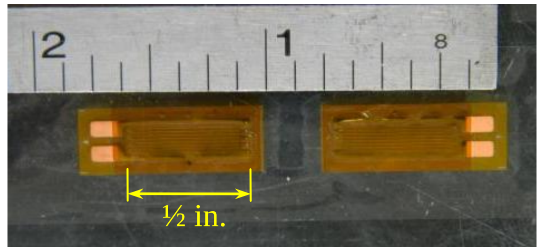


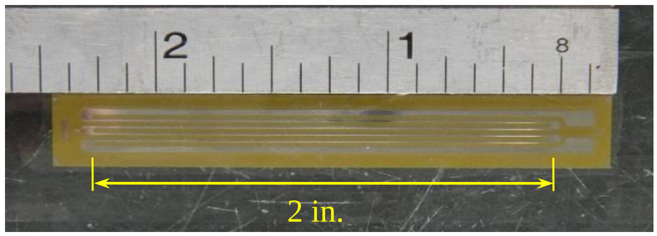
Figure 4.1 Strain Gage Foil Grid Detail



(a) ¼ in. strain gage



(b) ½ in. strain gage



(c) 2 in. strain gage

Figure 4.2 Strain Gages

In general, smaller gages are used to determine localized strain. One quarter inch strain gages were used for rebars to determine the stress near the head of the rebar. The smaller size also minimized the unbounded length of rebar in the concrete. The 0.25 in. gages were used for all the rebars. For concrete, longer strain gages are recommended by Micro-Measurements. Unlike steel, concrete is a composite material made up of different ingredients (i.e. cement, fine aggregate, coarse aggregate, and water). Coarse aggregate can cause excessive localized stress at the surface of the concrete. Using larger strain gages can mitigate this strain localization by using gage lengths that are longer than the diameter of the largest aggregate. This provides better readings compared to using shorter gages. Other factors must also be considered when determining gage size such as: space limitation, installation restrictions, and purpose of measurement. More details on concrete strain gages used for each test will be presented in Section 4.5. Strain gage specifications can be found in Appendix L.

The installation for the strain gages applied to rebar followed manufacturer procedures for steel installation as outlined in Micro-Measurements *Instruction Bulletin B-137,* strain gage installation for metallic structures. The first step was to grind down the deformations in the rebar and remove the epoxy coating. Both sides of the rebar near the heads were grinded. Surfaces were then sanded and cleaned with an acid solution. Marks were drawn on the rebar to align the strain gage. Strain gages were then attached to the rebar using M-Bond AE-10, a two part epoxy glue supplied by Micro-Measurements. After curing for 24 hours, strain gages were attached to the opposite side using the same procedure. Masking tape and electrical tape were used to protect the gages during handling and casting. After specimens were cast, the strain gages were inspected. Gages that were not fully bonded were removed and replaced. Replacement strain gages were attached with M-Bond 200, a cyanoacrylate glue, also supplied by Micro-Measurements. Although both are suitable for use on metal, M-Bond 200 was used for replacing gages instead of M-Bond AE-10 due to its faster setting time. After wires were attached to the gages, a protective coating (M-Coat F) was applied.

The installation for concrete strain gages followed procedure as outlined in Micro-Measurements *Tech Tip TT-611,* strain gage installation for concrete structures. Concrete strain gages were also used for this project. The surface was prepared by grinding the gage area. A base coat of epoxy was applied to fill any voids in the concrete. After the epoxy cured, the gage area was sanded down to the base material. The strain gages were then installed and wires attached. Concrete gages were applied with several coats of polyurethane (M-Coat A). Refer to Appendix L for more details about strain gage installation for rebar and concrete.

Figure 4.3 Rebar Strain Gage

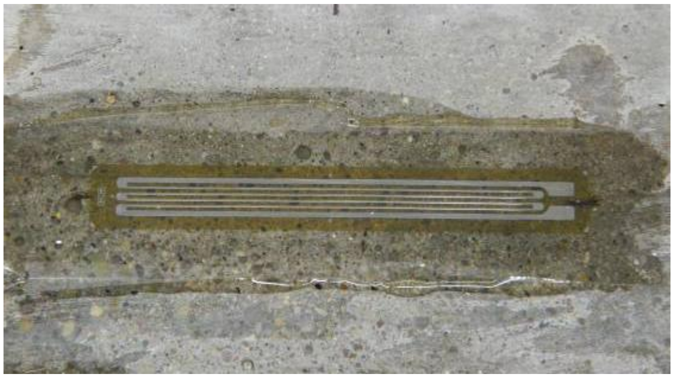


Figure 4.4 Concrete Strain Gage

4.3 Linear Variable Differential Transformer – LVDT

Displacements were measured using a linear variable differential transformer or LVDT, seen in Figure 4.5. Mounting blocks for the LVDTs were fabricated using 2 in. x 3 in. lumber to securely hold them in place. A hole was drilled through the block and a slit was cut along the side as Figure 4.6 shows. A screw was used to secure the LVDT to the block. No slipping occurred once the LVDT was clamped.

(a) LVDT and core with extension rod

(b) LVDT assembly

Figure 4.5 LVDT

(a) Mounting block

(b) Mount assembly

Figure 4.6 LVDT mounting block

4.4 Load Cells

Load cells were used to measure the applied force subjected to the specimens. The load cells for this project were strain gages based. Two load cells were needed; one for tension and another for compression. A tensile load cell was fabricated to be incorporated into the pullout tests. The compression load cell was used for the large beam tests.

4.4.1 Tensile Load Cell

A tensile load cell was fabricated for use with the headed bar pullout tests. This was done because the load cell of the United tensile testing machine could not be integrated with the Vishay Strain Smart Data Acquisition System. The load cell was fabricated using a one-inch threaded rod shown in Figure 4.7. The threaded rod was chosen so that the load cell could easily be integrated with the United Testing Machine's fixtures. Two 0.5 inch strain gages were attached on opposite sides of the threaded rod using the appropriate adhesion process. Two strain gages were used to cancel any bending effect. Threaded couplers were used to attach the load cell to the machine, as seen in Figure 4.8.

Calibrating the tensile load cell was done by applying load via the United tensile testing machine. Strain data was collected while load was applied in step fashion from zero to approximately 15,000 pounds. A camera was used to record the machine load and the corresponding strain values.

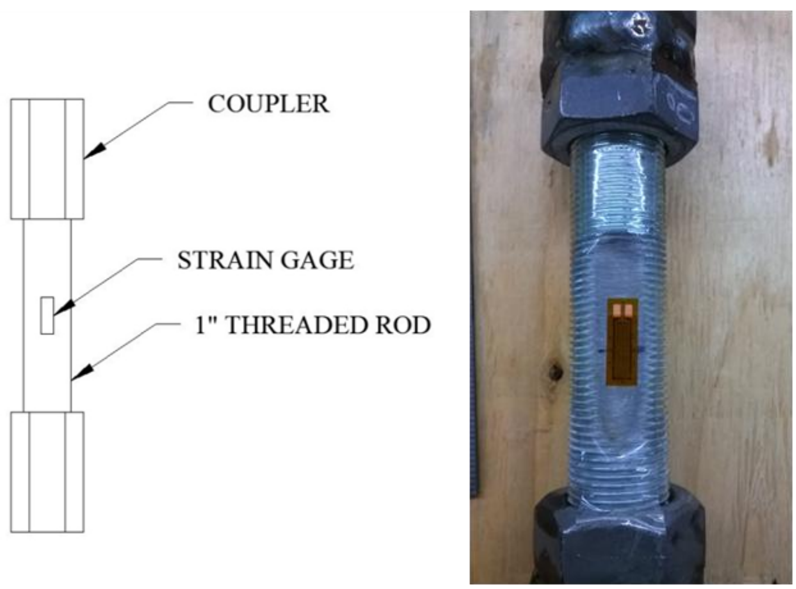


Figure 4.7 Tensile Load Cell Schematic

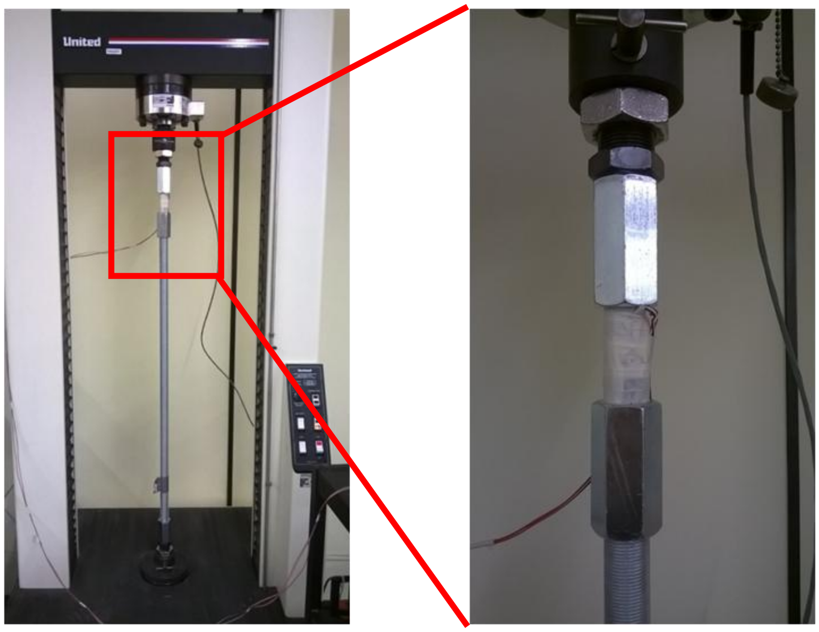


Figure 4.8 Tensile Load Cell Calibration Set-Up

The data shown in Table 4.1 was then imported into Excel and a plot of machine load versus load cell strain was created. A linear trendline was added to the plot with the intercept set at zero, as seen in Figure 4.9. The calibration factor is the constant value of 8.8186 lb/microstrain, which is obtained from the linear regression equation. The strain sum from the two gages on the load cell can be multiplied by the calibration factor to determine the applied force.

Table 4.1 Applied Force vs. Measured Strain Values

Machine Load (lb)	Stain 1	Strain 2	Strain Sum
0	0	0	0
5,008	208	359	567
8,019	380	527	907
10,050	494	645	1,139
13,061	659	822	1,481
14,956	762	936	1,698

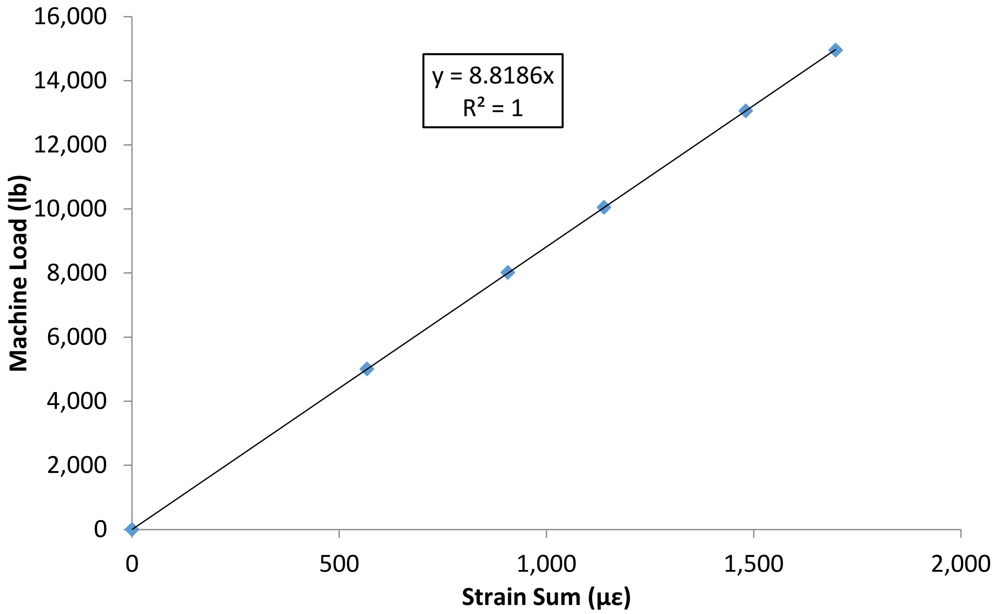


Figure 4.9 Load vs. Load Cell Strain

4.4.2 Compression Load Cell

A column load cell was used to measure the applied compressive force for the flexural beam tests. The load cell is a CLC-300K, shown in Figure 4.10. This load cell has a capacity of 300 kips. The calibration of this load cell was obtained from a calibration sheet supplied by Transducer Techniques. A test was conducted by placing the CLC load cell in the Gilson compression testing machine and connecting it to the DAQ as a strain gage based transducer. The calibration was verified by applying force and recording the loads from the Gilson machine and CLC load cell. Table 4.2 shows the calibration data and an average difference of 47 pounds between applied load (Gilson) and measured load (CLC-300K). Knowing the accuracy of the load cell was 35 pounds (i.e. increments of 35 pounds), the calibration was determined to be adequate for testing.

Figure 4.10 CLC-300K Load Cell

Table 4.2 CLC-300K Calibration Verification

Gilson (lb)	**CLC-300K (lb)**	**Difference**
1,620	1,595	25
2,000	1,977	23
2,860	2,879	-19
3,920	3,954	-34
5,120	5,064	56
6,130	6,070	60
7,040	6,971	69
8,000	7,943	57
9,020	8,983	37
10,070	10,024	46
11,060	10,995	65
12,110	12,035	75
13,050	12,937	113
13,970	13,908	62
15,050	14,983	67
		Average = 47

4.5 Specimen Instrumentation

The following section is a description of the instrumentation that was used for the headed bar (HB) pullout and large beam (LB) tests. Strain gage and LVDT details are explained for each test. Also presented are data analysis for the headed rebars and the large beam deflections.

4.5.1 Headed Bar Pullout

Data collected from pullout tests included strain, displacement, and force. Strain gages were included on the center rebar. Strain gages were placed at three locations: in the middle of the closure concrete and on either side of the top interface as shown in Figure 4.11. Smaller gages (i.e. 0.25 in. or 0.5 in.) were used at the interface between the closure and precast concretes in order to measure the strain at the same location on both sides of the interface. HB-1, 2, and 3 used 0.5 in. gages at the interface and HB-4, 5 and 6 used 0.25 in. gages. This was done to determine the differences, if any, between the two gage lengths. The 2 inch gages were used in the center of the closure concrete because there was space and longer gages are recommended by the manufacturer. Gages were installed on the front and back of the samples.. Interface strain gages were spaced one quarter inch apart and placed equidistant from the interface as shown in Figure 4.12. The interface gages were spaced similarly for both 0.25 and 0.5 in. gage lengths.

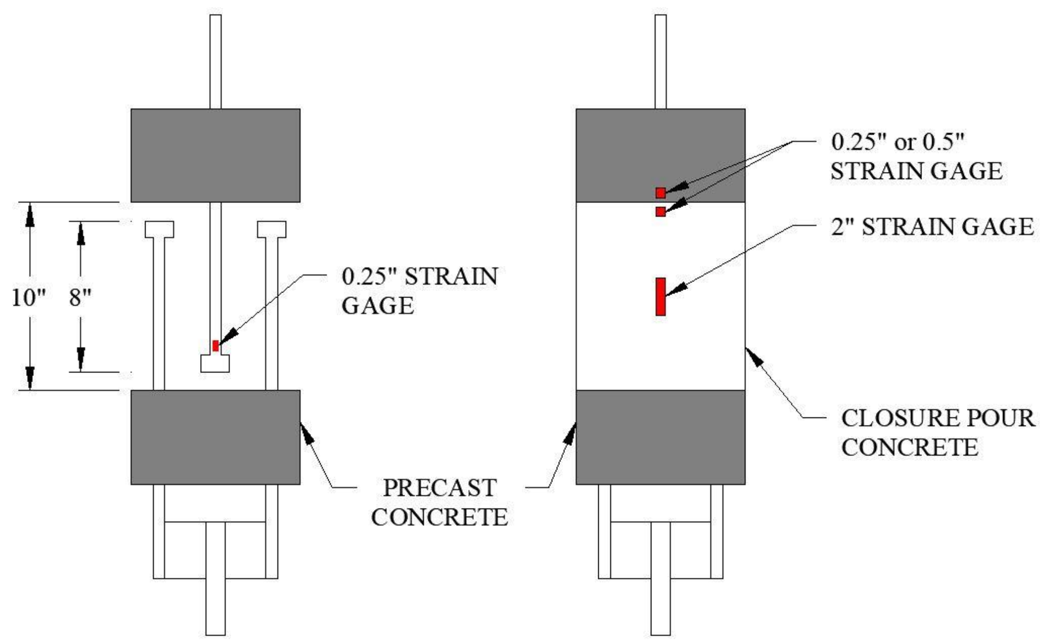


Figure 4.11 Instrumentation for Headed Bar Pullout Specimens

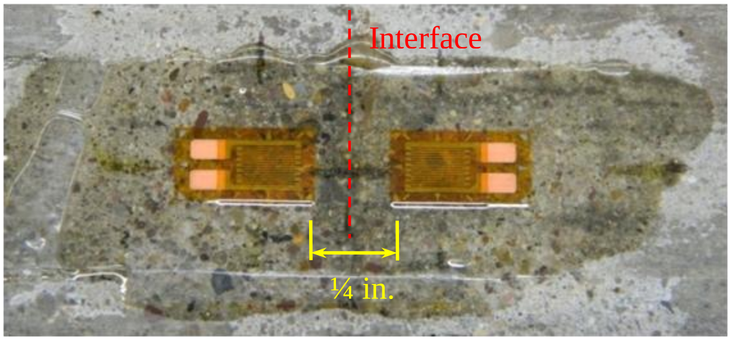
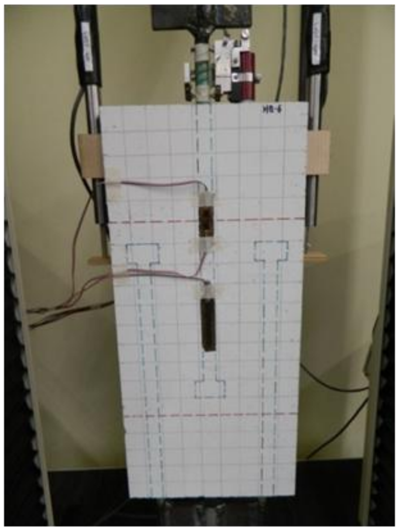


Figure 4.12 HB Interface Strain Gage Placement

LVDTs were attached to the left and right side to measure the expansion of the interface crack. Mounting blocks were attached to the precast concrete using a fast setting two-part epoxy. Supports made from L-brackets were also attached to the closure concrete for the LVDT extension rods as seen in Figure 4.13 and Figure 4.14. Threaded couplers were used to attach the ends of the pullout specimen and tensile load cell and from the load cell to the United Testing Machine, as shown in Figure 4.15. The crosshead was raised to insert the sample and then lowered to connect the couplings and fixtures.

(a) Headed rebar strain gages



(b) Concrete gages installed

Figure 4.13 Instrumentation of Headed Bar Pullout Specimen

Figure 4.14 LVDT Set-Up for Headed Bar Pullout Tests

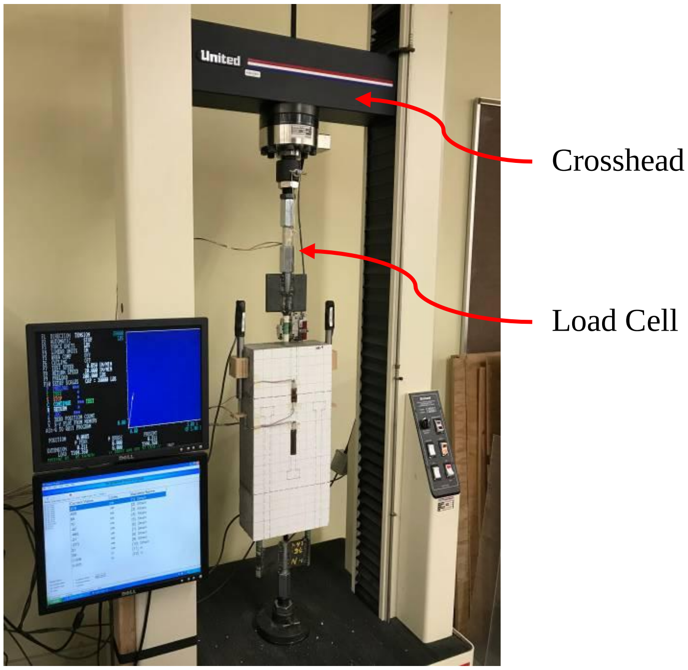


Figure 4.15 Headed Bar Pullout Specimen in United Testing Machine

Figure 4.16 shows a diagram of the load transfer from the bar to the head. The green squares represent the strain gages. Shear stresses along the perimeter of the bar are caused by the concrete. The head experiences bearing stress due to the enlarged coupling at the end. These two stresses are summed to equivalent forces. Rebar strain gages were used to calculate the force in the head as depicted by the red arrow.

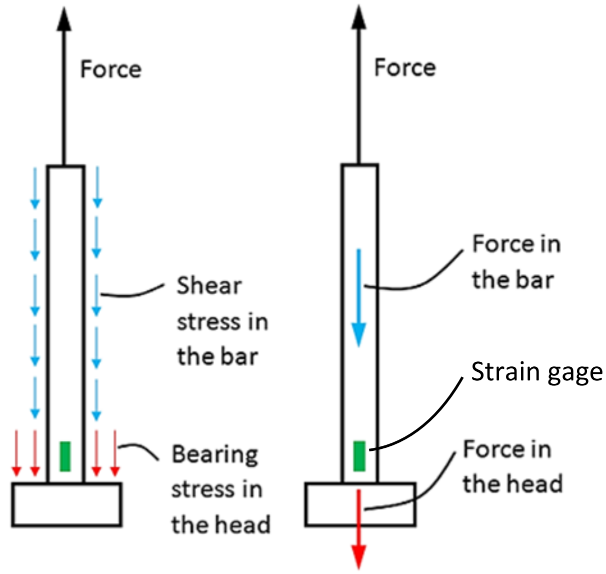


Figure 4.16 Diagram of Load Transfer in Head of Rebar

The equation for determining the force in the rebar was derived from modulus of elasticity and stress equations. Substituting Eq. (4.1) into Eq. (4.2) and solving for F gives Eq. (4.3). The force resisted by the head of the headed bar was calculated as:

$$E = \frac{\sigma}{\varepsilon} \tag{4.1}$$

$$\sigma = \frac{F}{A} \tag{4.2}$$

$$F_{rebar} = E_{steel}\, A_{rebar}\, \varepsilon_{avg.} \tag{4.3}$$

Where

E, E_{steel} = modulus of elasticity, modulus of steel = 29 x10^6 psi

σ = stress (psi)

$\varepsilon, \varepsilon_{avg.}$ = strain (με, 10^{-6})

F, F_{rebar} = force (lb)

A, A_{rebar} = cross-sectional area (in.^{2})

4.5.2 Flexural Large Beam

Beam tests used similar instrumentation. Two 0.25 in. strain gages were attached to each of the two central headed rebars. Since there was space on the bottom of the beams the longer 2 inch strain gages were used for the concrete. The concrete gages were placed in the center of the closure pour between the spliced rebar. Figure 4.17 shows the diagram of the strain gages and Figure 4.18 shows the gages installed. LVDTs were used to measure the mid-span deflection of the beam. The LVDT wood mounting blocks were attached to a piece of plywood that was clamped to the frame of the Tinius Olsen as seen in Figure 4.19. Steel plates were attached with epoxy to the top of the beam in the center which extended to the outside of the frame for the LVDTs to measure.

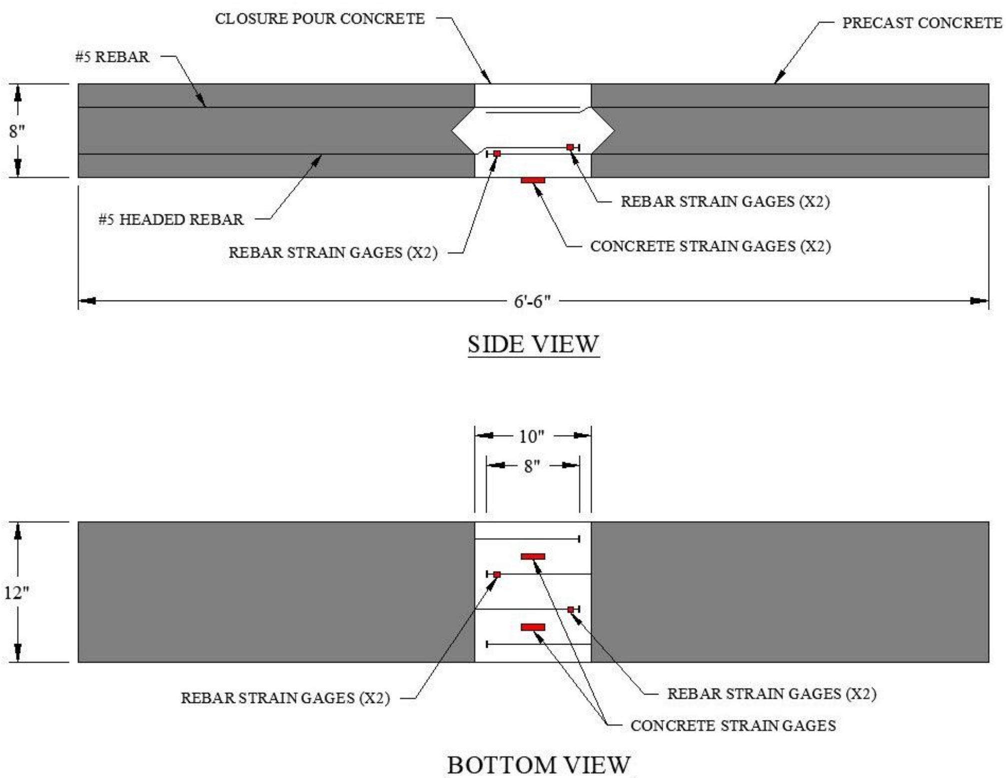
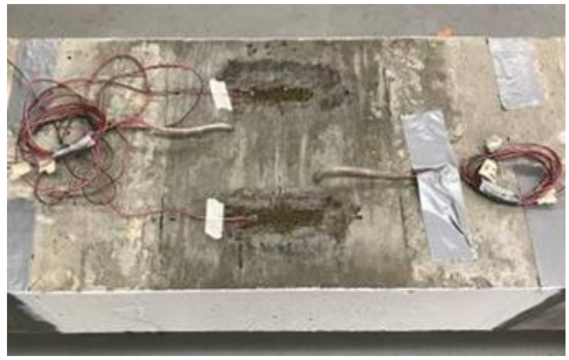


Figure 4.17 Instrumentation Plan for Large Beams

(a) Rebar gages



(b) Concrete gages

Figure 4.18 LB Strain Gages Installed

(a) LVDT mount clamped to frame

(b) LVDT extension bracket

Figure 4.19 LVDT Set-Up for LB Tests

Figure 4.20 shows the diagram of the flexural beam test set-up. Before loading the connecting bolts for the steel beam to the steel column supports were removed. The Tinius Olsen contains a hydraulic pump underneath the loading platform so when the pump is activated the platform is lifted up into the crosshead. This means the steel beam is unsupported at the ends and is only support by the loading platform.

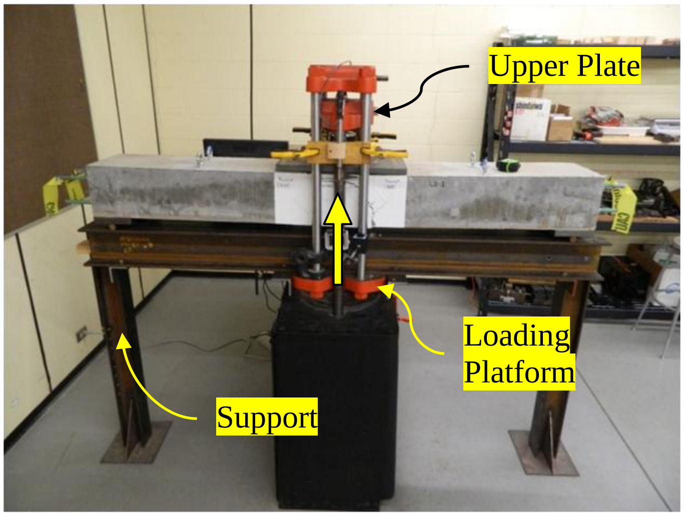


Figure 4.20 Beam Test Set-Up

LVDT measurements were adjusted to account for bending of the steel beams. Concrete beam and steel beam deflections are shown in Figure 4.21. As the Tinius Olsen platform rises the load is applied to the concrete beam and the steel beams. Half the load is transferred from the concrete beam to each of the roller supports at either end. Since the beam is bending two point loads are created at the ends of the loading platform. The deflection of the steel beam needs to be accounted for.

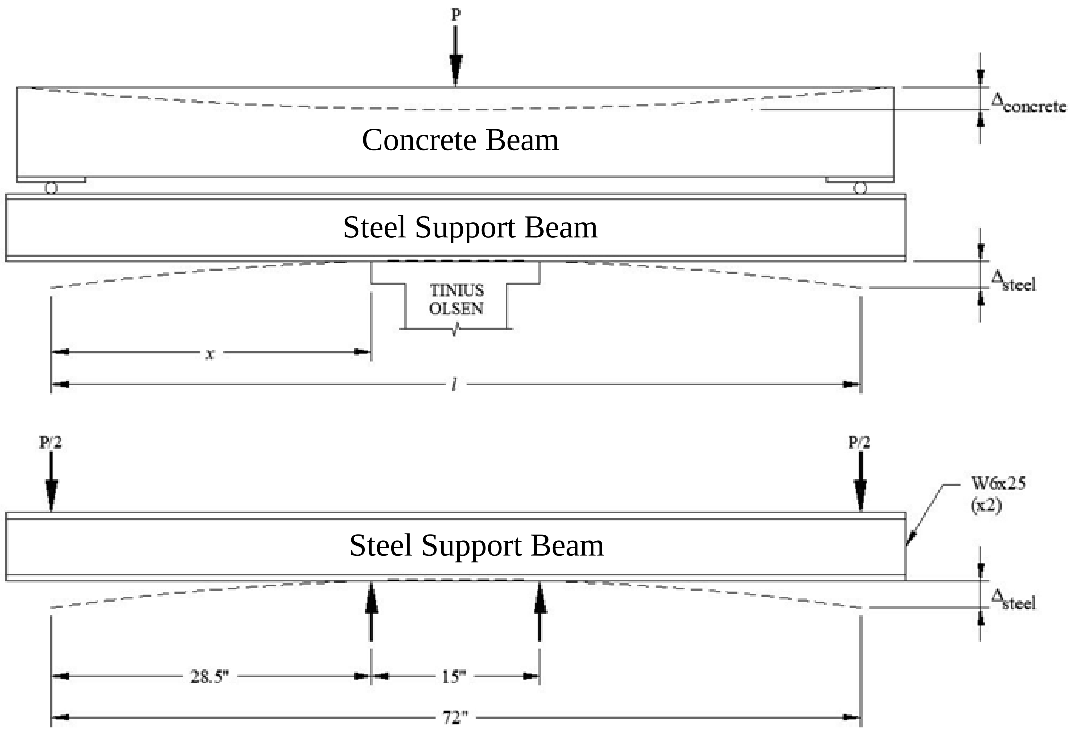


Figure 4.21 Beam Deflection Diagram

The measured displacement from the LVDTs is the sum of the concrete beam deflection and the steel beam deflection expressed as,

$$\Delta_{measured} = \Delta_{concrete} + \Delta_{steel} \tag{4.4}$$

Where

$\Delta_{concrete}$= mid-span concrete beam deflection (in.)

$\Delta_{measured}$ = displacement measured from LVDT (in.)

Δ_{steel} = deflection of the steel beams (in.)

The loading condition of the steel beam in Figure 4.21 is a simple beam with two equal concentrated loads symmetrically placed. From the deflection diagrams in Table 3-23 of the AISC Steel Construction Manual (2017) the deflection equation was obtained for the corresponding load case. Eq. (4.5) shows the steel deflection, Δ_{steel}, equation for the load case shown in Figure 4.21. Since there are two steel support beams, the moment of inertia is multiplied by two.

$$\Delta_{steel} = \frac{(P/_2)x}{6E(2I)}(3lx - 4x^2) = \frac{Px}{24EI}(3lx - 4x^2) \tag{4.5}$$

Where

P = applied load (lb)

x = distance from end of beam = 28.5 in.

E = modulus of steel = 29×10^6 psi

I = moment of inertia for W6×25 = 53.4 $in.^4$

l = length between roller supports = 72 in.

Solving for Δ_{steel} from Eq. (4.5) and using Eq. (4.4) to solve for the concrete deflection, $\Delta_{concrete}$, and substituting into Eq. (4.6) gives:

$$\Delta_{steel} = P(2.22915 * 10^{-6}) \tag{4.6}$$

$$\Delta_{concrete} = \Delta_{measured} - P(2.22915 * 10^{-6}) \tag{4.7}$$

Concrete beam deflections were calculated from Eq. (4.7) by inputting the corresponding $\Delta_{measured}$ and P values.

4.6 StrainSmart System 6000 Data Acquisition System

This section describes how the test data was processed using the Strain Smart software and the System 6000 DAQ. After being imported into an Excel file, the data was analyzed. The StrainSmart DAQ can select a scan rate ranging from 100-10,000 samples per second. The scan rate is the number of measurements that will be collected per second. The System 6000 is designed for dynamic measurements, which require higher sampling rates. Loading procedures for this project only considered static loading conditions. Static loading does not require high sample rates to capture results. Since this was the case in this project, the sample rate was kept low. Using a slower setting decreases the file size. The sample rate applies to each channel, meaning each channel with a measuring device will be sampled at the same rate. Figure 4.22 shows the different components to StrainSmart DAQ; the computer with the installed StrainSmart software to control the program, the scanner (Model 6100), and the wiring board.

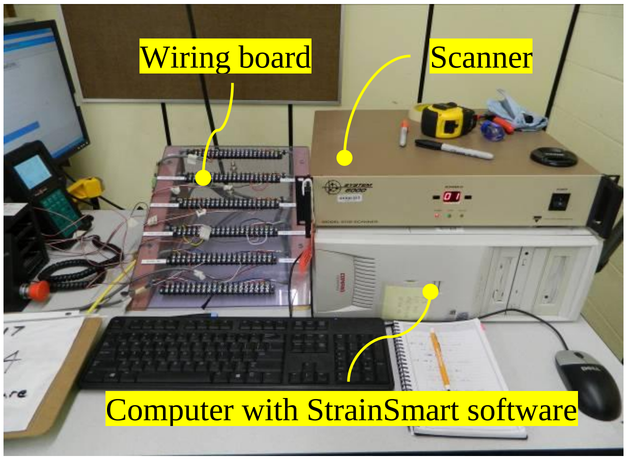


Figure 4.22 StrainSmart Data Acquisition System

Practice tests were conducted to check the sampling or scan rate. The lowest setting of 100 samples per second was selected. This setting repeatedly produced an error which ceased data collection due to inadequate scan rate. The error was corrected by increasing the scan rate to 200 samples per second. More trial testing showed this was sufficient and no errors were produced. To keep the data to a minimum, all tests used the 200 samples per second setting. Due to the slow loading for these tests, higher sampling speeds would not be beneficial. Two types of senor cards were used with the data acquisition system for instrumentation (i.e. strain gage and LVDT). Headed bar (HB) pullout tests had a total of 12 channels and large beam (LB) tests had 9 instrumentation channels. Table 4.3 shows the breakdown of instrumentation channels used for each test. More details about the channel assignments can be found in Appendix L. With the aid of the Idaho State University lab manual sensor and program setup were easy. The lab manual describes how to wiring strain gages and LVDTs along with how to run the StrainSmart program.

Table 4.3 Instrumentation Channels for Tests

		HB	LB
Strain gage	Load cell	2	1
	Rebar	2	4
	Concrete	6	2
LVDT		2	2
	Total	12	9

Using a scan rate of 200 scans per second, over 30,000 data points were collected for each channel for the HB pullout tests and 60,000 data points for LB tests. Data was exported to an Excel file. Precision of load and strain data was measured to the nearest one microstrain (i.e. 1×10^{-6}) and displacements were measured to the nearest thousandths of an inch.

To simplify analysis, the data was reduced in post-processing. A trial and error approach showed that taking data points at every ¼ second, or every 50 data points, was sufficient for the experimental tests in this project. Reduction in data did not affect outputs (i.e. maximum or minimum values). After the data reduction, HB pullout and beam tests had approximately 600 and 1,200 data points, respectively, for each channel. File size was decreased significantly after data reduction. Data processing (e.g. converting headed rebar strain data to force values) was completed following data reduction. Data processing was completed using Excel. Results were produced from the Excel data analysis and are presented in Chapter 5.

CHAPTER 5 RESULTS

5.1 Introduction

This chapter presents the results obtained from laboratory tests for mix design properties. It is divided into six main sections: compression strength, splitting tensile strength, shrinkage, interface bond strength, headed bar pullout results, and flexural beam results. Based on the results obtained from the tests conducted on the material properties, namely compression, split tensile, and shrinkage, an optimum mix was determined. Using this optimum mix, interface bond strength was tested which resulted in a best outcome for interface preparation methods. The headed bar pullout test and the flexural beam test determine the behavior of the alternative connection detail.

5.2 Compression Results

In order to obtain the compressive strength of a sample, the cross-sectional area was first determined. Two diameter measurements were taken perpendicular to each other at mid-height of the specimen. Using the average diameter, the cross sectional area was calculated utilizing the equation for a circle.

After the samples were measured, they were tested according to the methodology in Chapter 3.5.1. Following testing, the peak load was recorded and used to calculate the compressive strength using Eq. (5.1).

$$f'_c = \frac{P}{A} \tag{5.1}$$

Where,

f'_c = compressive strength (psi)

P = peak load (lb)

A = cross sectional area (in^2)

Standard deviation and coefficient of variation were also calculated for 28-day compressive strengths for each set. Results are shown in Table 5.1.

Table 5.1 Average Compressive Strength

Mix	1-day compressive strength (psi)	28-day compressive strength (psi)	Standard deviation, 28-day (psi)	Coefficient of variation, 28-day
A (control)	3,196	7,752	374	4.8%
B	3,442	8,471	281	3.3%
C	3,551	7,860	332	4.2%
D	3,074	8,864	623	7.0%
E	2,550	7,710	260	3.4%
F	2,723	8,161	385	4.7%

The 1-day compressive strength was a design criterion with a minimum value of 3,000 psi, as specified by ITD. At the same accelerator dosage Mixes E and F were below the minimum at 2,550 psi and 2,723 psi, respectively. Mixes containing SRA or SRA and BA (Mixes D, E, and F) resulted in lower 1-day strength values compared to the control. More accelerating admixture could be added to increase the 1-day compressive strength. As seen in Table 5.1, the highest 28-day compressive strength attained was 8,864 psi for Mix D. The coefficient of variation for all mix designs were within the 3-7% range. Figure 5.1 is a graphical representation of the data provided in Table 5.1. The error bars in Figure 5.1 represent the standard deviation of the mixes considered. Refer to Appendix D for compression result data for all specimens tested.

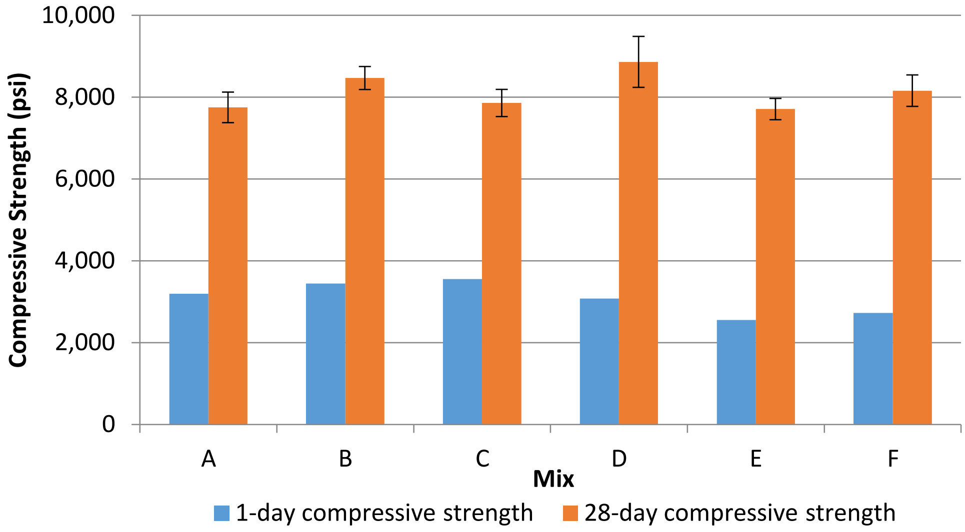


Figure 5.1 Average Compressive Strengths

5.3 Split Tensile Results

Using a straight-edge, a line was first drawn along the length of the sample. Utilizing a combination square, a second line was then drawn across the end connecting it with the first line. Three diameter and two length measurements were taken. After the specimens were measured, they were tested according to the methodology outlined in Chapter 3.5.2. After the conclusion of testing, the peak load was recorded and the splitting tensile strength was calculated using Eq. (5.2)

$$T = \frac{2P}{\pi LD} \tag{5.2}$$

Where,

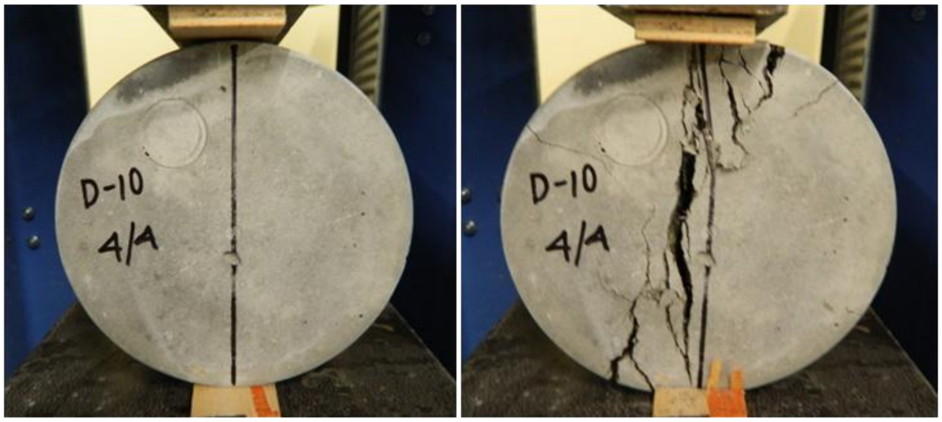
T = splitting tensile strength (psi)

P = peak load (lb)

L = average length (in)

D = average diameter (in)

Figure 5.2 shows a typical split cylinder test before and after failure. Standard deviation and coefficient of variation were also calculated for each set. The results are shown in Table 5.2.



(a) Before Test (b) After Failure

Figure 5.2 Split Tensile Test

During the progression of the test, a crack develops along the vertical plane of the cylinder. Mixes without fibers exhibited sudden failures, whereas mixes containing fibers had more ductility. This was a result of the fibers holding the concrete together which is shown in a close-up view in Figure 5.3, where fibers can be seen in the crack.

Table 5.2 Average Splitting Tensile Strength

Mix	Average split tensile strength (psi)	Standard Deviation (psi)	Coefficient of variation
A	767	31	4.0%
B	634	64	10.1%
C	733	45	6.2%
D	837	54	6.5%
E	749	36	4.9%
F	765	27	3.5%

Figure 5.3 Split Tensile Crack

As seen in Table 5.2 the highest splitting tensile strength was 837 psi obtained from mix D. The addition of polypropylene fibers for mixes B and C showed a decrease in strength compared to the control (mix A) of 133 psi and 34 psi, respectively. All mix designs' coefficient of variation fell in the 3-10% range. Figure 5.4 is a graphical representation of the average split tensile strength and standard deviation values of the data in Table 5.2. Refer to Appendix E for splitting tensile data of all mixes.

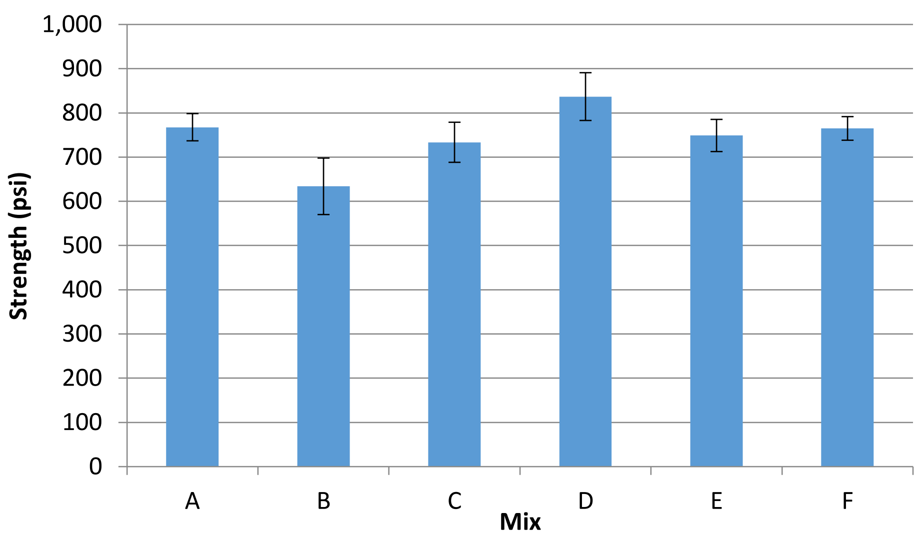


Figure 5.4 Average Splitting Tensile Strength

5.4 Length Change Results

Sample measuring methods followed the procedure outlined in Chapter 3.5.3. Length Change Test. For each mix, there were six samples. An initial length measurement, CRD_{inital}, was taken after removal from the mold at 24 hours, and this was the baseline for determining changes in length according to Eq.(5.3).

$$\Delta L_x = \frac{CRD - CRD_{inital}}{G} \tag{5.3}$$

Where,

ΔL_x = length change of specimen (με)

CRD = difference between the comparator reading of the specimen and the reference bar reading (in)

G = gage length = 10 in

Negative length change values indicate shrinkage and positive values represent swelling. Samples were stored on a metal rack spaced at least one inch apart in accordance with ASTM C157 (2017c) for air storage.

All specimens exhibited swelling, an increase in volume, during moist curing caused by the absorption of water. This swelling is shown on the shrinkage graphs indicated by the positive values at zero days. Once the specimens were removed from water they began to shrink caused by the loss of water. Specimens provided good reading except for one sample from mix A. After 28 days, sample A-4 began to plateau as seen in Figure 5.5. This inconsistency was due to the embedded depth of the gage studs in the concrete. One of the gage studs was set too far into the concrete thus reducing the overall length of the specimen. The reduced length was too short to properly seat into the comparator to get a reading. The comparator height needed to be set higher to accommodate the short sample, but this was not discovered until the end of testing. The comparator was reset and the reading for A-4 at 308 days was consistent with the other samples. All other specimens were not affected. Some graphs showed lines with abrupt changes and this is seen in Figure 5.5 with sample A-2 at 196 days where there is a decrease of 50 microstrain. This is only a small fluctuation which could be due to measurement error or room conditions (e.g. temperature or humidity). The majority of the samples follow a steady curve but there were some fluctuations as previously described either increasing or decreasing abruptly, but these variations were acceptable. Data was collected up to 336 days after removal from moist curing. Average shrinkage curves are shown in Figure 5.6. Average values for mix design "A" excluded specimen A-4.

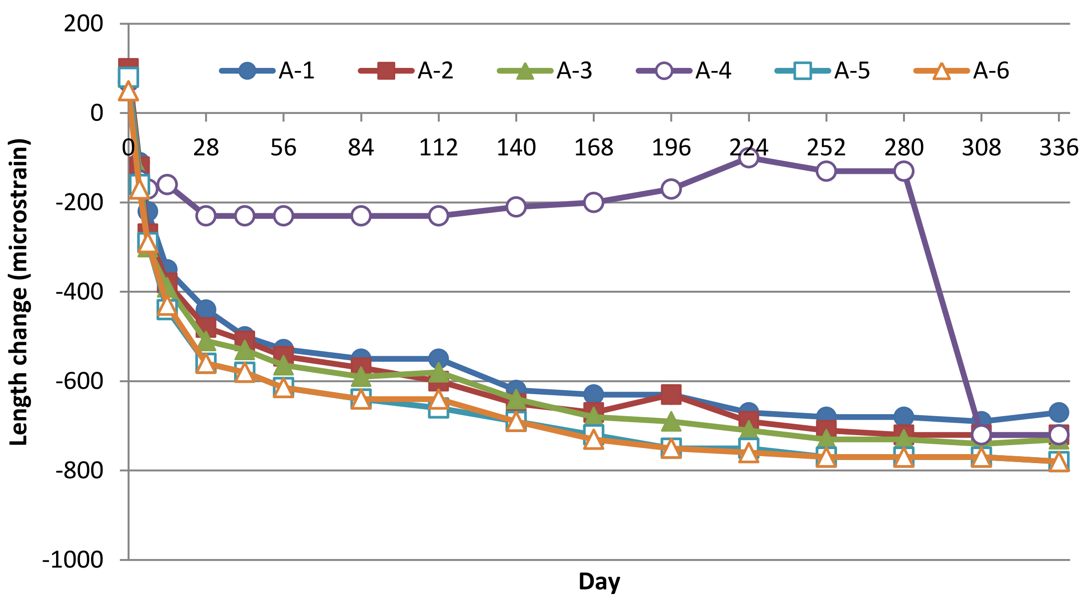


Figure 5.5 Shrinkage - Mix A

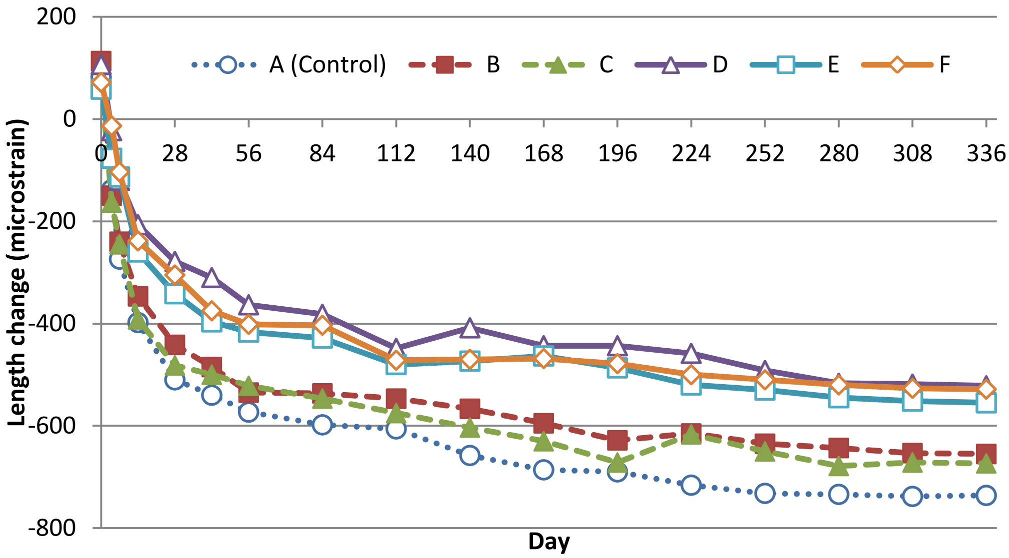


Figure 5.6 Average Shrinkage

The shrinkage curves follow similar trends with the majority of shrinkage, about 70-80% of the long-term shrinkage, occurring in the first two months. For this study, the long-term drying shrinkage is considered to be the length change at 336 days. Long-term shrinkage values for each mix are shown in Table 5.3. Also shown is the percent reduction from the control value.

Table 5.3 Long Term Drying Shrinkage

Sample	Microstrain	Percent Reduction
A (Control)	-736	-
B	-655	11%
C	-673	9%
D	-522	29%
E	-555	25%
F	-528	28%

As noted in Table 5.3, the lowest shrinkage value was 522 microstrain for mix D, a 29% reduction from the control. Addition of fibers decreased shrinkage, as seen with Mixes B and C. Use of shrinkage reducing admixture (SRA) had a significant effect on decreasing shrinkage. The average shrinkage for mixes containing SRA (i.e. Mix D, E, and F) is 535 microstrain compared to the average shrinkage for mixes only containing fibers (i.e. Mixes B and C) at 664 microstrain. The difference between Mixes B and C was not significant, this is most likely due to the low range of fiber dosages of 0.75 lb/yd^3 and1.5 lb/yd^3 (0.05% and 0.1% by volume), respectively. Refer to Appendix F for length change data and graphs for all mixes.

Based on the compression, split tensile, and shrinkage results, the optimum mix was determined to be Mix D, which contained 1.5 lb/yd^3 of fiber and SRA. Shrinkage results between Mix D, E, and F were comparable but Mix D had higher strength values and better workability.

5.5 Interface Bond Results

As described in Chapter 3.5.5 of the methodology section, the optimum mix was used to test interface bond strength with precast concrete. As recommended by the ITD Technical Advisory Committee, a set of samples with Mix E, which contained 0.75 lb/yd^3 of fiber, SRA, and BA, was also tested. The benefit of bonding admixture was not determined from the material property tests so the interface bond tests provided the researchers with an indication of the performance. A bonding agent, Tammsweld, was applied to half of the samples.

Prior to testing, marks were drawn on the top and bottom faces to align the specimen with the supports of the testing apparatus as shown in Figure 5.7. Samples were tested by turning them on their side to provide flat surfaces for loading. Beams were tested 28 days after casting the closure concrete. The age of the precast concrete was approximately 8 weeks at the time of testing. Once the samples were marked, they were tested following the methodology outlined in Chapter 3.5.5 Interface Bond Test. Once the test concluded, the break type and peak load were recorded. Three depth and three width measurements were taken at the plane of failure. Measurements were averaged to calculate bond strength using Eq. (5.4).

$$R = \frac{PL}{bd^2} \tag{5.4}$$

Where,

R = modulus of rupture or bond strength (psi)

P = maximum load indicated by testing machine (lb)

L = span length = 18 in.

b = average width

d = average depth

(a) Beam Specimen

(b) Alignment marks for support and loading locations

Figure 5.7 Interface Beam Test Set-Up

All interface beam specimens failed at the interface between the precast concrete and the closure pour concrete. Figure 5.8 shows the failure at the interface that was typical for all samples. During testing, two distinct failure behaviors were observed. The first was a sudden failure of the beam after the maximum load was attained, this was seen for samples without bonding agent. The second failure type was more ductile with samples reaching their peak load then undergoing more deflection before ultimate failure. Bonding agent caused this behavior. The majority of aggregates debonded while only a small portion of aggregates fractured, ranging from 6-11% for all sets. Mix D without bonding agent had 11% of aggregates fracture and 9% for samples with bonding agent. Mix E without bonding agent had 6% for samples prepared without bonding agent and about 7% for samples prepared with bonding agent. There was no significant difference in the percent of aggregates fractured between specimens prepared without bonding agent and specimens prepared with bonding agent. Figure 5.9 shows the failure plane of specimen D-2 (with bonding agent). The failure occurred along the interface between the two concretes. Data for interface bond tests can be found in Appendix H.

Figure 5.8 Interface Bond Test Sample –D-4 without Bonding Agent

The fractured aggregate in Figure 5.9 was determined by identifying simialar aggregates in both failure planes. Residue of bonding agent was seen on both interface surfaces, but the precast interface had more because the bonding agent was applied to the precast.

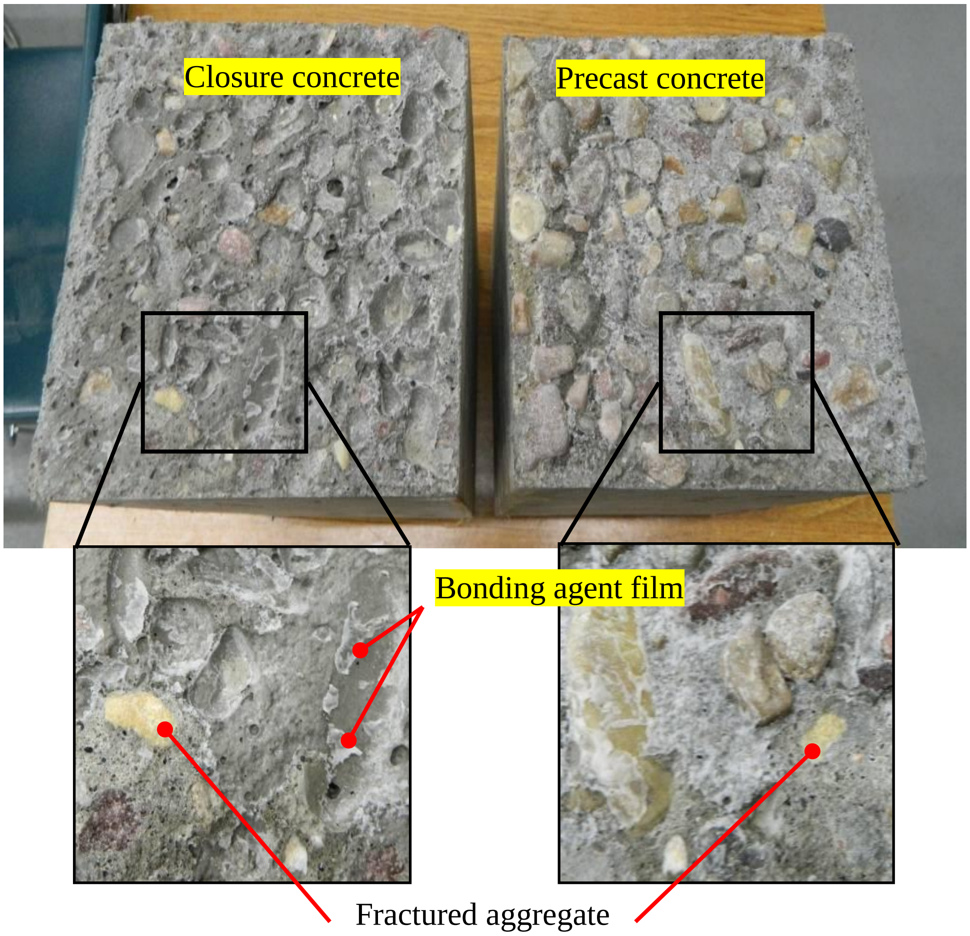


Figure 5.9 Interface Bond Failure – D-2 with Bonding Agent

Average bond strength results are shown in Table 5.4. Standard deviation and coefficient of variation have also been calculated for each set. Figure 5.10 is a graphical representation of the data provided in Table 5.4. The error bars in Figure 5.10 represent the standard deviation of the mixes considered.

Table 5.4 Interface Bond Strength Summary

Mix		Average strength (psi)	Standard deviation (psi)	Coefficient of variation	Difference w/ and w/o BG
D	w/o BG	612	78	12.8%	
	w/ BG	436	32	7.5%	176
E	w/o BG	561	57	10.1%	
	w/ BG	386	9	2.2%	175

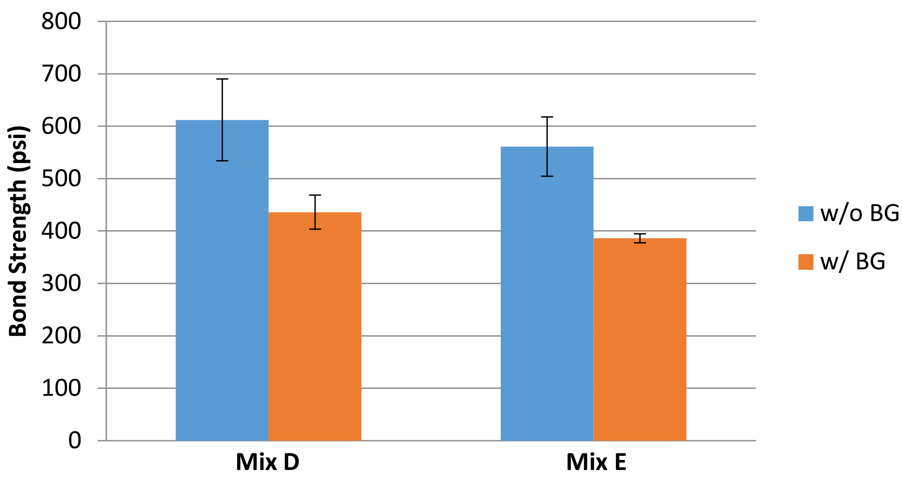


Figure 5.10 Average Interface Bond Strength

Bonding agent had an adverse effect on bond strength, resulting in a 29% and 31% reduction in strength for specimens D and E, respectively. Mix E without bonding agent had a lower strength by 51 psi (8% difference) than Mix D without bonding agent. In addition, Mix D, with no bonding agent, achieved the highest bond strength of 612 psi. Standard deviations differed by approximately 47 psi for mixes with and without bonding agent.

Based on these results, Mix D prepared with a saturated surface dry interface moisture condition, without bonding agent, was selected for use for the remaining headed bar pullout and flexural beam tests. HB pullout results are discussed in the next section.

5.6 Headed Bar Pullout Test

Pullout specimens were cast and prepared as described in the Chapter 3.5.6. A total of six specimens were tested in tension with the same loading. The headed bar specimens were designated as HB1, HB2, HB3, HB4, HB5, and HB6. Specimens were tested 28 days after casting the closure concrete. Precast concrete age was 74 days at the time of testing. A typical test set-up is shown in Figure 5.11.

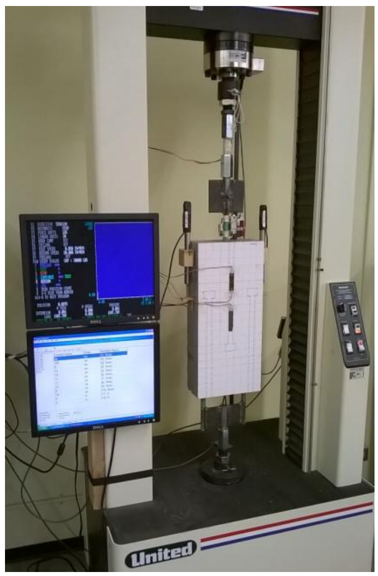


Figure 5.11 Typical Headed Bar Pullout Test Set-Up

5.6.1 Material Properties

A set of tests was conducted for each set of beams to determine material properties. As the precast and closure pour concrete were poured for the HB pullout specimens, a set of test cylinders were also prepared to determine material properties. These material properties will be used in the finite element computer modeling part of this project. Material properties for cylinders included: compressive strength, splitting tensile strength, modulus of elasticity, and Poisson's ratio.

Table 5.5 shows the results for the modulus of elasticity and Poisson's ratio for the HB pullout specimens. The specimens designated with "D" represent the closure pour Mix D material and the specimens designated with "PC" represent the precast material. The left column contains cylinder markings, where the letters "A" and "B" designate the first and second loading, respectively. Two loadings per sample are recommended by the ASTM to obtain appropriate values.

Table 5.5 Modulus and Poisson's Ratio for HB Pullout Specimen Concrete

Specimen	Modulus, E (psi)	Poisson's Ratio, μ
D-2A	4,444,338	0.190
D-2B	4,376,568	0.193
D-3A	4,306,558	0.192
D-3B	4,161,758	0.200
PC-1A	3,491,588	0.182
PC-1B	3,318,170	0.222*
PC-2A	3,504,815	0.075*
PC-2B	3,381,495	0.197
PC-3A	3,129,873	0.165
PC-3B	2,960,532	0.136

*Not included in average

Concrete material properties were averaged and summarized in Table 5.6. Since the concrete Poisson's ratio values typically have a range between 0.11 to 0.21, values outside this range were not included in calculating the average values. The precast concrete was tested 74 days after casting. Poisson's ratio values are typically higher for lower strength concrete and lower for higher strength concrete. Data for the compressive strength, split tensile strength, modulus of elasticity, and Poisson's rate can be found in Appendix D, Appendix E, and Appendix G, respectively.

Table 5.6 Headed Bar Pullout Tests Concrete Material Properties Summary

	Compressive strength (psi)	Split tensile strength (psi)	Modulus, E (psi)	Poisson's Ratio, μ	Age (days)
Closure	8,453	768	4,322,306	0.194	28
Precast	5,258	614	3,297,745	0.170	74

Design equations were used to compare with experimental results. The American Concrete Institute (ACI) and AASHTO equations for calculating the modulus of elasticity are related to the compressive strength of concrete. The ACI and AASHTO equation for compressive strength f'_c up to 6,000 psi is,

$$E_c = 33K_1 w_c^{1.5}\sqrt{f'_c} \tag{5.5}$$

For compressive strength f'_c greater than 6,000 psi ACI recommends the following equation,

$$E_c = \left(40{,}000\sqrt{f'_c} + 10^6\right)\left(\frac{w_c}{145}\right)^{1.5} \tag{5.6}$$

Where,

E_c = modulus of elasticity (psi)

f'_c = compressive strength (psi)

w_c = unit weight (lb/ft^3)

AASHTO includes an additional aggregate factor, K_1, in Eq. (5.5) which is taken as 1.0 unless determined by testing. Density of concrete for Mix D and precast was 143 lb/yd^3 and 144 lb/yd^3, respectively. Using Eq. 5.5 and 5.6, the calculated values are compared to the experimental in Table 5.7. Measured values were lower than calculated values by 6.8% and 21.4% for the closure and precast concrete, respectively.

Table 5.7 HB Material Modulus of Elasticity Comparison

Mix	Calculated (psi)	Measured (psi)	Percent Difference
Closure	4,464,431	4,158,839	6.8%
Precast	4,144,496	3,258,977	21.4%

5.6.2 Test Results

Samples were tested as described in Chapter 3.5.6. A sample was inserted into the United Testing Machine and instrumentation was connected to the Strain Smart system. Before testing, the instruments where zeroed and the DAQ started recording data. Testing began by loading at a constant rate of 0.01 in./min. Data for HB pullout specimens was collected up to the point of failure. Data past the failure point was not useful since many of the strain gages broke or were damaged in the process. Figure 5.12 shows the diagram referencing the labels corresponding to the concrete strain gages. Upper and middle strain gages are located at the top interface. The lower strain gage is located in the center of the closure concrete. Strain gages were applied to the front and the back side of specimens.

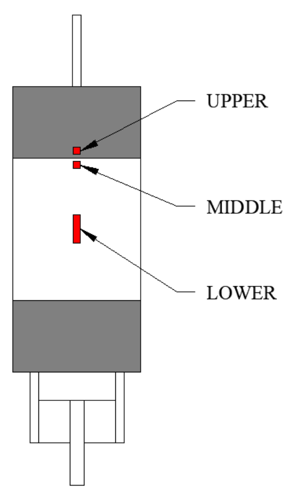


Figure 5.12 Strain Gage Label Diagram

Figure 5.13 shows a graph of the force versus time for all specimens. These graphs give an overview of the test. The initial portion of the lines, in the boxed area, represents the settlement of the specimen in the machine. The fixtures and connections resulted in slack for the specimen. Tensile testing typically applies a preload but to ensure correct zero values for calibration, this testing procedure did not use a preload. Each line has a distinct initial drop in load which indicates cracking at the top interface. Some specimens had several more drops in load before finally reaching the ultimate capacity. After reaching the ultimate load, there was a sudden failure that fractured the specimen at which point the test was concluded.

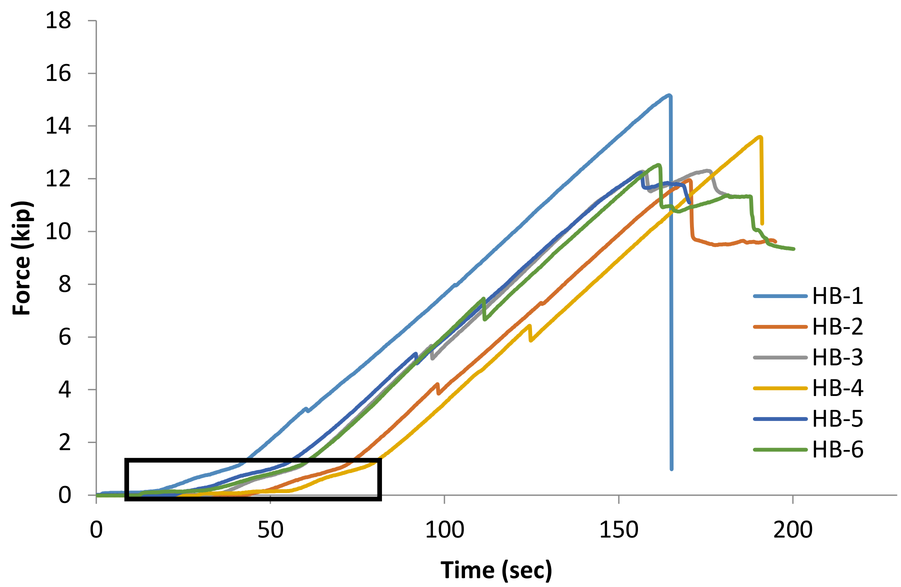
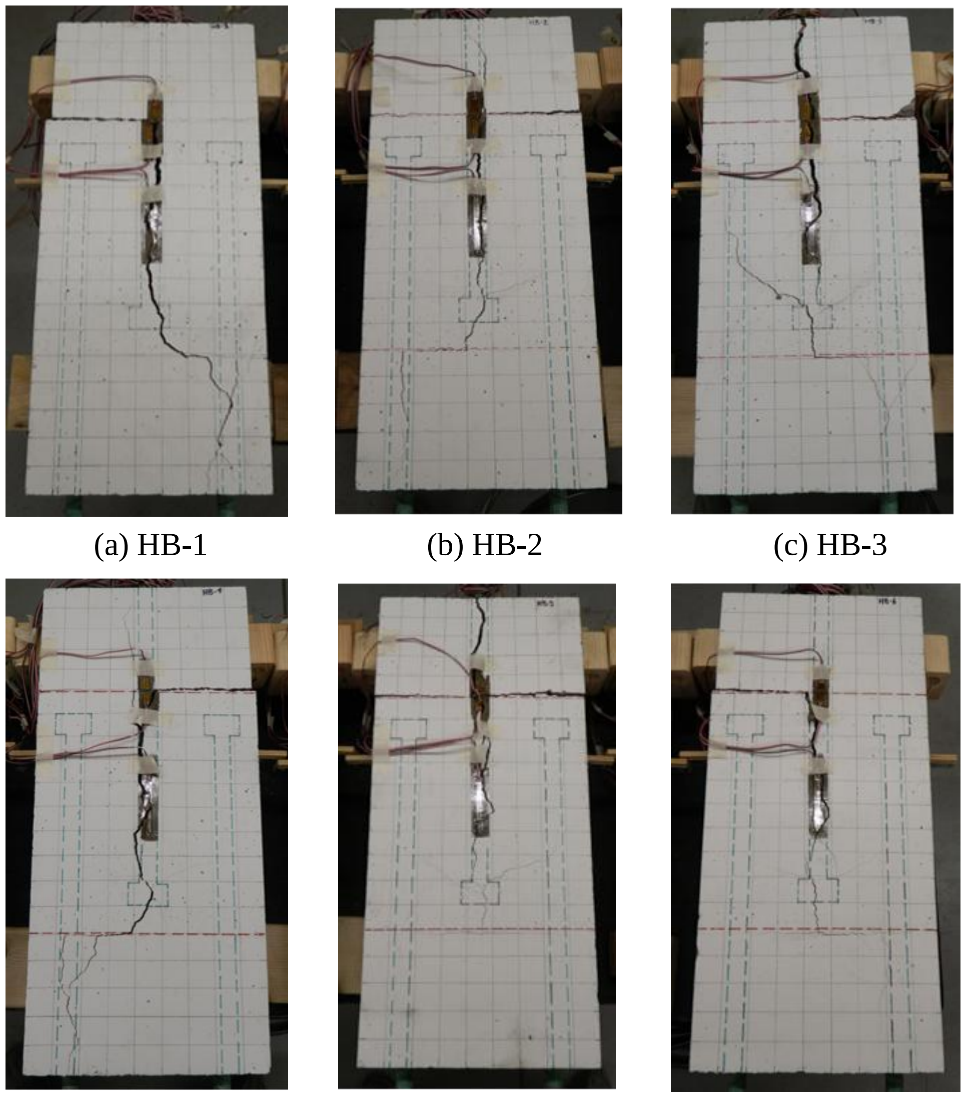


Figure 5.13 Force vs. Time

Cracking initiated at the top interface and propagated horizontally across the sample. The sudden drop around 4,000 lb was the point when the top interface fractured; this will be referred to as the cracking force. Load kept increasing and the bottom interface began to show formation of cracks. After reaching the ultimate load, there was a sudden drop in load and the specimen cracked along the center rebar and down into the lower precast concrete section. Conical cracks also developed at the location of the center rebar head. All six specimens showed similar cracking behavior. Figure 5.14 shows the cracking of all specimens after testing. More detailed pictures of specimen cracks are found in Appendix I.



(a) HB-1 (b) HB-2 (c) HB-3

(d) HB-4 (e) HB-5 (f) HB-6

Figure 5.14 Headed Bar Pullout Samples Crack Pattern

Since the Vishay DAQ could not be incorporated with the United Testing Machine, instrumentation data was extracted from test photos to determine force and displacement recorded by the machine. This data is graphed in Figure 5.15 and shows the points corresponding to cracking and ultimate failure. Data can be found in Appendix I.

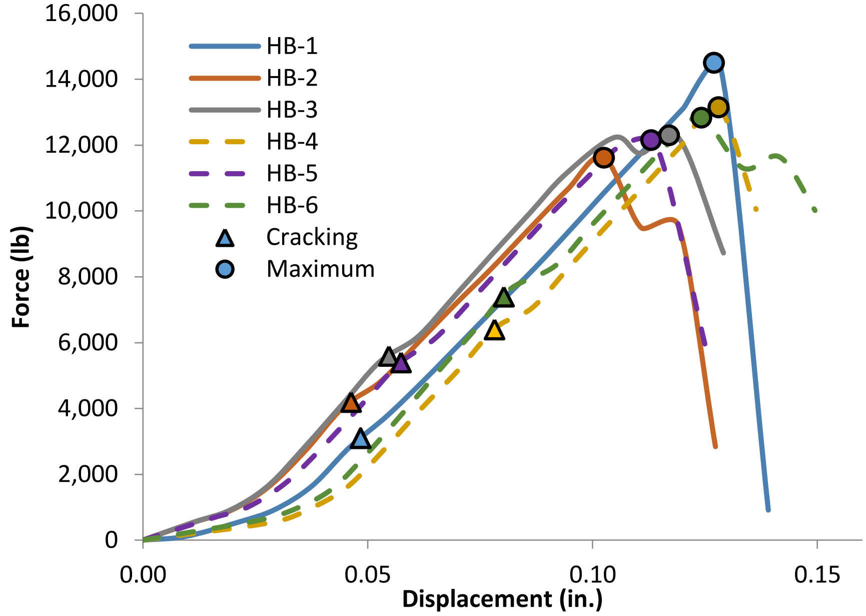


Figure 5.15 Machine Force vs. Machine Displacement

The force resisted by the head of the headed bar was calculated using Eq. (5.3) as described in Chapter 5.3. The applied force from the tensile load cell is graphed against the force in the rebar in Figure 5.16. Specimens showed similar behavior with the exception of HB-1. Lines show a linear behavior up to the cracking load. The applied load drops slightly then load increases again. After cracking, the behavior becomes nonlinear. The graph also indicates the maximum load attained by each specimen.

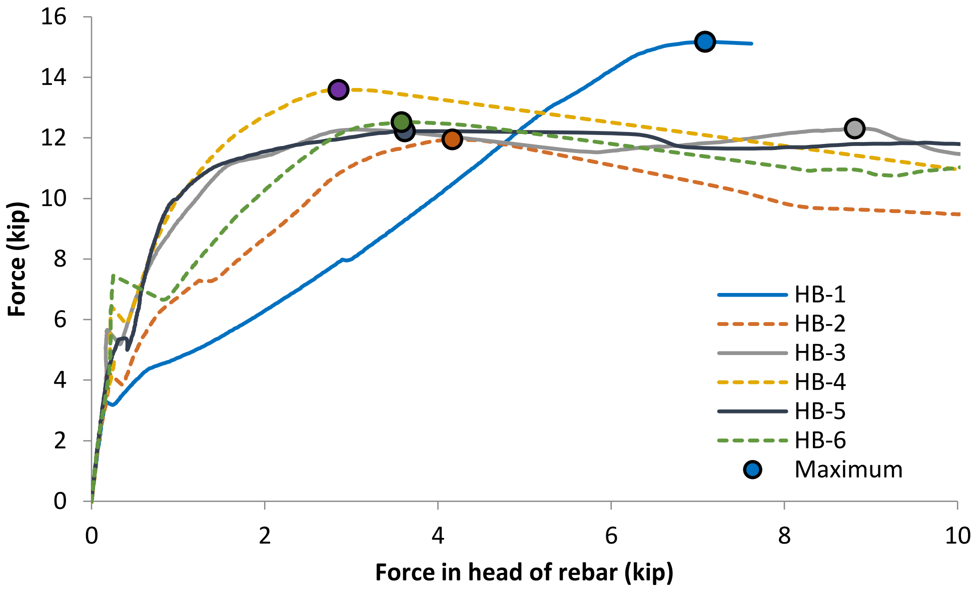


Figure 5.16 Applied Force vs. Force in Head of Rebar – HB Pullout Test

Figure 5.17 graphs the force versus crack expansion for specimen HB-2. The graph shows no displacement until the top interface begins to crack. The initial vertical line is not exactly zero, but this is due to the vibrations from the testing machine. The extension rods of the LVDTs were not secured to the brackets to prevent them from being damaged so any external movement of vibration is read from the LVDT. Once the readings increased this was the first sign of cracking, which was also confirmed by the test pictures. The point where the displacement increased was used as the cracking point. The graphs will also show similar changes in force, strain, or stress that correspond the interface cracking.

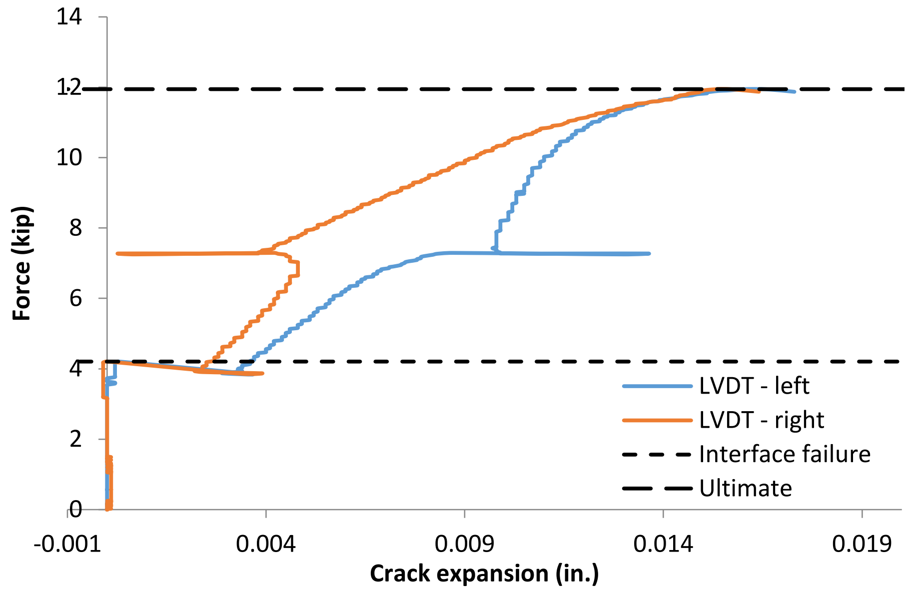


Figure 5.17 Force vs. Crack Expansion – HB-2

Figure 5.18 shows the graph of force versus crack expansion representing specimen HB-4. After the cracking point, the LVDTs increase similarly until they reach approximately 9 kips, where the left and right measurements diverge. This is due to bending of the beam specimen. The interface crack occurred on the right side, of HB-4, causing the crack to expand and put the left side into compression. Bending was most likely due to the rigid connections between the specimen and the fabrication (probable misalignment of end fixtures on the specimen). The other specimens displayed similar trends and are presented in Appendix I.

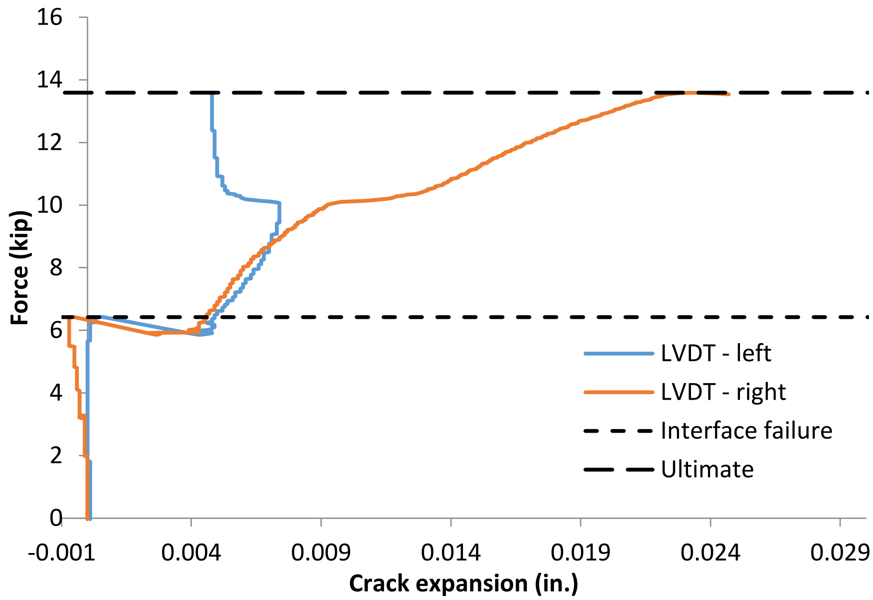


Figure 5.18 Force vs. Crack Expansion – HB-4

Upon reaching the cracking load, the strain gages at the top interface showed a significant drop in strain as shown in Figure 5.19. After cracking, the top and middle strain gages did not show useful data because the failure at the interface. Strain in the lower strain gage increased until cracking was reached, at which point there was a decrease in force. After cracking, the strain increases with the increase in load until failure.

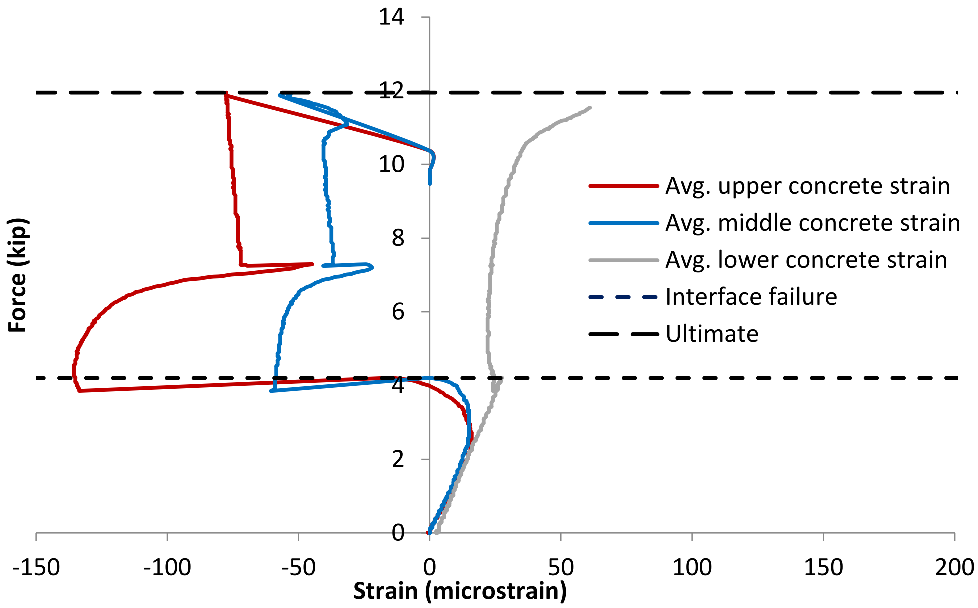


Figure 5.19 Force vs. Average Concrete Strain – HB-1

Table 5.8 shows the values corresponding to the interface cracking point and the maximum specimen capacity. HB-1 was not included in the cracking force and ultimate force averages. The average cracking force and ultimate force were 5,820 lb and 12,519 lb, respectively. Cracking and ultimate loads are graphically shown in Figure 5.20.

Table 5.8 HB Pullout Force, Strains, and Displacements at Cracking and Ultimate

	Failure	Force (lb)	Avg. rebar strain (με)	Avg. Rebar Force (lb)	Avg. upper conc. strain (με)	Avg. middle conc. strain (με)	Avg. lower conc. strain (με)	Avg. displ. (in)
HB-1	Interface	3,245*	25	193	-82	-78	21	0.001
	Maximum	15,176*	901	7,085	-52	273	82	0.019
HB-2	Interface	4,206	28	214	-16	-1	28	0.000
	Maximum	11,949	536	4,167	-78	-55	92	0.016
HB-3	Interface	5,661	24	184	-48	-27	27	0.000
	Maximum	12,310	1,132	8,861	-94	-	-	0.033
HB-4	Interface	6,420	29	235	-69	-38	32	0.000
	Maximum	13,589	365	3,004	-96	-40	131	0.014
HB-5	Interface	5,362	37	311	-40	-18	34	0.000
	Maximum	12,222	430	3,617	-72	343	177	0.013
HB-6	Interface	7,451	32	253	-21	-21	35	0.000
	Maximum	12,522	458	3,674	-171	116	113	0.013
Avg.	Interface	5,820						
	Maximum	12,519						

*Not included in average

Note: No value indicates one or both strain gages broke and an average was not calculated.

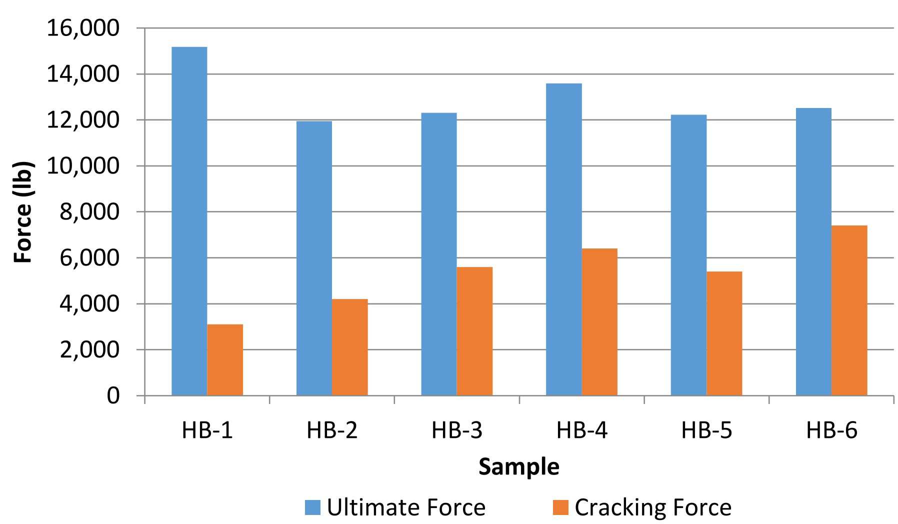


Figure 5.20 Headed Bar Pullout Force

5.7 Flexural Large Beam Test

Flexural beam tests were the final phase of the experimental work conducted for this research. Figure 5.21 shows a typical test for a beam in three-point bending. Two sets of beam specimens were made for determining the behavior of field-cast connections under three-point and four-point flexural bending. Three beams were used for testing in three-point bending and three were tested in four-point bending. As described in Chapter 3.5.7, the distributed loading represents the footprint of a truck tire. Large beams (LB) designated with 1, 3, and 5 were tested in three-point bending, and the beams designated with 2, 4, and 6 were tested in four-point bending.

Figure 5.21 Typical Three-Point Flexural Beam Test

Closure connections were cast in sets of two, so LB-1 and LB-2 were cast on the same day. Similarly, LB-3 and LB-4 were cast on the same day, as were LB-5 and LB-6. Material property tests were conducted with each set of beams. In all cases, the age of closure pour concrete was 28 days and the precast concrete was 119-123 days; this is the range for all sets.

5.7.1 Material Properties

Similar to the HB pullout tests, a set of cylinders was used to determine material properties of the precast and closure concretes. Concrete properties included: compressive strength, splitting tensile strength, modulus of elasticity, and Poisson's ratio. Modulus of elasticity and Poisson's ratio results are shown in Table 5.9. The specimens designated with "D" represent the closure pour Mix D material and the specimens designated with "PC" represent the precast material. The left column contains cylinder designations, with the digits indicating the sample number and the letters "A" and "B" designating the first and second loading, respectively. Two loadings per sample are recommended by the ASTM standard to obtain appropriate values.

Table 5.9 Modulus and Poisson's Ratio for LB Specimen Concrete

	LB 1 & 2		LB 3 & 4		LB 5 & 6	
	E (psi)	μ	E (psi)	μ	E (psi)	μ
D-1A	-	-	4,465,396	0.200	4,523,335	0.180
D-1B	4,395,077	0.172	4,334,100	0.190	4,464,865	0.192
D-2A	4,665,052	0.128	4,592,779	0.160	4,330,180	0.185
D-2B	4,471,958	0.167	4,456,101	0.187	4,296,753	0.213*
D-3A	4,287,998	0.105*	4,608,954	0.200	4,210,472	0.185
D-3B	4,181,021	0.119	4,533,595	0.195	4,147,582	0.183
PC-2A	3,223,867	0.147	3,334,850	0.163	3,061,867	0.156
PC-2B	3,123,060	0.199	3,302,784	0.122	2,978,904	0.208
PC-3A	3,138,129	0.171	3,250,180	0.157	3,411,948	0.147
PC-3B	3,071,865	0.112	3,226,734	0.078*	3,211,181	0.137
PC-4A	3,333,316	0.080*	3,130,301	0.161	3,271,878	0.258*
PC-4B	3,298,630	0.098*	3,110,401	0.125	3,263,574	0.239*

*Not included in average

Poisson's ratio values outside the 0.11 to 0.21 range were not included in the average. Table 5.10 shows the summary of material properties for beam specimens. Average compressive and splitting tensile strengths for the closure pour concrete and precast concrete were similar to the averages from the headed bar pullout tests. Compressive strengths were 8,354 psi and 4,969 psi for closure pour concrete and precast concrete, respectively. Split tensile strengths were 773 psi for closure pour concrete and 596 psi for the precast. Modulus of elasticity and Poisson's ratio values were still within the ACI design-recommended range. Data for the compressive strength, split tensile strength, modulus of elasticity, and Poisson's rate can be found in Appendix D, Appendix E, and Appendix G.

Table 5.10 Beam Concrete Material Properties Summary

	Compression strength (psi)	Split tension strength (psi)	Modulus, E (psi)	Poisson's ratio, μ	Age (days)
Closure	8,354	773	4,425,365	0.176	28
Precast	4,969	596	3,180,718	0.154	119-123

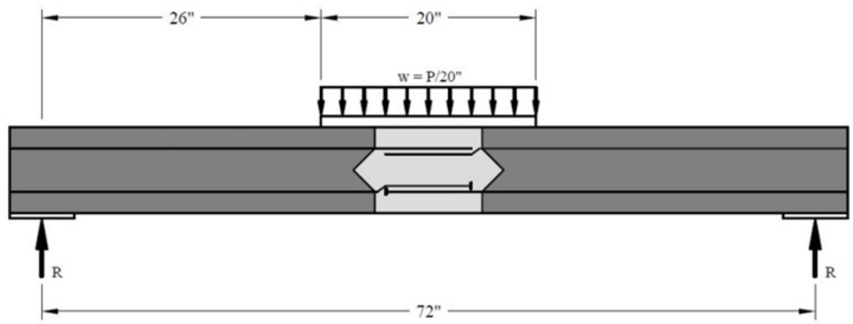
Using Eq. 5.5 and 5.6, the calculated values are compared to the experimental in Table 5.11. Measured values were lower than calculated values by 3.2% and 20.1% for the closure and precast concrete, respectively.

Table 5.11 LB Material Modulus of Elasticity Comparison

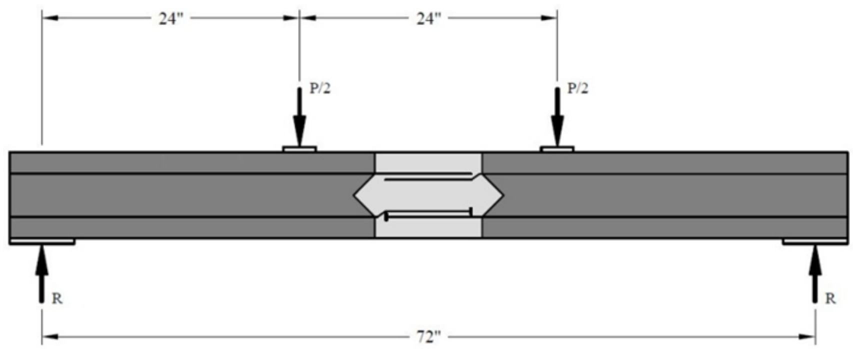
Mix	Calculated (psi)	Measured (psi)	Percent Difference
Closure	4,557,086	4,409,191	3.2%
Precast	4,014,310	3,207,970	20.1%

5.7.2 Test Results

Beam specimens were tested as described in the methodology section in Chapter 3.5.7. Beams were prepared for testing by attaching concrete strain gages as described in Chapter 4.5. Flexural beams were tested in 3-poing and 4-point bending (Figure 5.22). Strain gages were attached to the concrete on the underside of the closure concrete. A coat of white paint was applied to the middle sections of the beam and the interface outlined with marker as shown in Figure 5.23. Similar to the HB pullout tests, the paint assisted with identifying the formation of cracks.



(a) Three-Point Loading



(a) Four-Point Loading

Figure 5.22 Beam Loading Diagrams

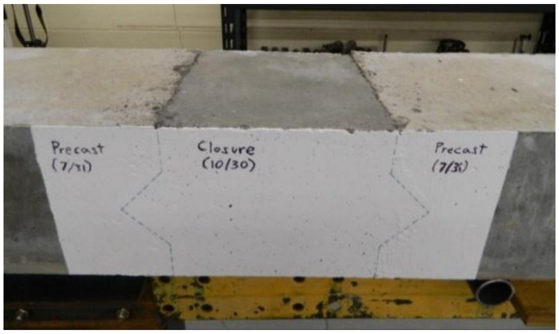


Figure 5.23 Beam Specimen Painted and Interface Lines Drawn

Due to the one-inch thick steel plate distributing the load, the loading for the three-point bending is not a true three-point loading. Even though it is a distributed load it will be referred to as three-point bending. The steel plate with dimensions 20 in. x 10 in. x 1 in. was placed in the center of the beam. The plate distributed the load as seen in Figure 5.22. The load was assumed to be uniformly distributed along the length of the plate. For calculating the moment, the measured load from the DAQ was divided by 20 inches. Eq. (5.7) was used to calculate the moment for the three-point bending case at the center of the beam for any applied load.

$$M = R\left(a + \frac{R}{2w}\right) \tag{5.7}$$

Where,

M = moment (lb-in)

R = support reaction = $\frac{P}{2}$

w = distributed load = $\frac{P}{20''}$ (lb/in)

a = distance from the end support to start of load application

Eq. (5.8) was used to calculate the maximum beam moments in four-point flexural bending.

$$M = \frac{Pa}{2} \tag{5.8}$$

Where,

M = moment (lb-in)

P = load (lb)

L = span length = 72 in

a = distance from the support (reaction) force to the force at the one-third location used in the four-point flexural beam test = 24 in.

d = average depth

Distributing the load over the 20 in. x 10 in. steel plate was not ideal. The top surface of the beam was not perfectly flat or uniform. Gaps between the steel plate and the beam could be seen. A rubber pad was added under the loading plate for samples LB-3 and LB-5 to provide more even distribution of the load and to minimize gaps. The rubber reduced gaps between the steel plate and the concrete with only a few larger gaps visible.

The testing schematic shown in Figure 5.24 identifies the instruments and equipment for beam tests. The schematic refers to the designations for "left" and "right" instrumentation. A monitor and camera were set up to monitor and record the progress of the tests. Figure 5.25 shows the test set-up for a three-point bending test.

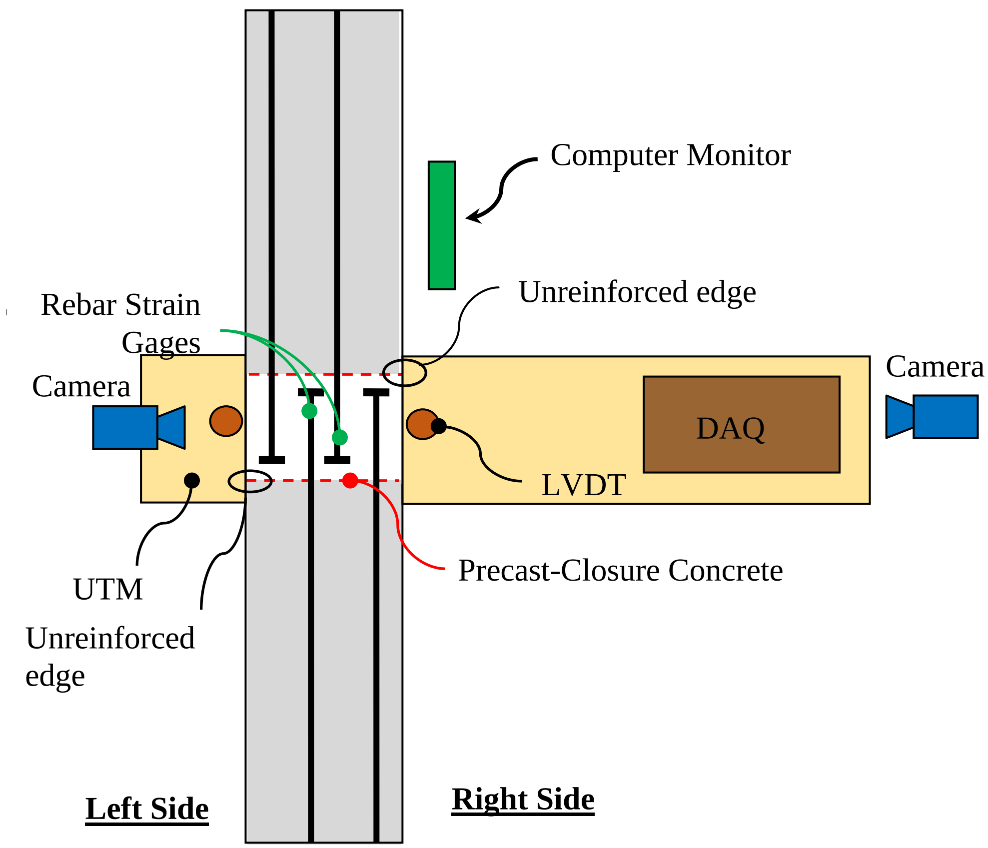


Figure 5.24 Top View of the Flexural Beam Test

Figure 5.25 LB Test Set-Up

The beam failure for LB-2 is shown in Figure 5.26. After testing the beam cracking was mapped by indicating the sequence in which they appeared. The sequence of numbers represents the stages of cracking which are defined as:

1. **Primary**: Precast-closure interface cracked (at both interfaces) and propagated up along the shear key.
2. **Precast**: Several smaller cracks formed (1-3 in. long) in the precast segments. These occurred prior to reaching the ultimate beam capacity.
3. **Shear**: Shear crack started from the lower corner of the shear key and extended up toward the opposite interface. This occurred after reaching the ultimate beam capacity.
4. **Secondary**: Cracks formed as a result of excess displacement. These occurred well beyond the ultimate load.

In all cases the shear crack always formed on the lower right and extended to the top left of the connection. This was a result of the reinforcing and the width of the beam. The right side of the connection was unreinforced as shown in Figure 5.24 and this induced a torsional rotation causing the specimens to form similar cracks. Primary cracks on the right side of the connection were also much larger than the cracks on the left side. Figure 5.27 shows a diagram of the typical cracking for flexural beam specimens. The crack appeared on the unreinforced edge on both sides of the beam. The secondary cracks were mainly to observe the cracks extending toward the top of the beam. After inspecting the beams, no cracks were seen extending to the surface. The secondary horizontal cracks were 0.5-1 in. below the surface. Figure 5.28 shows the bottom of LB-1. Cracks in the connection portion as well as in the precast concrete can be seen. Longitudinal cracks (along the length of the beam) formed below the two headed rebar in the middle of the beam as seen in Figure 5.28. Precast cracks can be seen marked in red located about 8 inches away from the connection. Refer to Appendix J for pictures of all beam specimen cracks.

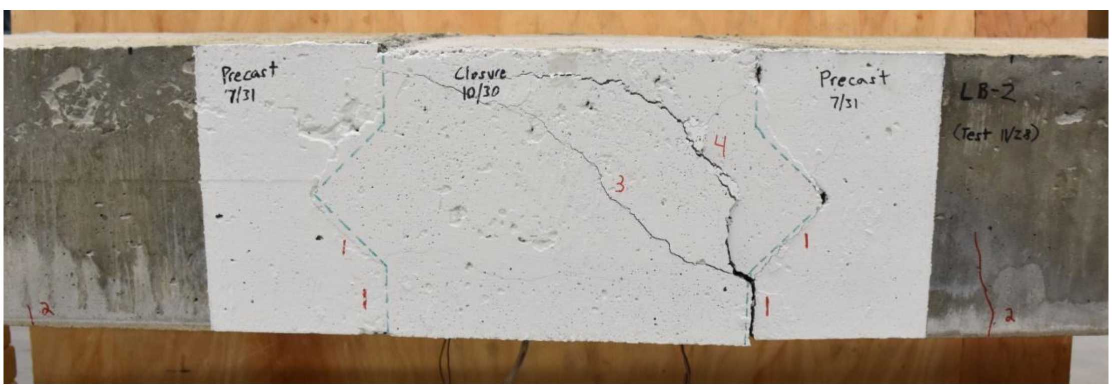


Figure 5.26 Beam Cracking – Specimen LB-2

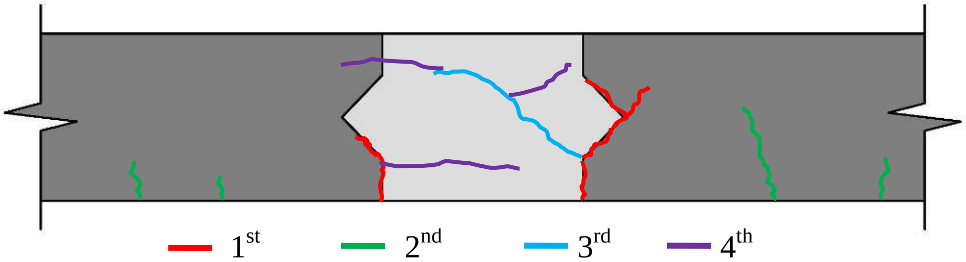
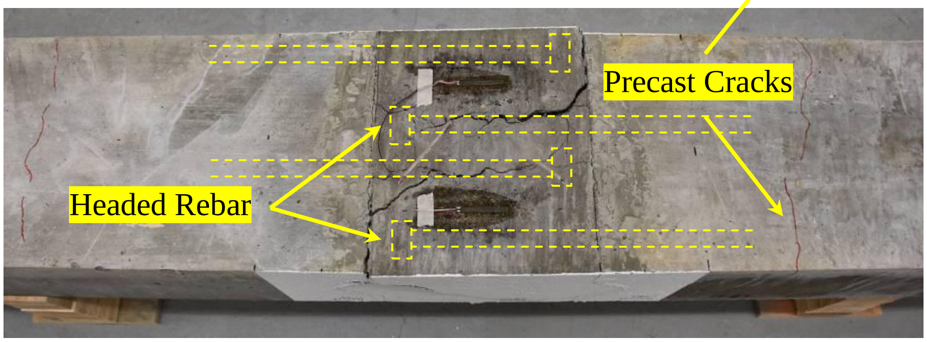
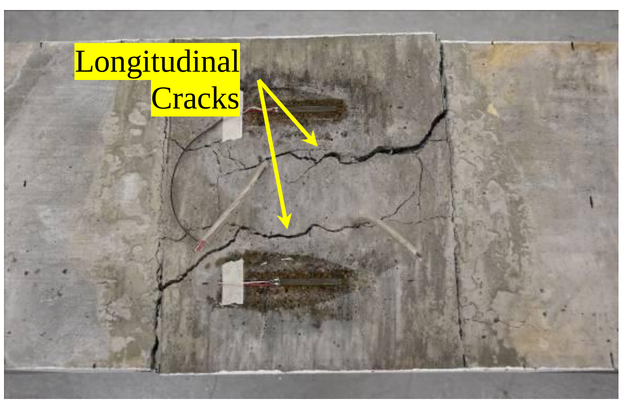


Figure 5.27 Typical Beam Cracking Diagram

(a) Side view



(b) Plan view of bottom



(c) Connection cracks

Figure 5.28 Cracking on Bottom of Specimen LB-1

The force-deflection graph (Figure 5.29) shows all six beams and the points of maximum force for each specimen. The solid lines represent the beams under three-point (3P) bending and the dashed lines represent beams under four-point (4P) bending. Performance of beams LB-1, LB-3, and LB-5 were similar. This is also the case for LB-2, LB-4, and LB-6. Ultimate loads are shown in Table 5.12. Three-point and four-point specimens averaged 9,492 lb and 12,209 lb, respectively. The graph (Figure 5.29) shows failure after the loads decrease until they settle around 6,000 lb. At this point the secondary cracks began to form.

The point of cracking load was not determined from the data, unlike to the HB tests. The cracking load was determined from visual observations of the interface during testing.

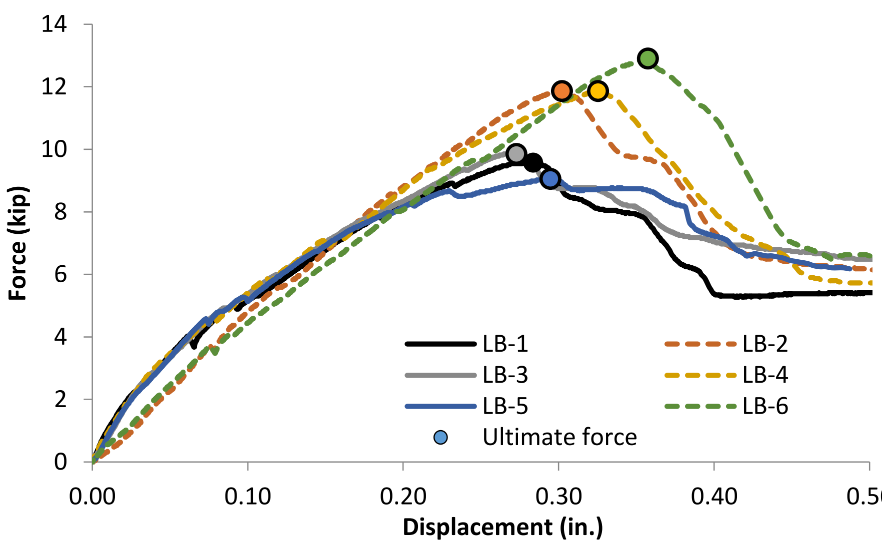


Figure 5.29 Beam Force vs. Deflection

Table 5.12 Ultimate Loads for Flexural Beam Tests

Specimen	Ultimate Load (lb)
LB-1	9,573
LB-2	11,862
LB-3	9,850
LB-4	11,862
LB-5	9,052
LB-6	12,902

Stress at the head of the rebar Moment-Rebar Stress graph is shown in Figure 5.30. From Table 5.13 the average moments for 3P and 4P bending were 147.1 kip-in. and 146.5 kip-in., respectively. Ultimate moment capacity for both test sets were The stress in the rebar head at ultimate capacity had a wide range, from 15 ksi for LB-2 up to 44 ksi for LB-5.

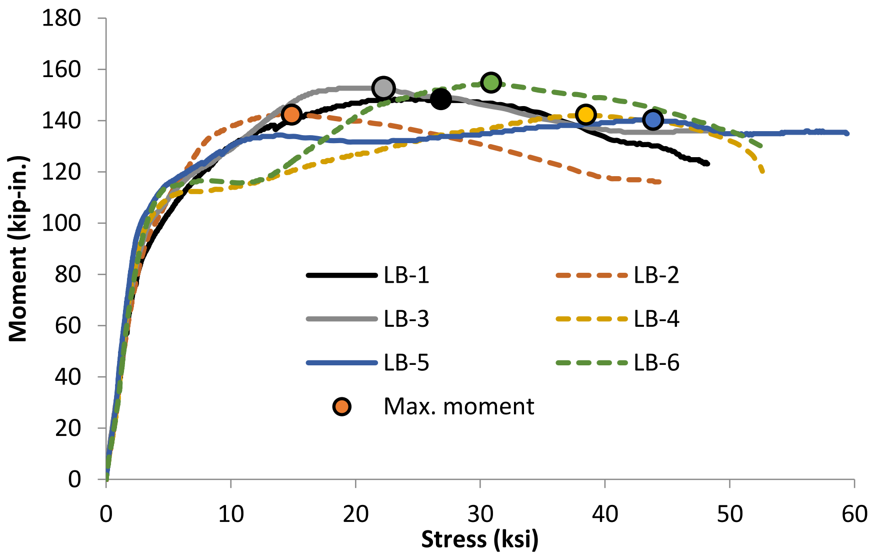


Figure 5.30 Moment vs. Rebar Stress

Table 5.13 Flexural Beam Ultimate Loads and Moments Summary

	Cracking Load (kip)	Cracking Moment (kip-in.)	Ultimate Load (kip)	Ultimate Moment (kip-in.)
3-point	2.9	44.4	9.5	147.1
4-point	2.9	35.2	12.2	146.5

Theoretical cracking and ultimate moments were calculated from reinforced concrete analysis equations. Since the experimental beam included a splice joint the analysis was simplified to a beam with two continuous rebars along the top and bottom and an identical cross-sectional area. Cracking moment was calculated as,

$$M_{cr} = \frac{f_r I_g}{y} \tag{5.9}$$

Where,

M_{cr} = cracking moment (lb-in)

f_r = modulus of rupture of concrete (psi)

I_g = gross moment of inertia of beam cross-section (in^4)

y = distance from bottom of beam to centroid of cross-section (in)

Ultimate moment was calculated as,

$$M_{ult} = A_s f_y \left(d - \frac{a}{2}\right) \tag{5.10}$$

Where,

M_{ult} = ultimate moment (lb-in)

A_s = cross-sectional area of rebar (in^2)

f_y = yield strength of steel (psi)

d = distance from the top of beam to centroid of rebar (in)

a = depth of equivalent rectangular stress block (in)

$= \frac{A_s\, f_y}{0.85\, f'_c\, b}$

f'_c = compressive strength of concrete (psi)

b = width of beam (in)

The theoretical cracking and ultimate moments were 78.3 kip-in. and 206.5 kip-in., respectively. Theoretical values were significantly greater than experimental values. Differences between the calculated and experimental cracking moments were 43% and 55%, for 3-point and 4-point respectively. The ultimate moments had lower percent differences with 29% for both 3-point and 4-point.

Concrete strain gage data is plotted against the beam moment in Figure 5.31. LB-1, LB-2, and LB-3 showed behavior that was comparable to the interface strain gages of the HB pullout tests. They showed an increase in strain up to approximately 50 microstrain then decreased until failure. Specimens LB-4, LB-5, and LB-6 showed a rapid increase in concrete strain as shown in Figure 5.31. Inspecting the strain gages revealed that cracking had occurred under some of the gages and some showed partial debonding of the gage.

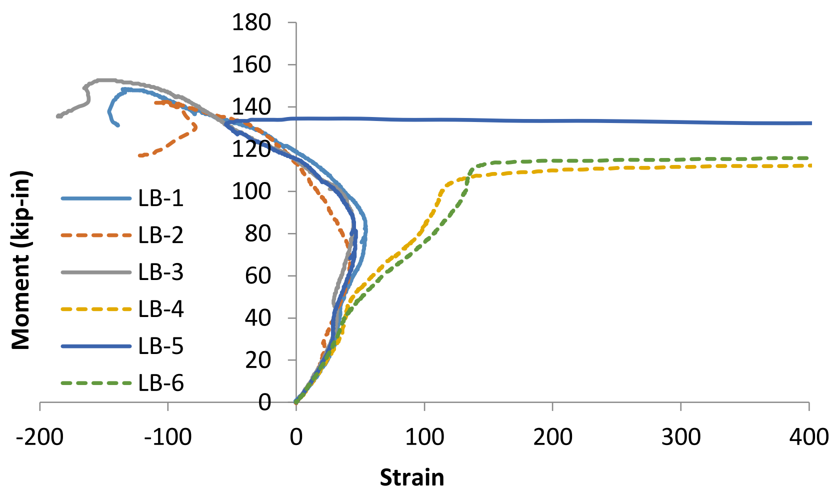


Figure 5.31 Moment vs. Average Concrete Strain

A summary of average values for large beam tests is shown in Table 5.14. The values correspond to the ultimate beam capacity. The results don't show any significant differences between 3-point and 4-point tests expect for displacements. Average displacements for 3-point and 4-point were 0.283 in. and 0.328 in., respectively. The difference between the average displacements was 0.045 inches. Average stress in the rebars ranged from 15-44 ksi. Concrete strain data was inconclusive due to the unreliability of the gages on concrete. Refer to Appendix J for graphs and data for flexural beam tests.

Table 5.14 Flexural Beam – Average Values at Ultimate Capacity

Specimen	Load cell (lb)	Moment (lb-in.)	Avg. rebar stress (ksi)	Avg. rebar force (lb)	Avg. concrete strain (με)	Adjusted displacement (in.)
LB-1	9,573	148,377	25.6	6,879	-134	0.281
LB-2	11,862	142,342	14.9	4,060	-108	0.302
LB-3	9,850	152,678	22.3	6,055	-155	0.273
LB-4	11,862	142,342	38.5	10,197	2,944*	0.326
LB-5	9,052	140,313	43.9	11,818	1,570*	0.295
LB-6	12,902	154,828	30.9	8,231	3,284*	0.357

*Strain gage data is unreliable due to damage caused by cracking or debonding of the strain gages

CHAPTER 6 SUMMARY, CONCLUSIONS, AND FUTURE WORK

The purpose of this chapter is to summarize the experimental results concluded from this research project. In conjunction with the Idaho Transportation Department, this research project set out to design a cost-effective concrete mix that could be used as an alternative for field-cast connections of precast elements in accelerated bridge construction.

6.1 Mix Design Summary

First, six HES concrete mix designs were developed utilizing three variables: polypropylene fiber dosage, shrinkage reducing admixture, and bonding admixture. The optimum mix was chosen based on the following concrete properties:

- Compression strength
- Splitting tensile strength
- Shrinkage

6.2 Compressive Strength Summary

Compression tests were conducted using the Gilson Compression Testing Machine. All samples were tested at a loading rate of 440 lb/sec until failure. Peak loads were recorded. Cross sectional area and peak load were used to calculate the compressive strength of each specimen. The six mixes were each tested at day 1 for design criterion, with a minimum standard of 3,000 psi. Mixes A, B, C, and D all achieved the minimum 1-day strength requirement; Mixes E and F were below the minimum at 2,550 psi and 2,723 psi, respectively. Strength tests were also conducted at 28-days. It was determined that Mix D had the highest compressive strength of 8,864 psi which was 1,112 psi greater than the control (Mix A).

6.3 Splitting Tensile Strength Summary

Each specimen was subjected to a loading rate of 126 lb/sec which was applied until failure. The peak load was recorded and the splitting tensile strength was calculated. During testing, cracks developed along the vertical plane of the cylinder. Mixes without fibers exhibited sudden failure while mixes with fibers were more ductile. It was determined that Mix D had the highest splitting tensile strength of 837 psi.

6.4 Shrinkage Summary

Initial length measurements were taken on each sample after 24 hours which was the baseline used to determine changes. Six samples for each mix were prepared. After removal from the water at day 28, samples were measured and then air dried according to the ASTM standard. Samples were measured at 4, 7, 14, 28, 42, and 56 days, and then every 28 days thereafter. Final measurements were recorded at 336 days. Addition of fibers and shrinkage reducing admixture (SRA) decreased shrinkage, with SRA having a significant effect. Long-term shrinkage values showed that Mix D, which contained both fibers and SRA, had the lowest shrinkage value at 522 microstrain, which was a 29% reduction from the control Mix A.

6.5 Interface Bond Strength Summary

Based on the compression, split tensile, and shrinkage results, the optimum mix was Mix D. This mix was used to test the interface bond strength. The bonding agent Tammsweld, was applied to half of the samples. A set made with Mix E were also tested to determine the effect of bonding agent. Precast concrete interface surface preparation for test specimens followed the recommendation based on the literature review; exposed aggregate (EA) surface finish with a saturated surface dry moisture condition at the precast interface surface. Specimens using bonding agent were prepared with the EA finish and then the agent was applied following the manufacturers recommendations. Next, bond strength tests were conducted to determine the performance of a bonding agent at the interface. All beam failures occurred at the interface. Results showed that bonding agent had an adverse effect on bond strength. Mix D with no bonding agent achieved the highest bond strength of 612 psi. Mix D was chosen to be used for the remaining headed bar pullout and flexural beam tests.

6.6 Headed Bar Pullout Summary

A set of six headed bar pullout samples of were tested to determine the behavior of the connection under a tensile load. Specimens were prepared with exposed aggregate surface finish at the precast interface and connected with the optimum mix (D). Testing was done such that the closure concrete was 28 days and the precast concrete sections were age 74 days at the time of testing. Data on each was collected up the point of failure. The ultimate force was 12,519 lb, the average cracking force was 5,820 lb, and the average force in the head of the rebar was 5,068 lb.

6.7 Flexural Beam Summary

Flexural beam tests were the final phase of the experimental work. Beams consisted of two precast segments connected with the closure pour Mix D using a non-contact splice connection. Utilizing two sets of beams, the samples were tested under three-point and four-point flexural bonding. The distributed loading represented the footprint of a truck tire. In all cases, the age of the closure pour concrete was 28 days, and the precast was 119-123 days. Stages of cracking included: primary, precast, shear, and secondary. Three-point and four-point specimens averaged 9,492 lb and 12,209 lb, respectively. The three-point specimens had an average moment capacity of 147,123 lb-in. as compared to the four-point specimens which were 146,504 lb-in. In addition, the three-point specimens performed higher on average rebar force at 8,251 lb compared to the four-point specimens at 7,496 lb.

6.8 Conclusion

In this research project it was determined that the material behavior and cost of HES concrete with polypropylene fibers and shrinkage reducing admixture was an effective alternative for field-cast connections of precast elements in accelerated bridge construction. This optimum HES concrete consisted of 1.5 lb/yd^3 of polypropylene fiber dosage, with shrinkage reducing admixture and no bonding agent added.

The advantages of the proposed HES concrete are the cost and time savings over UHPC. HES concrete can be batched in a ready mix plant, brought to the field in a mixing truck, and placed similarly to conventional concrete. UHPC is a more time consuming construction process with higher labor costs. Another time-saving advantage of HES concrete is that, ITD allows removal of forms after one day, while UHPC requires a minimum curing time of four days, and a minimum compressive strength of 14,000 psi before removal of the forms. The cost of using HES concrete is comparable to conventional concrete ($600-$700 per cubic yard). The estimated cost saving of using HES over UHPC can range from $50,000 to $100,000. Table 6.1 shows the comparison of material properties between the optimum HES concrete and UHPC. Material properties of the precast concrete are also included. Strength values of UHPC far exceed that of HES-D. Two important values are the interface bond strength and shrinkage. Bond strength for HES-D is comparable to that of UHPC. HES-D also has a lower long-term shrinkage.

Table 6.1 Material Comparison

	HES-D	UHPC[a]	Precast
Compressive Strength (ASTM C39), *psi*	8,864	24,000	5,041
Tensile Strength (ASTM C496), *psi*	837	1,300	600
Interface Bond Strength with Precast Concrete (ASTM C78)[b], *psi*	612	712[c]	-
Shrinkage (ASTM C157), *micro-strain*	522	555	-
Modulus of Elasticity (ASTM 469), *psi*	4,390,000	7,000,000	3,230,000
Poisson's Ratio (ASTM C469)	0.18	-	0.16
Material Cost	$200	$2,000	-

Note: HES-D = High early strength concrete Mix D; UHPC = Ultra High Performance Concrete.

[a]Average values from Table 1 of FHWA Publication No: FHWA-HRT-14-084 (Graybeal, B., 2014).

[b]28-days and precast concrete had exposed aggregate (EA) surface preparation.

[c]Value from De la Varga, Haber, and Graybeal, 2016.

6.9 Future Work

The experimental program of this thesis is being used to develop a Finite Element (FE) model of the closure pour detail. Strength and stiffness properties obtained from experimental work will be used to define the materials in the FE model. Results from the HB pullout and flexural beam tests will be used for calibration so that the response of the FE model is in-line with that of the experimental. After the calibration with the laboratory tests, the model will be utilized to determine the performance of a bridge using the proposed closure pour detail. This will give an indication of the adequacy of the connection.

The narrow scope of this study regarding the application of HES concrete as an alternative for UHPC in concrete bridge connections requires further investigation. These closure pour connections need to be applied in real world situations to see the long-term effects of their durability and performance. Instrumentation of a bridge then becomes a next critical step to validate these results and solve future problems. A similar closure pour material to HES-D is already being implemented into the SH-86 Bridge over Bear River, Idaho. Instrumentation of this bridge is part of a separate research project. The goal is to obtain long term performance of this mix as it is implemented in the field that this research did not address.

REFERENCES

Ahmed, S., Bukhari, I. A., Siddiqui, J. I., and Qureshi, S. A. (2006). "A study on properties of polypropylene fiber reinforced concrete." *31st Conference on Our World in Concrete & Structures*, Singapore.

AISC. (2017). *Manual of Steel Construction*, 15th Ed., AISC, Chicago, IL.

ASTM. (1999). "Standard test method for bond strength of latex systems used with concrete by slant shear (Withdrawn 2008)." *ASTM C1042-99*, West Conshohocken, PA.

ASTM. (2013a). "Standard specification for latex agents for bonding fresh to hardened concrete." *ASTM C1059-13*, West Conshohocken, PA.

ASTM. (2013b). "Standard test method for bond strength of epoxy-resin systems used with concrete by slant shear." *ASTM C882-13a*, West Conshohocken, PA.

ASTM. (2013c). "Standard test method for tensile strength of concrete surfaces and the bond strength or tensile strength of concrete repair and overlay materials by direct tension (Pull-off method)." *ASTM C1583-13*, West Conshohocken, PA.

ASTM. (2013d). "Standard test method for total evaporable moisture content of aggregate by drying." *ASTM C566-13*, West Conshohocken, PA.

ASTM. (2014a). "Standard practice for sampling aggregates." *ASTM D75-14*, West Conshohocken, PA.

ASTM. (2014b). "Standard test method for sieve analysis of fine ad coarse aggregates." *ASTM C136-14*, West Conshohocken, PA.

ASTM. (2014c). "Standard test method for static modulus of elasticity and Poisson's ratio of concrete in compression." *ASTM C469-14*, West Conshohocken, PA.

ASTM. (2015a). "Standard practice for capping cylindrical concrete specimens." *ASTM C617-15*, West Conshohocken, PA.

ASTM. (2015b). "Standard test method for relative density (specific gravity) and absorption of coarse aggregate." *ASTM C127-15*, West Conshohocken, PA.

ASTM. (2015c). "Standard test method for relative density (specific gravity) and absorption of fine aggregate." *ASTM C127-15*, West Conshohocken, PA.

ASTM. (2015d). "Standard test method for slump of hydraulic-cement concrete." *ASTM C143-15a*, West Conshohocken, PA.

ASTM. (2016). "Standard practice for making and curing concrete test specimens in the laboratory." *ASTM C192-16a*, West Conshohocken, PA.

ASTM. (2017a). “Standard test method for air content of freshly mixed concrete by the pressure method.” *ASTM C231-17a*, West Conshohocken, PA.

ASTM. (2017b). “Standard test method for compressive strength of cylindrical concrete specimens.” *ASTM C39-17b*, West Conshohocken, PA.

ASTM. (2017c). “Standard test method for length change of hardened hydraulic-cement mortar and concrete.” *ASTM C157-17*, West Conshohocken, PA.

ASTM. (2017d). “Standard test method for splitting tensile strength of cylindrical concrete specimens.” *ASTM C496-17*, West Conshohocken, PA.

ASTM. (2018). “Standard test method for flexural strength of concrete (Using simple beam with third-point loading).” *ASTM C78-18*, West Conshohocken, PA.

Banthia, N., and Gupta, R. (2006). “Influence of polypropylene fiber geometry on plastic shrinkage cracking in concrete.” *Cement and Concrete Research*, 36(7), 1263-1267.

Bentz, D., De la Varga, I., Muñoz, J., Spragg, R., Graybeal, B., Hussey, D., Jacobson, D., Jones, S., and LaManna, J. (2017). “Interface of substrate moisture state and roughness on interface microstructure and bond strength: slant shear vs. pull-off testing.” *Cement and Concrete Composites*, 87, 63-72.

Culmo, M. P. (2011). “Accelerated bridge construction - Experience in design, fabrication and erection of prefabricated bridge elements and systems.” *FHWA-HIF-12-013*, FHWA, McLean, VA.

De la Varga, I., and Graybeal, B. (2016). “Dimensional stability of grout-type materials used as connections for prefabricated bridge elements.” *FHWA-HRT-16-008*, FHWA, McLean, VA.

De la Varga, I., Haber, Z. B., and Graybeal, B. (2016). “Performance of grouted connections for prefabricated bridge elements-part I: material-level investigation on shrinkage and bond.” *2016 PCI National Bridge Conference*, PCI, Nashville, TN.

De la Varga, I., Haber, Z. B., and Graybeal, B. (2017). “Bond of Field-Cast Grouts to Precast Concrete Elements.” *FHWA-HRT-16-081*, FHWA, McLean, VA.

De la Varga, I., Muñoz, J., Bentz, D. P., Spragg, R. P., Stutzman, P. E., and Graybeal, B. A. (2018). “Grout-concrete interface bond performance: Effect of interface moisture on the tensile bond strength and grout microstructure.” *Construction and Building Materials*, 170, 747-756.

Emmons, P. H. (1994). *Concrete Repair and Maintenance Illustrated*, RS Meas Company Inc., Kingston, MA.

Federal Highway Administration (2018). *National Bridge Inventory (NBI).* https://www.fhwa.dot.gov/bridge/nbi.cfm

Graybeal, B. (2014). “Design and construction of field-cast UHPC connections.” *FHWA-HRT-14-084*, FHWA, McLean, VA.

Haber, Z. B., De la Varga, I., and Graybeal, B. (2016). “Performance of grouted connections for prefabricated bridge elements-part II: component-level investigation on bond and cracking.” *2016 PCI National Bridge Conference*, PCI,Nashville, TN.

Haber, Z. B., and Graybeal, B. A. (2016). “Performance of multiple UHPC-class materials in prefabricated bridge deck connections.” *First International Interactive Symposium on UHPC*, Des Moines, IA.

Hoomes, L. C, Ozyildrim, H. C., and Brown, M. C. (2017). “Evaluation of high-performace fiber-reinforced concrete for bridge deck connections, closure pours, and joints.” *FHWA/VTRC 17-R15*, VTRC, Charlottesville, VA.

Idaho Transportation Department (2012). *Standard Specifications for Highway Construction*, Boise, ID.

Julio, E. S., Branco, F. B., and Silva, V. D. (2004). “Concrete-to-concrete bond strength. Influence of the roughness of the substrate surface.” *Construction and Building Materials*, 18, 675-681.

Julio, E. S., Branco, F. B., and Silva, V. D. (2005). “Concrete-to-concrete bond strength: Influence of an epoxy-based bonding agent on a roughened substrate surface.” *Magazing of Concrete Research*, 57(8), 463-468.

Kakooei, S., Akil, H. M., Jamshidi, M., and Rouhi, J. (2012). “The effects of polypropylene fibers on the properties of reinforced concrete structures.” *Construction and Building Materials*, 27(1), 73-77.

Madhavi, T. C., Raju, L. S., and Mathur, D. (2014). “Polypropylene fiber reinforced concrete - A review.” *International Journal of Emerging Technology and Advanced Engineering*, 4(4) 114-119.

Mamlouk, M. S., and Zaniewski, J. P. (2011). *Materials for civil and construction engineers,* 3rd Ed., Prentice Hall, Upper Saddle River, NJ.

Patel, P. A., Desai, A. K., and Desai, J. A. (2012). “Evaluation of engineering properties for polypropylene fibre reinforced concrete.” *International Journal of Advanced Engineering Technology, 3*(1), 42-45.

Santos, P. M. D., and Julio, E. N. B. S. (2011). “Factors affecting bond between new and old concrete.” *ACI Materials Journal*, 108(4), 449-456.

Silfwerbrand, J. (2003). “Shear bond strength in repaired concrete structures.” *Materials and Structures*, 36, 419-424.

Serdar, M., Baričević, A., Rukavina, M. J., Pezer, M., Bjegović, D., and Štirmer, N. (2015). "Shrinkage behaviour of fibre reinforced concrete with recycled tyre polymer fibres." *International Journal of Polymer Science*, 2015.

Swenty, M. K., and Graybeal, B. A. (2013). "Material characterization of field-cast connection grouts." *FHWA-HRT-13-041*, FHWA, McLean, VA.

Tayeh, B. A., Abu Bakar, B. H., and Megat Johari, M. A. (2013). "Characterization of the interfacial bond between old concrete substrate and ultra high performance fiber concrete repair composite." *Materials and Structures*, 46, 743-753.

Yildirim, G., Sahmaran, M., Al-Emam, M. K. M., Hameed, R. K. H., Al-Najjar, Y., and Lachemi, M. (2015). "Effects of compressive strength, autogenous shrinkage and testing methods on the bond behavior of HES-ECC." *ACI Materials Journal*, 112 (3), 409-418.

APPENDICES

Appendix A Material Data Sheets

ASH GROVE

ASH GROVE CEMENT COMPANY
WESTERN REGION
33060 SHIRTTAIL CREEK ROAD
P.O. BOX 287
DURKEE, OREGON 97905
(541) 877-2411

Durkee Plant
Mill Test Report

Mill Analysis No. 17-5 | Cement Type II-V L.A. | Date 03-07-2017
Bin No. 2,3,4,D | Production Period February 1 thru February 28, 2017

STANDARD REQUIREMENTS
ASTM C150

CHEMICAL

Item	Spec. Limit	Test Result
SiO_2 (%)	A	22.3
Al_2O_3(%)	6.0 max.	3.4
Fe_2O_3(%)	6.0 max.	3.1
CaO (%)	A	64.3
MgO (%)	6.0 max.	2.3
SO_3 (%)	D	2.5
Loss On Ignition (%)	3.0 max.	0.88
Na_2O (%)	A	0.22
K_2O (%)	A	0.47
TiO_2 (%)	A	0.25
P_2O_5 (%)	A	0.12
Mn_2O_3 (%)	A	0.06
Insoluble Residue (%)	0.75 max.	0.34
CO_2 (%)	A	0.36
Limestone %	5.0 max.	0.89
$CaCO_3$ in Limestone	70 min.	91.85
$C_3S + 4.75C_3A$	100 max.	76
Potential Compounds (%)	C	
C_3S	A	57
C_2S	A	21
C_3A	5 max.	4
C_4AF	A	9
$C_4AF+2(C_3A)$	25 max.	17

PHYSICAL

Item		Spec. Limit	Test Result
Air Content of Mortar (volume %)			
C185		12 max.	6.2
Fineness (m^2/kg)			
C204 (Air permeability)		260 min.	375
Autoclave Expansion (%)		0.80 max.	0.05
C151			
Compressive Strength Psi (Mpa)		Min.:	
C109	1 Day	A	2120 (14.6)
	3 Days	1450 (10.0)	3920 (27.0)
	7 Days	2470 (17.0)	5160 (35.6)
	28 Days	3050 (21.0)	F
Time of Setting (minutes)			
C191 (Vicat)			
Initial	Not less than	45	126
	Not more than	375	

OPTIONAL REQUIREMENTS
ASTM C150, (other)

CHEMICAL

Item	Spec. Limit	Test Result
Equivalent Alkalies (%)	0.60 max.	0.52
Chloride (%)	B	0.002

PHYSICAL

Item		Spec. Limit	Test Result
Time of Set - Final (minutes)	C191	B	205
False Set (%) C451		50 min.	92
Heat of Hydration (cal /g)	C186		
	7 days	B	81
Sulfate Resistance (%)	C452	0.040	0.025
Water Expasion (%)	C1038	0.020	0.016
% retain on 45μm sieve		B	1.22

A = Not applicable
B = Test results represents most recent value and is provided for informational purposes only.
C = Adjusted per A 1.6.
D = C1038 expansion in water does not exceed 0.02% at 14 days.
F = Test results for this production period not yet available.

We certify that the above described cement, at the time of shipment, meets the chemical and physical requirement of the ASTM C150-16 or AASHTO M-85 -12 Type I-II-V specification also will meet CSA A3000-13 Type GU, MS and HS.

NSF
Certified to NSF/ANSI 61

Signature: Christopher W. Patterson

Title: Chief Chemist

Figure A.1 Portland Cement Data Sheet

HEADWATERS RESOURCES

Materials Testing & Research Facility
2650 Old State Hwy 113
Taylorsville, GA 30178
770-684-0102

ASTM C618 / AASHTO M295 Testing of Navajo Generating Station Fly Ash

Sample Date:	2/10 - 2/13/17	**Report Date:**	3/24/2017
Sample Type:	3200-ton	**MTRF ID:**	360NV
Sample ID:	NV-014-17		

Chemical Analysis	**Results**	**ASTM Limit Class F/C**	**AASHTO Limit Class F/C**
Silicon Dioxide (SiO2)	58.96 %		
Aluminum Oxide (Al2O3)	25.10 %		
Iron Oxide (Fe2O3)	4.31 %		
Sum (SiO2+Al2O3+Fe2O3)	88.37 %	70.0/50.0 min	70.0/50.0 min
Sulfur Trioxide (SO3)	0.36 %	5.0 max	5.0 max
Calcium Oxide (CaO)	4.74 %		
Magnesium Oxide (MgO)	1.12 %		
Sodium Oxide (Na2O)	2.12 %		
Potassium Oxide (K2O)	1.21 %		
Moisture	0.06 %	3.0 max	3.0 max
Loss on Ignition	0.57 %	6.0 max	5.0 max
Available Alkalies, as Na2Oe	1.27 %	Not Required	1.5 max*

*when required by purchaser

Physical Analysis			
Fineness, % retained on 45-μm sieve	21.26 %	34 max	34 max
Strength Activity Index - 7 or 28 day requirement			
7 day, % of control	80 %	75 min	75 min
28 day, % of control	83 %	75 min	75 min
Water Requirement, % control	96 %	105 max	105 max
Autoclave Soundness	0.03 %	0.8 max	0.8 max
Density	2.26		

Headwaters Resources certifies that pursuant to current ASTM C618 protocol for testing, the test data listed herein was generated by applicable ASTM methods and meets the requirements of ASTM C618.

Doug Rhodes, CET
Facility Manager

AAP SM
AASHTO R18

Figure A.2 Fly Ash Data Sheet

BASF
We create chemistry

3 | 03 30 00 Cast-in-Place Concrete
03 40 00 Precast Concrete

MasterSet® AC 534

Accelerating Admixture

Formerly Pozzolith NC 534*

Description

MasterSet AC 534 patented, ready-to-use, liquid admixture is formulated to accelerate time of setting and to increase early concrete strengths. MasterSet AC 534 admixture does not contain calcium chloride and is formulated to comply with ASTM C 494/C 494M Type C, accelerating, admixture requirements.

Applications

Recommended for use in:

- Reinforced, precast, pumped, flowable, lightweight or normal weight concrete and shotcrete (wet mix)
- Concrete placed on galvanized steel floor and roof systems which are left in place
- Prestressed concrete
- Fast-track concrete construction
- Concrete subject to chloride ion constraints
- 4x4™ Concrete
- Self-consolidating concrete (SCC)
- Pervious concrete

Features

- Accelerated setting time across a wide range of temperatures
- Increased early compressive and flexural strengths

Benefits

- Earlier finishing of slabs — reduced labor costs
- Reduced in-place concrete costs
- Reduced or eliminated heating and protection time in cold weather
- Earlier stripping and reuse of forms
- Superior finishing characteristics for flatwork and cast surfaces

Performance Characteristics

Mixture Data: 453 lb/yd^3 (269 kg/m^3) of Type I cement; 3-4 in. (75-100 mm) slump; concrete temperature 74 °F (23 °C); ambient temperature 50 and 75 °F (10 and 24 °C); non-air-entrained concrete.

Setting time	@ 50 °F (10 °C)	
	Initial Set (h:min)	Difference (h:min))
Plain	13:44	REF
MasterSet AC 534 admixture @		
› 20 fl oz/cwt (1,300 mL/100 kg)	7:11	- 6:33
› 40 fl oz/cwt (2,600 mL/100 kg)	6:05	- 7:39

Setting time	@ 75 °F (24 °C)	
	Initial Set (h:min)	Difference (h:min))
Plain	8:18	REF
MasterSet AC 534 admixture @		
› 20 fl oz/cwt (1,300 mL/100 kg)	4:59	- 3:19
› 40 fl oz/cwt (2,600 mL/100 kg)	4:18	- 4:00

MASTER® BUILDERS SOLUTIONS

Figure A.3 Master Set Data Sheet (a)

MasterSet AC 534 Technical Data Sheet

Guidelines for Use

Dosage: The recommended dosage range for MasterSet AC 534 admixture is 10-45 fl oz/cwt (0.65 – 2.9 L/100 kg) of cementitious materials for most concrete mixtures using average concrete ingredients. Because of variations in job conditions and concrete materials, dosage rates other than the recommended amounts may be required. In such cases, contact your local sales representative.

The maximum dosage of MasterSet AC 534 in potable water applications that require the use of NSF Certified products is 30 fl oz/cwt (2.0 L/kg) of cementitious materials. For specialty concrete mixtures such as 4x4 Concrete, dosages up to 100 fl oz/cwt (6.5 L/100 kg) may be required.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: MasterSet AC 534 admixture will neither initiate nor promote corrosion of reinforcing steel in concrete.

Compatibility: MasterSet AC 534 admixture may be used in combination with any BASF admixtures. When used in conjunction with other admixtures, each admixture must be dispensed separately into the mixture.

Storage and Handling

Storage Temperature: MasterSet AC 534 admixture should be stored above freezing temperatures. If MasterSet AC 534 admixture freezes, thaw at 35 °F (2 °C) or above and completely reconstitute by mild mechanical agitation. **Do not use pressurized air for agitation.**

Shelf Life: MasterSet AC 534 admixture has a minimum shelf life of 18 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your local sales representative regarding suitability for use and dosage recommendations if the shelf life of MasterSet AC 534 admixture has been exceeded.

Packaging

This product is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Safety Data Sheets: MasterSet AC 534 admixture

BASF Corporation
Admixture Systems
www.master-builders-solutions.basf.us

page 2 of 3

Figure A.4 Master Set Data Sheet (b)

Additional Information

For additional information on MasterSet AC 534 admixture or its use in developing a concrete mixture with special performance characteristics, contact your local sales representative.

The Admixture Systems business of BASF's Construction Chemicals division is the leading provider of solutions that improve placement, pumping, finishing, appearance and performance characteristics of specialty concrete used in the ready-mixed, precast, manufactured concrete products, underground construction and paving markets. For over 100 years we have offered reliable products and innovative technologies, and through the Master Builders Solutions brand, we are connected globally with experts from many fields to provide sustainable solutions for the construction industry.

Limited Warranty Notice

BASF warrants this product to be free from manufacturing defects and to meet the technical properties on the current Technical Data Guide, if used as directed within shelf life. Satisfactory results depend not only on quality products but also upon many factors beyond our control. BASF MAKES NO OTHER WARRANTY OR GUARANTEE, EXPRESS OR IMPLIED, INCLUDING WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE WITH RESPECT TO ITS PRODUCTS. The sole and exclusive remedy of Purchaser for any claim concerning this product, including but not limited to, claims alleging breach of warranty, negligence, strict liability or otherwise, is shipment to purchaser of product equal to the amount of product that fails to meet this warranty or refund of the original purchase price of product that fails to meet this warranty, at the sole option of BASF. Any claims concerning this product must be received in writing within one (1) year from the date of shipment and any claims not presented within that period are waived by Purchaser. BASF WILL NOT BE RESPONSIBLE FOR ANY SPECIAL, INCIDENTAL, CONSEQUENTIAL (INCLUDING LOST PROFITS) OR PUNITIVE DAMAGES OF ANY KIND.

Purchaser must determine the suitability of the products for the intended use and assumes all risks and liabilities in connection therewith. This information and all further technical advice are based on BASF's present knowledge and experience. However, BASF assumes no liability for providing such information and advice including the extent to which such information and advice may relate to existing third party intellectual property rights, especially patent rights, nor shall any legal relationship be created by or arise from the provision of such information and advice. BASF reserves the right to make any changes according to technological progress or further developments. The Purchaser of the Product(s) must test the product(s) for suitability for the intended application and purpose before proceeding with a full application of the product(s). Performance of the product described herein should be verified by testing and carried out by qualified experts.

* Pozzolith NC 534 became MasterSet AC 534 under the Master Builders Solutions brand, effective January 1, 2014.

Certified to NSF/ANSI 61

BASF Corporation
Admixture Systems
www.master-builders-solutions.basf.us

United States
23700 Chagrin Boulevard
Cleveland, Ohio 44122-5544
Tel: 800 628-9990 ■ Fax: 216 839-8821

Canada
1800 Clark Boulevard
Brampton, Ontario L6T 4M7
Tel: 800 387-5862 ■ Fax: 905 792-0651

Figure A.5 Master Set Data Sheet (c)

BASF
We create chemistry

3	03 30 00	Cast-in-Place Concrete
	03 40 00	Precast Concrete
	03 70 00	Mass Concrete

MasterAir® AE 200

Air-Entraining Admixture

Formerly Micro Air*

Description

MasterAir AE 200 air-entraining admixture provides concrete with extra protection by creating air bubbles that are ultrastable, small and closely spaced – a characteristic especially useful in the types of concrete known for their difficulty to entrain and maintain the air content desired.

Even when used at a lower dosage than standard air-entraining admixtures, MasterAir AE 200 admixture meets the requirements of ASTM C 260, AASHTO M 154, and CRD-C 13.

Applications

Recommended for use in:

- Concrete exposed to cyclic freezing and thawing
- Production of high-quality normal or lightweight concrete (heavyweight concrete normally does not contain entrained air)

Features

- Ready-to-use in the proper concentration for rapid, accurate dispensing
- Greatly improved stability of air-entrainment
- Ultra stable air bubbles

Benefits

- Increased resistance to damage from cyclic freezing and thawing
- Increased resistance to scaling from deicing salts
- Improved plasticity and workability
- Improved air-void system in hardened concrete
- Improved ability to entrain and retain air in low-slump concrete, concrete containing high-carbon content fly ash, concrete using large amounts of fine materials, concrete using high-alkali cements, high-temperature concrete, and concrete with extended mixing times
- Reduced permeability – increased watertightness
- Reduced segregation and bleeding

Performance Characteristics

Concrete durability research has established that the best protection for concrete from the adverse effects of freezing and thawing cycles and deicing salts results from: proper air content in the hardened concrete, a suitable air-void system in terms of bubble size and spacing and adequate concrete strength, assuming the use of sound aggregates and proper mixing, transporting, placing, consolidation, finishing and curing techniques. MasterAir AE 200 admixture can be used to obtain adequate freezing and thawing durability in a properly proportioned concrete mixture, if standard industry practices are followed.

MASTER®
» BUILDERS
SOLUTIONS

Figure A.6 Master Air Data Sheet (a)

Air Content Determination: The total air content of normal weight concrete should be measured in strict accordance with ASTM C 231, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method" or ASTM C 173/C 173M, "Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method."

The air content of lightweight concrete should only be determined using the Volumetric Method. The air content should be verified by calculating the gravimetric air content in accordance with ASTM C 138/C 138M, "Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete." If the total air content, as measured by the Pressure Method or Volumetric Method and as verified by the Gravimetric Method, deviates by more than 1.5%, the cause should be determined and corrected through equipment calibration or by whatever process is deemed necessary.

Guidelines for Use

Dosage: There is no standard dosage for MasterAir AE 200 admixture. The exact quantity of air-entraining admixture needed for a given air content of concrete varies because of differences in concrete making materials and ambient conditions. Typical factors that might influence the amount of air entrained include: temperature, cementitious materials, sand gradation, sand-aggregate ratio, mixture proportions, slump, means of conveying and placement, consolidation and finishing technique.

The amount of MasterAir AE 200 admixture used will depend upon the amount of entrained air required under actual job conditions. In a trial mixture, use 0.125 to 1.5 fl oz/cwt (8-98 mL/100 kg) of cement. In mixtures containing water-reducing or set-control admixtures, the amount of MasterAir AE 200 admixture needed is somewhat less than the amount required in plain concrete. Due to possible changes in the factors that can affect the dosage of MasterAir AE 200 admixture, frequent air content checks should be made during the course of the work. Adjustments to the dosage should be based on the amount of entrained air required in the mixture at the point of placement. If an unusually high or low dosage of MasterAir AE 200 admixture is required to obtain the desired air content, consult your Local sales representative. In such cases, it may be necessary to determine that, in addition to a proper air content in the fresh concrete, a suitable air-void system is achieved in the hardened concrete.

Dispensing and Mixing: Add MasterAir AE 200 admixture to the concrete mixture using a dispenser designed for air-entraining admixtures; or add manually using a suitable measuring device that ensures accuracy within plus or minus 3% of the required amount. For optimum, consistent performance, the air-entraining admixture should be dispensed on damp, fine aggregate or with the initial batch water. If the concrete mixture contains lightweight aggregate, field evaluations should be conducted to determine the best method to dispense the air-entraining admixture.

Precaution

In a 2005 publication from the Portland Cement Association (PCA R&D Serial No. 2789), it was reported that problematic air-void clustering that can potentially lead to above normal decreases in strength was found to coincide with late additions of water to air-entrained concretes. Late additions of water include the conventional practice of holding back water during batching for addition at the jobsite. Therefore, caution should be exercised with delayed additions to air-entrained concrete. Furthermore, an air content check should be performed after post-batching addition of any other materials to an air-entrained concrete mixture.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: MasterAir AE 200 admixture will neither initiate nor promote corrosion of reinforcing and prestressing steel embedded in concrete, or of galvanized steel floor and roof systems. No calcium chloride or other chloride-based ingredients are used in the manufacture of this admixture.

Compatibility: MasterAir AE 200 admixture may be used in combination with any BASF admixture, unless stated otherwise on the data sheet for the other product. When used in conjunction with other admixtures, each admixture must be dispensed separately into the mixture.

Figure A.7 Master Air Data Sheet (b)

MasterAir AE 200 Technical Data Sheet

Storage and Handling

Storage Temperature: MasterAir AE 200 admixture should be stored and dispensed at 35 °F (2 °C) or higher. Although freezing does not harm this product, precautions should be taken to protect it from freezing. If it freezes, thaw and reconstitute by mild mechanical agitation. Do not use pressurized air for agitation.

Shelf Life: MasterAir AE 200 admixture has a minimum shelf life of 18 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your Local sales representative regarding suitability for use and dosage recommendations if the shelf life of MasterAir AE 200 admixture has been exceeded.

Safety: MasterAir AE 200 admixture is a caustic solution. Chemical goggles and gloves are recommended when transferring or handling this material. (See SDS and/or product label for complete information.)

Packaging

MasterAir AE 200 admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Safety Data Sheets: MasterAir AE 200 admixture

Additional Information

For suggested specification information or for additional product data on MasterAir AE 200 admixture, contact your local sales representative.

The Admixture Systems business of BASF's Construction Chemicals division is the leading provider of solutions that improve placement, pumping, finishing, appearance and performance characteristics of specialty concrete used in the ready-mixed, precast, manufactured concrete products, underground construction and paving markets. For over 100 years we have offered reliable products and innovative technologies, and through the Master Builders Solutions brand, we are connected globally with experts from many fields to provide sustainable solutions for the construction industry.

Limited Warranty Notice

BASF warrants this product to be free from manufacturing defects and to meet the technical properties on the current Technical Data Guide, if used as directed within shelf life. Satisfactory results depend not only on quality products but also upon many factors beyond our control. BASF MAKES NO OTHER WARRANTY OR GUARANTEE, EXPRESS OR IMPLIED, INCLUDING WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE WITH RESPECT TO ITS PRODUCTS. The sole and exclusive remedy of Purchaser for any claim concerning this product, including but not limited to, claims alleging breach of warranty, negligence, strict liability or otherwise, is shipment to purchaser of product equal to the amount of product that fails to meet this warranty or refund of the original purchase price of product that fails to meet this warranty, at the sole option of BASF. Any claims concerning this product must be received in writing within one (1) year from the date of shipment and any claims not presented within that period are waived by Purchaser. BASF WILL NOT BE RESPONSIBLE FOR ANY SPECIAL, INCIDENTAL, CONSEQUENTIAL (INCLUDING LOST PROFITS) OR PUNITIVE DAMAGES OF ANY KIND.

Purchaser must determine the suitability of the products for the intended use and assumes all risks and liabilities in connection therewith. This information and all further technical advice are based on BASF's present knowledge and experience. However, BASF assumes no liability for providing such information and advice including the extent to which such information and advice may relate to existing third party intellectual property rights, especially patent rights, nor shall any legal relationship be created by or arise from the provision of such information and advice. BASF reserves the right to make any changes according to technological progress or further developments. The Purchaser of the Product(s) must test the product(s) for suitability for the intended application and purpose before proceeding with a full application of the product(s). Performance of the product described herein should be verified by testing and carried out by qualified experts.

* Micro Air became MasterAir AE 200 under the Master Builders Solutions brand, effective January 1, 2014.

Certified to NSF/ANSI 61

BASF Corporation
Admixture Systems
www.master-builders-solutions.basf.us

United States
23700 Chagrin Boulevard
Cleveland, Ohio 44122-5544
Tel: 800 628-9990 ■ Fax: 216 839-8821

Canada
1800 Clark Boulevard
Brampton, Ontario L6T 4M7
Tel: 800 387-5862 ■ Fax: 905 792-0651

Figure A.8 Master Air Data Sheet (c)

BASF
We create chemistry

3	03 30 00	Cast-in-Place Concrete
	03 40 00	Precast Concrete
	03 70 00	Mass Concrete

MasterGlenium® 1466
High-Range Water-Reducing Admixture
Formerly PS1466*

Description
MasterGlenium 1466 ready-to-use high-range water-reducing admixture is a new generation, patent pending admixture based on polycarboxylate chemistry. MasterGlenium 1466 admixture is very effective in producing concretes with different levels of workability.

MasterGlenium 1466 admixture is particularly effective in improving concrete mixtures with reduced portland cement contents without compromising 28-day strength requirements. MasterGlenium 1466 admixture meets ASTM C 494/C 494M requirements for Type A, water-reducing, and Type F, high-range water-reducing, admixtures.

Applications
Recommended for use in:
- Concrete with varying water reduction requirements (5-40%)
- Concrete where high flowability, increased stability and durability are needed
- Producing self-consolidating concrete (SCC)
- Strength-on-demand concrete, such as 4x4™ Concrete
- Pervious concrete

Features
- Maximum dosage effectiveness for a given water reduction
- Controlled rheology
- Robust air-entraining admixture compatibility
- Improved strength development

Benefits
- Can be used in a wide variety of concrete mixtures as a Type A or Type F admixture
- Improved finishability and surface appearance
- Mixture development flexibility for cement reductions and/or increased use of supplementary cementitious materials

Performance Characteristics
Compressive Strength: Concrete produced with MasterGlenium 1466 admixture achieves significantly higher 28-day compressive strength compared to plain concrete and concrete mixtures containing naphthalene, melamine, and early generation polycarboxylate high-range water-reducing admixtures.

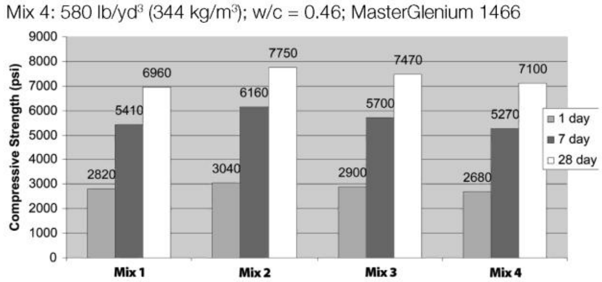
Mixture Data: Type I portland cement; Ambient Temperature, 70 °F (21 °C)

Mix 1: 620 lb/yd^3 (367 kg/m^3); w/c = 0.43; Conventional PC HRWR

Mix 2: 620 lb/yd^3 (367 kg/m^3); w/c = 0.43; MasterGlenium 1466

Mix 3: 600 lb/yd^3 (356 kg/m^3); w/c = 0.44; MasterGlenium 1466

Mix 4: 580 lb/yd^3 (344 kg/m^3); w/c = 0.46; MasterGlenium 1466



MASTER® BUILDERS SOLUTIONS

Figure A.9 Master Glenium Data Sheet (a)

Guidelines for Use

Dosage: MasterGlenium 1466 admixture has a recommended dosage range of 2-10 fl oz/cwt (130-650 mL/100 kg) of cementitious materials. For most applications, dosages in the range of 2-6 fl oz/cwt (130-390 mL/100 kg) will provide excellent performance. Because of variations in concrete materials, job site conditions and/or applications, dosages outside of the recommended range may be required. In such cases, contact your local sales representative.

Mixing: MasterGlenium 1466 admixture can be added with the initial batch water or as a delayed addition. However, optimum water reduction is generally obtained with a delayed addition.

Product Notes

Corrosivity – Non-Chloride, Non-Corrosive: MasterGlenium 1466 admixture will neither initiate nor promote corrosion of reinforcing steel embedded in concrete, prestressing steel or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of MasterGlenium 1466 admixture.

Compatibility: MasterGlenium 1466 admixture is compatible with most admixtures used in the production of quality concrete, including normal, mid-range and high-range water-reducing admixtures, air-entrainers, accelerators, retarders, extended set control admixtures, corrosion inhibitors, and shrinkage reducers.

Do not use MasterGlenium 1466 admixture with admixtures containing naphthalene sulfonate. Erratic behaviors in slump, workability retention and pumpability may be experienced.

Storage and Handling

Storage Temperature: MasterGlenium 1466 admixture must be stored at temperatures above 40 °F (5 °C). If MasterGlenium 1466 admixture freezes, thaw and reconstitute by mechanical agitation. **Do not use pressurized air for agitation.**

Shelf Life: MasterGlenium 1466 admixture has a minimum shelf life of 6 months. Depending on storage conditions, shelf life may be greater than standard. Please contact your local sales representative regarding suitability for use and dosage recommendations if the shelf life of MasterGlenium 1466 admixture has been exceeded.

Packaging

MasterGlenium 1466 admixture is supplied in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Related Documents

Safety Data Sheets: MasterGlenium 1466 admixture

Additional Information

For additional information on MasterGlenium 1466 admixture or its use in developing concrete mixtures with special performance characteristics, contact your local sales representative.

The Admixture Systems business of BASF's Construction Chemicals division is the leading provider of solutions that improve placement, pumping, finishing, appearance and performance characteristics of specialty concrete used in the ready-mixed, precast, manufactured concrete products, underground construction and paving markets. For over 100 years we have offered reliable products and innovative technologies, and through the Master Builders Solutions brand, we are connected globally with experts from many fields to provide sustainable solutions for the construction industry.

Figure A.10 Master Glenium Data Sheet (b)

Limited Warranty Notice

BASF warrants this product to be free from manufacturing defects and to meet the technical properties on the current Technical Data Guide, if used as directed within shelf life. Satisfactory results depend not only on quality products but also upon many factors beyond our control. BASF MAKES NO OTHER WARRANTY OR GUARANTEE, EXPRESS OR IMPLIED, INCLUDING WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE WITH RESPECT TO ITS PRODUCTS. The sole and exclusive remedy of Purchaser for any claim concerning this product, including but not limited to, claims alleging breach of warranty, negligence, strict liability or otherwise, is shipment to purchaser of product equal to the amount of product that fails to meet this warranty or refund of the original purchase price of product that fails to meet this warranty, at the sole option of BASF. Any claims concerning this product must be received in writing within one (1) year from the date of shipment and any claims not presented within that period are waived by Purchaser. BASF WILL NOT BE RESPONSIBLE FOR ANY SPECIAL, INCIDENTAL, CONSEQUENTIAL (INCLUDING LOST PROFITS) OR PUNITIVE DAMAGES OF ANY KIND.

Purchaser must determine the suitability of the products for the intended use and assumes all risks and liabilities in connection therewith. This information and all further technical advice are based on BASF's present knowledge and experience. However, BASF assumes no liability for providing such information and advice including the extent to which such information and advice may relate to existing third party intellectual property rights, especially patent rights, nor shall any legal relationship be created by or arise from the provision of such information and advice. BASF reserves the right to make any changes according to technological progress or further developments. The Purchaser of the Product(s) must test the product(s) for suitability for the intended application and purpose before proceeding with a full application of the product(s). Performance of the product described herein should be verified by testing and carried out by qualified experts.

*PS 1466 became MasterGlenium 1466 under the Master Builders Solutions brand, effective May 11, 2016.

BASF Corporation
Admixture Systems
www.master-builders-solutions.basf.us

United States
23700 Chagrin Boulevard
Cleveland, Ohio 44122-5544
Tel: 800 628-9990 ■ Fax: 216 839-8821

Canada
1800 Clark Boulevard
Brampton, Ontario L6T 4M7
Tel: 800 387-5862 ■ Fax: 905 792-0651

Figure A.11 Master Glenium Data Sheet (c)

BASF
We create chemistry

3	03 30 00	Cast-in-Place Concrete
	03 40 00	Precast Concrete
	03 70 00	Mass Concrete

MasterLife® SRA 035

Shrinkage-Reducing Admixture

Description

MasterLife SRA 035 shrinkage-reducing admixture was developed specifically to reduce drying shrinkage of concrete and mortar, and the potential for subsequent cracking. MasterLife SRA 035 admixture functions by reducing capillary tension of pore water, a primary cause of drying shrinkage. MasterLife SRA 035 admixture will meet ASTM C 494/C 494M requirements for Type S, Specific Performance, admixtures.

Applications

Recommended for use in:

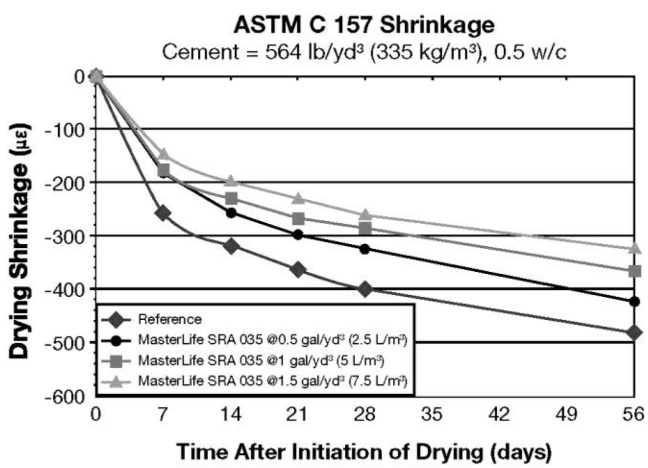
- Ready-mixed or precast concrete structures requiring shrinkage reduction and long-term durability
- Wet mix shotcrete

Features

- Reduces the capillary tension of pore water in cementitious mixtures
- Provides moderate to significant reductions in the drying shrinkage of cementitious mixtures
- Reduces stresses induced from one-dimensional surface drying in concrete slabs, walls and other elements

Benefits

- Reduces microcracking and drying shrinkage cracking in concrete, mortar and paste
- Minimizes curling in concrete slabs
- Improves aesthetics, watertightness and durability in concrete elements and structures
- Minimizes prestress loss in prestressed concrete applications



MASTER® BUILDERS SOLUTIONS

Figure A.12 Master Life Data Sheet (a)

Performance Characteristics

MasterLife SRA 035 admixture does not substantially affect slump. MasterLife SRA 035 admixture may slightly increase bleed time and bleed ratio. MasterLife SRA 035 admixture may also delay time of set by 1-2 hours depending upon dosage and temperature. Compressive strength loss is minimal with MasterLife SRA 035 admixture. For air-entrained concrete applications, truck trial evaluations as detailed in the section titled "Compatibility" must be performed to verify that the specified air content can be achieved consistently. Therefore, contact your local sales representative when concrete treated with MasterLife SRA 035 admixture is being proposed for applications exposed to freezing and thawing environments.

Guidelines for Use

Dosage: Knowledge of the shrinkage characteristics of the concrete mixture proposed for use is required prior to the addition of MasterLife SRA 035 admixture. The dosage of MasterLife SRA 035 admixture will be dependent on the desired drying shrinkage and the reduction in drying shrinkage required. Therefore, it is strongly recommended that drying shrinkage testing be performed to determine the optimum dosage for each application and each set of materials. The typical dosage range of MasterLife SRA 035 admixture is 0.5 to 1.5 gal/yd^3 (2.5 to 7.5 L/m^3). However, dosages outside of this range may be required depending on the level of shrinkage reduction needed for a given application and because of variations in concrete materials, jobsite conditions and other factors. In such cases, contact your local sales representative for further guidance.

Dispensing and Mixing: MasterLife SRA 035 admixture may be added to the concrete mixture during the initial batch sequence or at the jobsite. The mix water content should be reduced to account for the quantity of MasterLife SRA 035 admixture used. If the delayed addition method is used, mixing at high speed for 3-5 minutes after the addition of MasterLife SRA 035 admixture will result in mixture uniformity.

Product Notes

Corrosivity: Non-Chloride, Non-Corrosive: MasterLife SRA 035 admixture will neither initiate nor promote corrosion of reinforcing steel, prestressing steel or of galvanized steel floor and roof systems. Neither calcium chloride nor other chloride-based ingredients are used in the manufacture of MasterLife SRA 035 admixture.

Compatibility: MasterLife SRA 035 admixture is compatible with all air entrainers, water-reducers, mid-range water-reducers, high-range water reducers, set retarders, accelerators, silica fume, and corrosion inhibitors. For air-entrained concrete applications, MasterAir® AE 200 admixture is the preferred air entrainer. The dosage of air entrainer must be established through truck trial evaluations. The trials should include a simulated haul time of at least 20 minutes to assess air content stability. MasterLife SRA 035 admixture should be added separately to the concrete mixture to ensure desired results.

Storage and Handling

Storage Temperature: MasterLife SRA 035 admixture is a potentially combustible material with a flash point of 198 °F (92 °C). This is substantially above the upper limit of 140 °F (60 °C) for classification as a flammable material, and below the limit of 200 °F (93 °C) where DOT requirements would classify this as a combustible material. Nonetheless, this product must be treated with care and protected from excessive heat, open flame or sparks. For more information refer to the Safety Data Sheet. MasterLife SRA 035 admixture should be stored at ambient temperatures above 35 °F (2 °C), and precautions should be taken to protect the admixture from freezing. If MasterLife SRA 035 admixture freezes, thaw and reconstitute by mild mechanical agitation. Do not use pressurized air for agitation.

Shelf Life: MasterLife SRA 035 admixture has a minimum shelf life of 12 months. Depending on storage conditions, the shelf life may be greater than stated. Please contact your local sales representative regarding suitability for use and dosage recommendations if the shelf life of MasterLife SRA 035 admixture has been exceeded.

Packaging

MasterLife SRA 035 admixture is available in 55 gal (208 L) drums, 275 gal (1040 L) totes and by bulk delivery.

Figure A.13 Master Life Data Sheet (b)

MasterLife SRA 035 Technical Data Sheet

Related Documents

Safety Data Sheets: MasterLife SRA 035 admixture

Additional Information

For additional information on MasterLife SRA 035 admixture, or its use in developing a concrete mixture with special performance characteristics, contact your local sales representative.

The Admixture Systems business of BASF's Construction Chemicals division is the leading provider of solutions that improve placement, pumping, finishing, appearance and performance characteristics of specialty concrete used in the ready-mixed, precast, manufactured concrete products, underground construction and paving markets. For over 100 years we have offered reliable products and innovative technologies, and through the Master Builders Solutions brand, we are connected globally with experts from many fields to provide sustainable solutions for the construction industry.

Limited Warranty Notice

BASF warrants this product to be free from manufacturing defects and to meet the technical properties on the current Technical Data Guide, if used as directed within shelf life. Satisfactory results depend not only on quality products but also upon many factors beyond our control. BASF MAKES NO OTHER WARRANTY OR GUARANTEE, EXPRESS OR IMPLIED, INCLUDING WARRANTIES OF MERCHANTABILITY OR FITNESS FOR A PARTICULAR PURPOSE WITH RESPECT TO ITS PRODUCTS. The sole and exclusive remedy of Purchaser for any claim concerning this product, including but not limited to, claims alleging breach of warranty, negligence, strict liability or otherwise, is shipment to purchaser of product equal to the amount of product that fails to meet this warranty or refund of the original purchase price of product that fails to meet this warranty, at the sole option of BASF. Any claims concerning this product must be received in writing within one (1) year from the date of shipment and any claims not presented within that period are waived by Purchaser. BASF WILL NOT BE RESPONSIBLE FOR ANY SPECIAL, INCIDENTAL, CONSEQUENTIAL (INCLUDING LOST PROFITS) OR PUNITIVE DAMAGES OF ANY KIND.

Purchaser must determine the suitability of the products for the intended use and assumes all risks and liabilities in connection therewith. This information and all further technical advice are based on BASF's present knowledge and experience. However, BASF assumes no liability for providing such information and advice including the extent to which such information and advice may relate to existing third party intellectual property rights, especially patent rights, nor shall any legal relationship be created by or arise from the provision of such information and advice. BASF reserves the right to make any changes according to technological progress or further developments. The Purchaser of the Product(s) must test the product(s) for suitability for the intended application and purpose before proceeding with a full application of the product(s). Performance of the product described herein should be verified by testing and carried out by qualified experts.

BASF Corporation
Admixture Systems
www.master-builders-solutions.basf.us

United States
23700 Chagrin Boulevard
Cleveland, Ohio 44122-5544
Tel: 800 628-9990 ■ Fax: 216 839-8821

Canada
1800 Clark Boulevard
Brampton, Ontario L6T 4M7
Tel: 800 387-5862 ■ Fax: 905 792-0651

Figure A.14 Master Life Data Sheet (c)

The Euclid Chemical Company

AKKRO-7T

LIQUID BONDING ADMIXTURE

EUCLID CHEMICAL

BONDING AGENTS AND ADHESIVES

DESCRIPTION

AKKRO-7T is a non-redispersable, liquid bonding admixture used to produce polymer modified concrete and mortar. AKKRO-7T is a milky white, water based emulsion of high solids acrylic polymers and modifiers. AKKRO-7T is non-yellowing and has excellent resistance to ultraviolet degradation, heat and most common chemicals. AKKRO-7T does not alter the color of the mixture.

PRIMARY APPLICATIONS

- Cement based coatings, toppings, patching mortars, leveling compounds, stucco, and terrazzo
- Mixing liquid for TAMOSEAL, CONCRETE FINISHER, TAMMS SBC and TAMMS STUCCO FINISH

FEATURES/BENEFITS

- Increases flexural and tensile strength
- Improves bond strength
- Reduces shrinkage
- Resists water penetration
- Protects against freeze-thaw damage
- Non-yellowing

TECHNICAL INFORMATION

PROPERTY	UNMODIFIED SAND-CEMENT	SAND-CEMENT WITH AKKRO-7T
Compressive Strength, psi (MPa) ASTM C 109	3 days: 1,640 (11.3) 7 days: 2,345 (16.2)	3 days: 2,890 (19.9) 7 days: 3,410 (23.5)
Flexural Strength, 7 days, psi (MPa) ASTM C 78	410 (2.8)	1,096 (7.6)
Bond Strength, 7 days, psi (MPa) ASTM C 321	56 (0.4)	486 (3.3)
Tensile Strength, 7 days, psi (MPa) ASTM C 190	246 (1.7)	510 (3.5)

AKKRO-7T

PROPERTY OF AKKRO-7T	VALUE
Unit Weight	8.65 lb/gal
VOC Content	< 10 g/L
Specific Gravity	1.04

Properties determined at laboratory conditions.

PACKAGING

AKKRO-7T is packaged in 55 gal (208 L) drums, 5 gal (18.9 L) pails and cases of 6/1 gal (3.79 L) plastic bottles.

SHELF LIFE

1 year in original, unopened container

SPECIFICATIONS/COMPLIANCES

Complies with ASTM C 1059, Type II
USDA compliant

MASTER FORMAT #: 03 05 00

19215 Redwood Road • Cleveland, OH 44110
800-321-7628 t • 216-531-9596 f

www.euclidchemical.com

Figure A.15 Bonding Admixture Data Sheet (a)

DIRECTIONS FOR USE

Surface Preparation: Surface must be structurally sound, clean, free of dust, dirt, oil, curing or form release compounds, paint, laitance, efflorescence and other contaminants. New concrete and masonry surfaces must be cured 7 days. Provide an absorptive surface on smooth precast or formed concrete by abrading the surface. Follow the surface preparation directions for the product in which AKKRO-7T is being used as an admixture.

Mixing: Stir AKKRO-7T slowly and thoroughly using slow speed mixing equipment and a clean container. Do not aerate the AKKRO-7T.

Recommended Mixtures:

Bonding Slurry: Dry mix 1 part portland cement with 3 parts dry sand and add undiluted AKKRO-7T to produce a pourable slurry coat. Brush mixture thoroughly into voids and pores of surface to eliminate air pockets. Before slurry coat dries, follow immediately with patching materials that are compatible with portland cement.

Patches, Overlays And Toppings: For depths 1/2 in (12.7 mm) or greater, mix 1 part AKKRO-7T with 3 parts potable water. Proportion dry sand, cement, aggregate and add measured amount of mixing liquid. Featheredging and repairs less than 1/2 in (12.7 mm) deep will require additional AKKRO-7T in the mix.

Terrazzo: Make up a mixing liquid of 1 part AKKRO-7T to 3 parts potable water.

Thin Coat Wall Applications: Manufactured wall coatings including TAMOSEAL, CONCRETE FINISHER, TAMMS SBC, and TAMMS STUCCO FINISH; mix 1 part AKKRO-7T to 3 parts potable water for mixing liquid, or as directed in instructions for mixing coating.

Field Mixed Stucco: Mix 1 part AKKRO-7T to 3 parts potable water.

Cement Plaster Coatings: Applications 3/8 in (9.5 mm) or less; use 1 part AKKRO-7T to 2 parts potable water. Applications over 3/8 in (9.5 mm); use a 1:3 mixture.

Pointing Mortars: Manufactured tuckpointing mortars and other special mortars for thin applications; mix 1 part AKKRO-7T to 3 parts potable water for mixing liquid.

Application: Use only light pressure on trowel and finish with as few strokes as possible. Keep trowel clean and wet to prevent sticking. Do not over trowel. Mixes with AKKRO-7T do not normally require curing, but on hot, dry or windy days, it is advisable to cover with moist burlap for 24 hours or as recommended. Air-cure the surface for 2 to 4 days for normal use, and 4 to 7 days for heavy traffic areas. Refer to "PRECAUTIONS," for curing water containment structures.

CLEAN-UP

Clean tools and equipment with detergent and water immediately following use. Clean drips, spills and smears while mix is still wet. Mixes containing AKKRO-7T are extremely difficult to remove if allowed to dry before cleaning.

PRECAUTIONS/LIMITATIONS

- Do not apply mixes modified with AKKRO-7T when the temperature is below, or expected to fall below 40°F (4°C) within 48 hours.
- Do not use with air entrained cement-based products.
- Excessive moisture and high humidity will slow curing time.
- Provide adequate ventilation when using AKKRO-7T in enclosed tanks, reservoirs or other areas where air circulation is limited.
- Air-cure cisterns, tanks or pools for minimum 7 to 10 days before filling with water.
- Concrete mix designs using AKKRO-7T should be evaluated and tested for performance and application properties prior to use.
- Store between 40°F to 90°F (4°C to 32°C). Protect from freezing.
- In all cases, consult the Safety Data Sheet before use.

Rev. 11.14

Figure A.16 Bonding Admixture Data Sheet (b)

The Euclid Chemical Company

TAMMSWELD

REWETTABLE LATEX BONDING AGENT FOR CONCRETE

EUCLID CHEMICAL

BONDING AGENTS AND ADHESIVES

TAMMSWELD

MASTER FORMAT #: 03 05 00

DESCRIPTION

TAMMSWELD is a rewettable liquid bonding agent and polymer modifier for concrete and cement mortars. TAMMSWELD is a high film build, ethylene vinyl acetate copolymer emulsion.

PRIMARY APPLICATIONS

- Concrete
- Brick
- Tile
- Stone
- Concrete block
- Plaster
- Gypsum board
- Lath
- Plywood
- Hardboard
- Wood
- Interior and exterior surfaces

FEATURES/BENEFITS

- High build bonding agent or polymer admixture
- Increases bond strength
- Improves durability
- Long open time

TECHNICAL INFORMATION

PHYSICAL PROPERTY	VALUE
Solids Content (by weight)	34%
Unit Weight, Specific Gravity	8.9 lbs/gal, 1.07
VOC Content	≤ 70 g/L
Viscosity	500 to 1,000 cp
Bond Strength, ASTM C1042	4,600 psi

Properties determined at laboratory conditions.

PACKAGING

TAMMSWELD is packaged in 5 gal (18.9 L) pails and in cases of 1 gal (3.8 L) jugs (6 jugs per case).

SHELF LIFE

2 years in original, unopened container

SPECIFICATIONS/COMPLIANCES

ASTM C1059, Type I

COVERAGE

200 to 250 ft²/gal (4.91 to 6.14 m²/L) on dense surfaces. Porous surfaces may require more material. Do not exceed 300 ft²/gal (7.3 m²/L).

DIRECTIONS FOR USE

Surface Preparation: Surface must be clean, dry and structurally sound. The substrate must also be free of all curing compounds, form release agents and any other contaminants, which may prevent the proper adhesion of TAMMSWELD. The preferred method of surface preparation is mechanical abrasion. For oil-contaminated surfaces, using steam cleaning in conjunction with a strong emulsifying detergent may be considered. Rinse thoroughly with potable water. Allow the concrete to dry before applying TAMMSWELD.

19215 Redwood Road • Cleveland, OH 44110
800-321-7628 t • 216-531-9596 f

www.euclidchemical.com

Figure A.17 Bonding Agent Data Sheet (a)

Application, Bonding Agent: Stir TAMMSWELD thoroughly before use. Do not dilute. For hand application, wet brushes or rollers before use and shake out excess water. For larger areas or faster application, use airless spray equipment with 0.015 in. to 0.020 in. (0.38 to 0.51 mm) orifice size spray tips. Hold spray gun 12 to 18 inches (30 to 46 cm) from the surface and apply TAMMSWELD using a cross coat technique consisting of a horizontal pass followed by a vertical pass. Extremely porous surfaces may require two coats of TAMMSWELD.

Allow the TAMMSWELD to dry before placing repair mortars, concrete, or toppings. TAMMSWELD will dry in approximately one hour depending on the temperature and humidity. If more than 7 days pass between TAMMSWELD application and placement of the concrete, topping, or mortar, check several areas to ensure adequate adhesion. Make test applications on questionable surfaces.

Application, Polymer Modifier: When using TAMMSWELD to produce a polymer modified mortar, add approximately 3 gal (11.36 L) of TAMMSWELD per 100 lbs (45.4 kg) of cement content in the mortar material. The properties achieved by using TAMMSWELD as a polymer modifier will vary depending on the composition of the mortar, and a thorough evaluation of properties should be completed prior to using the polymer modified mix.

CLEAN-UP

Clean tools and equipment with detergent and water immediately following use. Clean drips and over-spray with water while still wet. Dried TAMMSWELD may require mechanical abrasion for removal.

PRECAUTIONS/LIMITATIONS

- Do not use TAMMSWELD where constant moisture or hydrostatic pressure is present (swimming pools, cisterns or other areas that will be immersed).
- Do not dilute TAMMSWELD.
- Keep from freezing.
- Do not apply to frozen or frost filled surfaces.
- Do not apply if temperature is below 50°F (10°C).
- Do not over-trowel, or overwork cement mortars modified with TAMMSWELD.
- Store at temperatures between 50°F to 90°F (10°C to 32°C).
- In all cases, consult the Safety Data Sheet before use.

Rev. 03.17

Figure A.18 Bonding Agent Data Sheet (b)

FIBERMESH® 150

PRODUCT DATA SHEET

SPECIFY FIBERMESH® 150 FIBERS:

- REDUCED PLASTIC SHRINKAGE CRACKING
- IMPROVED IMPACT, SHATTER AND ABRASION RESISTANCE
- REDUCED WATER MIGRATION AND DAMAGE FROM FREEZE/THAW
- IMPROVED DURABILITY
- AREAS REQUIRING NON-METALLIC MATERIALS
- CONCRETE THAT NEEDS AN ARCHITECTURAL FINISH

FIBERMESH® 150 SYNTHETIC FIBER

Fibermesh 150, formerly Stealth®e3®, micro-reinforcement system for concrete—100 percent virgin homopolymer polypropylene multifilament fibers containing no reprocessed olefin materials. Specifically engineered and manufactured in an ISO 9001:2000 certified facility for use as concrete reinforcement at an application rate of 1.0 to 1.5 lbs per cubic yard (.60 to .90 kg per cubic meter). UL Classified. Complies with National Building Codes and ASTM C 1116/C 1116M, Type III fiber reinforced concrete.

ADVANTAGES

Non-magnetic • Rustproof • Alkali proof • Requires no minimum amount of concrete cover • Is always positioned in compliance with codes • Safe and easy to use • Saves time and hassle.

FEATURES & BENEFITS

- Inhibits and controls the formation of intrinsic cracking in concrete
- Reinforces against impact forces
- Reinforces against abrasion
- Reinforces against the effect of shattering forces
- Reinforces against water migration
- Provides improved durability
- Reduces plastic shrinkage and settlement cracking
- Alternate system to traditional reinforcement when used for secondary (crack control) reinforcing in concrete.

PRIMARY APPLICATIONS

Applicable to all types of concrete which demonstrate a need for resistance to intrinsic cracking and improved water tightness and an aesthetic finish.

- Slabs-on-ground
- Stucco
- Slope paving
- Sidewalks
- Curbs
- Exposed aggregate
- Driveways
- Overlays & toppings

CHEMICAL AND PHYSICAL PROPERTIES

Absorption	Nil	Melt Point	324°F (162°C)
Specific Gravity	0.91	Ignition Point	1100°F (593°C)
Fiber Length*	Graded	Thermal Conductivity	Low
Electrical Conductivity	Low	Alkali Resistance	Alkali Proof
Acid & Salt Resistance	High		

*Also available in single cut lengths

MAKE SURE IT'S TRUE FIBERMESH FIBERMESH.COM

Figure A.19 Fibermesh Data Sheet (a)

FIBERMESH® 150 PRODUCT DATA SHEET

PRODUCT USE

MIXING DESIGNS AND PROCEDURES: Fibermesh® 150 micro reinforcing is a mechanical, not chemical, process.The addition of Fibermesh 150 multifilament fibers do not require any additional water or other mix design changes at normal rates. Fibermesh 150 fibers are added to the mixer before, during or after batching the other concrete materials. Mixing time and speed are specified in ASTM C 94.

FINISHING: Fibermesh 150 micro-reinforced concrete can be finished by any finishing technique. Exposed aggregate, broomed and tined surfaces are no problem.

APPLICATION RATE: The application rate for Fibermesh 150 fibers is 1.0 to 1.5 lbs per cubic yard (.60 to .90 kg per cubic meter). Note: .75 lbs per cubic yard (.44 kg per cubic meter) may be acceptable based on local building codes.

GUIDELINES

Fibermesh 150 fibers should not be used to replace structural, load-bearing reinforcement. Fibermesh 150 fibers should not be used as a means of using thinner concrete sections than original design. Fibermesh 150 fibers should not be used to increase joint spacing past those dimensions suggested by PCA and ACI industry standard guidelines.

COMPATIBILITY

Fibermesh 150 fibers are compatible with all concrete admixtures and performance enhancing chemicals, but require no admixtures to work.

PACKAGING

Fibermesh 150 fibers are available in a variety of packaging options. Special packaging is available for full truckload addition. Fibermesh 150 fibers are packaged, packed into cartons, shrinkwrapped and palletized for protection during shipping.

TECHNICAL SERVICES

Trained Propex Concrete Systems specialists are available worldwide to assist and advise in specifications and field service. Propex Concrete Systems representatives do not engage in the practice of engineering or supervision of projects and are available solely for service and support of our customers.

REFERENCE DOCUMENTS

- ASTM C 94/C 94M Standard Specification for Ready-Mixed Concrete.
- ASTM C 1116/C 1116M Standard Specification for Fiber-Reinforced Concrete.
- ASTM C 1399 Standard Test Method for Obtaining Average Residual-Strength of Fiber-Reinforced Concrete.
- ASTM C 1436 Standard Specification for Materials for Shotcrete.
- ASTM C 1609/C 1609M Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading). Replaces ASTM C 1018.
- ACI 304 Guide for Measuring, Mixing,Transporting and Placing Concrete.
- ACI 506 Guide for Shotcrete. • International Code Council (ICC) NER-414 Evaluation Report.

UL® Classified: Type Fibermesh 150. For use as an alternate or in addition to the welded wire fabric used in Floor-Ceiling D700, D800, D900 Series Designs. Fibers may also be used in Floor-Ceiling Design Nos. G229, G243, G256, G514. Fiber added to concrete mix at a rate of 1.0 lb of fiber for each cubic yard of concrete.

SPECIFICATION CLAUSE

Use Fibermesh 150 only 100 percent virgin polypropylene multifilament fibers containing no reprocessed olefin materials and specifically engineered and manufactured in an ISO 9001:2000 certified facility for use as concrete secondary reinforcement. Application per cubic yard shall equal a minimum of 1.0 lb/yd3 (.60 kg/m3). Fibers are for the control of cracking due to plastic shrinkage, plastic settlement and thermal expansion/contraction, lowered permeability, increased impact, abrasion and shatter resistance. Fiber manufacturer shall document evidence of ten year satisfactory performance history, ISO 9001:2000 certification of manufacturing facility, compliance with applicable building codes and ASTM C 1116/C 1116M, Type III fiber reinforced concrete. Fibrous concrete reinforcement shall be manufactured Propex Operating Company, LLC, 6025 Lee Highway, Suite 425, PO Box 22788, Chattanooga,TN 37422, USA, tel: 423 892 8080, fax: 423 892 0157, web site: fibermesh.com.

Propex™ Fibermesh®

NORTH AMERICA
Propex Operating Company, LLC
6025 Lee Highway, Suite 425
Chattanooga, TN 37421
Tel: 800 621 1273
Tel: 423 892 8080
Fax: 423 892 0157

INTERNATIONAL
Propex Concrete Systems Ltd.
Propex House
9 Royal Court, Basil Close
Chesterfield, Derbyshire, S41 7SL
United Kingdom
Tel: +44 (0) 1246 564200
Fax: +44 (0) 1246 465201

Figure A.20 Fibermesh Data Sheet (b)

The Euclid Chemical Company

CONCRETE SURFACE RETARDER

EUCLID CHEMICAL

FORMULA F & FORMULA S FOR EXPOSED AGGREGATE SURFACES

MISCELLANEOUS

DESCRIPTION

CONCRETE SURFACE RETARDER F & S are chemical formulations which retard, but do not "kill" the set of the mortar at the surface of concrete. When the underlying concrete has hardened, the retarded mortar surface can be flushed off with a stream of water and/or removed by scrubbing with a stiff brush. Since these compounds do not "kill" the set, if they are left on the concrete or unintentionally splashed on other fresh concrete, they will permit the concrete to eventually attain a set and achieve full strength. CONCRETE SURFACE RETARDER is available in two formulations to meet varying job requirements:

Formula F is a paint-like emulsion designed for application directly to forms.

Formula S is a neutral, sprayable liquid for application to freshly placed horizontal concrete surfaces.

PRIMARY APPLICATIONS

- Creation of exposed aggregate surfaces
- Precast panels
- Decorative sidewalks and walkways
- Bond improvement for water-proofing materials
- Slip-resistant surfaces
- Formulations for both horizontal and vertical

FEATURES/BENEFITS

- Safe to use - easy to apply
- Works quickly and effectively
- Provides up to 1/4" (6 mm) depth retardation
- Reduces cost of mechanically preparing surfaces for waterproofing, stucco or plaster application
- Etch depth can be adjusted as desired

TECHNICAL INFORMATION

	Formula S	Formula F
Weight/gal	8.9 lb/gal (1.1 kg/L)	8.8 lb/gal (1.0 kg/L)
Solids Content	17%	30%
VOC	$\leq$5 g/L	653 g/L

Appearance: CONCRETE SURFACE RETARDER **Formula F** is a tan paint-like emulsion for application on vertical surfaces such as forms. CONCRETE SURFACE RETARDER **Formula S** is a low viscosity green liquid for application directly on freshly placed horizontal concrete surfaces.

PACKAGING

CONCRETE SURFACE RETARDER F & S are packaged in 55 gal (208 L) drums and 5 gal (18.9 L) pails.

CONCRETE SURFACE RETARDER S is also available in cases of 1 gal (3.8 L) jugs (6 jugs per case).

SHELF LIFE

2 years in original, unopened containers

COVERAGE

Formula F: 150 ft²/gal (3.7 m²/L). This coverage rate will provide up to 1/4" (6 mm) of surface retardation.
Formula S: 100 to 200 ft²/gal (2.5 to 4.9 m²/L). This coverage rate will provide 1/8" to 3/16" (3.2 to 4.8 mm) of surface retardation.

DIRECTIONS FOR USE

Surface Preparation: Forms to be coated should be clean and free of oil, dirt and form release agents.

Mixing: CONCRETE SURFACE RETARDER does not require pre-blending. These products should be used directly from the container.

CONCRETE SURFACE RETARDER

MASTER FORMAT #: 03 35 23

19215 Redwood Road • Cleveland, OH 44110
800-321-7628 t • 216-531-9596 f

www.euclidchemical.com

Figure A.21 Concrete Surface Retarder Data Sheet (a)

Application: CONCRETE SURFACE RETARDER Formula F should be painted on forms without thinning in a continuous unbroken film. Forms may be coated several days in advance or in as short a time as will allow complete drying of the film. Drying time varies between one and four hours depending on weather conditions. In warm weather, forms may be stripped in one day, in cooler weather allow two to three days. Immediately after stripping remove the retarded surface mortar by flushing off with a stream of water and/or remove by scrubbing with a stiff brush. Pre-cast structural members should be stripped from their forms in their usual time and the surface mortar then removed.

CONCRETE SURFACE RETARDER Formula S is applied to freshly placed horizontal concrete surfaces immediately after final finishing operations. It should be applied by low pressure spray and the treated surfaces then covered to prevent rapid evaporation. The retarded mortar should be flushed off with water in 12 to 24 hours after application depending upon weather conditions.

Use BROWNTONE CS to cure and seal exposed aggregate concrete to give these surfaces a subtle, earthtoned look with an attractive gloss.

Clean Up

Clean tools and equipment with soap and water before the material dries.

Precautions/Limitations

- These products are affected by environmental conditions. Warmer temperatures will allow earlier stripping of forms and earlier surface flushing, while cooler temperatures delay these procedures.
- Store in a dry place and protect from freezing.
- In all cases, consult the Safety Data Sheet before use.

Rev. 02.17

Figure A.22 Concrete Surface Retarder Data Sheet (b)

Appendix B Mix design

B.1 ACI Mix Design Procedure

The following is the procedure for determining a control mix. The section at the end shows the calculation for determining the water adjustments for admixtures.

Note: Following example 7.4 from Materials for Civil and Construction Engineers[1] *which follows the American Concrete Institute (ACI) design procedure.*

Wanted : Design a concrete mix that meets the requirements of Idaho Transportation Department (ITD).

Given:

Design parameters

"High early strength concrete class 50AF with polypropylene fibers"

- Required design strength, f'_c = 5000 psi (*Table 502.01.1*)[2]
- Minimum cement content = 660 lb/yd^3 (*Table 502.01.1*)[2]
- Minimum secondary cementitious material (SCM) content = 20% by weight of total cementitious material (cement + SCM) (*Table 502.01-2*)[2]
- Maximum w/c ratio = 0.42* (Table 502.01-2)[2]
- Air content = 6.5±1.5% (range = 5-8%) (*Table 502.01-2*)[2]
- Class F fly ash shall be used (*Table 502.01-2*)[2]
- Fine aggregate - sand (bridge deck sand) gradation. *Table 703.02-3 – Fine Aggregate Gradation*[2]

[1] Mamlouk, M. S. & Zaniewski, J. P. (2017). *Materials for civil and construction engineers* (4th Ed.). Saddle River, NJ: Pearson Education.

[2] Idaho Transportation Department. (2012) *Standard Specifications for Highway Construction*, Boise, ID.

- Coarse aggregate – ½" nominal aggregate size *Table 703.02-6 –Coarse Aggregate Size No. and Gradation**
- Minimum form work dimension = 8 in.
- Minimum space between rebar = 2.375 in.
- Minimum cover = 2.5 in.
- Fibers = Polypropylene fibers (Fibermesh 150, *Fibermesh*)
- Admixtures
  - Non-chloride accelerator MasterSet AC 534, *BASF*)
  - Air entrainer (MasterAir AE 200, *BASF*)
  - High-range water-reducer (MasterGlenium 1466, *BASF*)
  - Shrinkage reducer(MasterLife SRA 035, *BASF*)
  - Bonding admixture (AKKRO-7T, *Euclid Chemical*)

Materials

- Cement - Type II (G_{cement} = 3.15)
- Fly ash - Class F ($G_{fly\ ash}$ = 2.34)
- Coarse aggregate (CA)
  - Nominal maximum size, river gravel (round) = 1/2 in.
  - Bulk oven-dry specific gravity, G_{CA} = 2.64
  - Absorption = 1.3%
  - Oven dry-rodded density = 99.7 lb/ft^3 = 2692 lb/yd^3
  - Moisture content, MC_{CA} = 1.2%

- Fine aggregate (FA)
  - Natural sand
  - Bulk oven-dry specific gravity, G_{FA} = 2.62
  - Absorption = 2.1%
  - Moisture content, MC_{FA} = 4.9%
    - Fineness modulus = 2.60

Solution

1. Strength requirements

   Standard deviation of compressive strength (less than 15 samples) = 1200 psi

   $s = 1200\, psi$

   $f'_{cr} = f'_c + 1200$

   $f'_{cr} = 5000 + 1200 = 6200\, psi$

   $f'_{cr} = 6200\, psi$

2. Water-cement ratio

   For air entrained concrete, w/c = 0.32; Maximum w/c = 0.42

   ITD suggested w/c = 0.36

   w/c = 0.36

3. Coarse aggregate requirements

   1/2 in. nominal size corresponds to 3/4 in. maximum size.

   3/4 in. < 1/5 of minimum dimension = (1/5)(8) = 1.60 in.

   3/4 in. < 3/4 of rebar spacing = (3/4)(2.375) = 1.78 in.

   3/4 in. < 3/4 of rebar cover = (3/4)(2.5) = 1.88 in.

   Aggregate size is okay

1/2 in. nominal maximum size coarse aggregate and 3.20 FM fine aggregate

Coarse aggregate factor = 0.53

Dry weight of coarse aggregate = (2692)(0.53) = 1427 lb/yd^3

Coarse aggregate (dry) = 1427 lb/yd^3

4. Air content

Range = 5% to 8%

Target air content = 6.5%

Design using 7%

5. Workability

Recommended 6 in. maximum slump[1]

Use 6 in. slump

6. Cementing materials content

Cementitious materials = 660 lb/yd^3

Fly ash = (0.2)(660) = 132 lb/yd^3

Cement = 660 $-$ 132 = 528 lb/yd^3

w/c ratio = 0.36

Water = (660) (0.36) = 237.6 lb/yd^3

Fly ash = 132 lb/yd^3

Cement = 528 lb/yd^3

Required water = 237.6 lb/yd^3

7. Fine aggregate requirements

Find fine aggregate content using the absolute volume method.

Water volume = 237.6 ÷ 62.4 = 3.808 ft^3/yd^3

Cement volume = 528 ÷ (3.15 × 62.4) = 2.686 ft^3/yd^3

Fly ash volume = 132 ÷ (2.34 × 62.4) = 0.904 ft^3/yd^3

Air volume = 0.07 × 27 = 1.890 ft^3/yd^3

Coarse aggregate volume = 1427 ÷ (2.64 × 62.4) = 8.661 ft^3/yd^3

Subtotal volume = 17.949 ft^3/yd^3

Fine aggregate volume = 27 − 17.949 = 9.051 ft^3/yd^3

Fine aggregate dry weight = (9.051)(2.62)(62.4) = 1480 lb/yd^3

Fine aggregate (dry) = 1480 lb/yd^3

8. Moisture corrections

Coarse aggregate: Need 1427 lb/yd^3 in dry condition, so increase by 1.2% for moisture

Moist coarse aggregate = (1427)(1.012) = 1444 lb/yd^3

Fine aggregate (moist) = 1444 lb/yd^3

Fine aggregate: Need 1480 lb/yd^3 in dry condition, so increase by 4.9% for moisture

Moist fine aggregate = (1480)(1.049) = 1552 lb/yd^3

Coarse aggregate (moist) = 1552 lb/yd^3

Water: Adjust the water content

Adjustment = aggregate (absorption - moisture content)

Coarse = 1427 × (0.013 − 0.012) = 1.43 lb/yd^3

Fine = 1480 × (0.021 − 0.049) = −41.43 lb/yd^3

Adjusted water content = 237.6 + 1.43 + (−41.43) = 198 lb/yd^3

Adjusted water content = 198 lb/yd^3

Table 1: Summary of batch ingredients

	per yd^3
Water	198 lb
Cement	528 lb
Fly ash	132 lb
Coarse aggregate	1444 lb
Fine aggregate	1552 lb

[1] Water content is the amount needed after aggregate adjustments

Water Adjustments for Admixtures

When admixtures are included in a mix the percentage of liquids added from each admixture must be considered to maintain the correct water-cement ratio. Water adjustments were made for Accelerator, Shrinkage Reducer, and Bonding Admixture. Air Entrainer and High-Range Water-Reducer were not included because their contribution was insignificant compared to the others.

Accelerator admixture is composed of solids and liquids, with 46.5% being liquids. The liquid component will factor into the water content of the mix and need to be calculated. The amount of AC is calculated with the following equation,

$$Water_{AC} = Dosage \times \frac{TCM}{100} \times \%_{Liquids} \times 29.57351563 \frac{cm^3}{fl\ oz}$$

Where,

$Water_{AC}$ = amount of water that will be taken out, g

$Dosage$ = admixture dosage of admixture, $fl\ oz/cwt$

cwt = hundred weight of cement

TCM = total cementitious materials, lb

$\rho_{admixture}$ = density of admixture, g/cm^3

$\%_{Liquids}$ = percent liquids = 0.465

$29.57351563\ cm^3/fl\ oz$ is a conversion factor

Shrinkage Reducing Admixture (SRA) specifies 100% water replacement

$$Water_{SRA} = Dosage \times \rho_{water} \times 3785.4 \frac{cm^3}{gal}$$

Where,

$Water_{SRA}$ = amount of water that will be taken out, g

$Dosage$ = admixture dosage of admixture, $fl\ oz/cwt$

ρ_{water} = density of water = $1\ g/cm^3$

$3785.4\ cm^3/gal$ is a conversion factor

Bonding Admixture (BA) specifies 100% water replacement. Dosage is one part BA to three parts water (1:3), so there are a total of four parts. The mix water is the water required after the aggregate adjustments. The following equation is used to calculate the equivalent amount of water to be taken out,

$$Water_{BA} = \left(\frac{1}{4}\right) \times Mix\ water$$

Where,

$Water_{SRA}$ = amount of water that will be taken out, g

Finally, the adjusted water is calculated as,

$$Adjusted\ Water = Mix\ Water - Water_{AC} - Water_{SRA} - Water_{BA}$$

The adjusted water is the actual amount of water, after aggregate and admixture adjustments, used for mixing.

B.2 Mix Designs

Table B.1 Mix Proportions - Trial Mix A

Date: 4/12/2017	*Mix A Trial*			
	FA	CA	w/c	0.36
Moisture Content	5.7%	1.9%	Slump (in)	4.75
Absorption	2.1%	1.3%	Air content	8.0%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	10	fl oz/cwt
Fine aggregate (FA)	1564	HRWR	8	fl oz/cwt
Coarse aggregate (CA)	1454	SRA	-	gal/yd^3
Water	177	BA	-	
Water used (adjusted)	156	Fibers	-	lb/yd^3

Note: "Water" accounts for aggregate moisture and absorption. "Water used" is the water needed adjusted for admixtures.

Table B.2 Mix Proportions - Mix A

Date: 4/13/2017	*Mix A - Cylinders, length change prisms (2 batches)*			
	FA	CA	w/c	0.36
Moisture Content	5.7%	1.9%	Slump (in)	3.0, 3.25
Absorption	2.1%	1.3%	Air content	7.2%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	7	fl oz/cwt
Fine aggregate (FA)	1564	HRWR	8	fl oz/cwt
Coarse aggregate (CA)	1454	SRA	-	gal/yd^3
Water	177	BA	-	
Water used (adjusted)	156	Fibers	-	lb/yd^3

Note: The two slump values correspond to the first batch and the second batch respectively. Air content was only taken for the first batch.

Table B.3 Mix Proportions - Trial Mix B

Date: 3/16/2017	*Mix B Trial*			
	FA	CA	w/c	0.36
Moisture Content	6.1%	1.2%	Slump (in)	4.25
Absorption	2.1%	1.3%	Air content	6.4%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	10	fl oz/cwt
Fine aggregate (FA)	1570	HRWR	8	fl oz/cwt
Coarse aggregate (CA)	1443	SRA	-	gal/yd^3
Water	180	BA	-	
Water used (adjusted)	160	Fibers	0.75	lb/yd^3

Table B.4 Mix Proportions – Mix B

Date: 3/16/2017	*Mix B - Cylinders, length change prisms*			
	FA	CA	w/c	0.36
Moisture Content	6.1%	1.2%	Slump (in)	7.0
Absorption	2.1%	1.3%	Air content	6.8%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	10	fl oz/cwt
Fine aggregate (FA)	1570	HRWR	8	fl oz/cwt
Coarse aggregate (CA)	1443	SRA	-	gal/yd^3
Water	180	BA	-	
Water used (adjusted)	160	Fibers	0.75	lb/yd^3

Table B.5 Mix Proportions – Mix C

Date: 3/21/2017	*Mix C - Cylinders, length change prisms*			
	FA	CA	w/c	0.36
Moisture Content	6.1%	1.2%	Slump (in)	5.25
Absorption	2.1%	1.3%	Air content	7.2%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	10	fl oz/cwt
Fine aggregate (FA)	1570	HRWR	8	fl oz/cwt
Coarse aggregate (CA)	1443	SRA	-	gal/yd^3
Water	180	BA	-	
Water used (adjusted)	160	Fibers	1.5	lb/yd^3

Table B.6 Mix Proportions – Trial Mix D

Date: 4/12/2017	*Mix D Trial*			
	FA	CA	w/c	0.36
Moisture Content	4.9%	1.2%	Slump (in)	3.5
Absorption	2.1%	1.3%	Air content	4.8%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	10	fl oz/cwt
Fine aggregate (FA)	1552	HRWR	8	fl oz/cwt
Coarse aggregate (CA)	1443	SRA	1	gal/yd^3
Water	197	BA	-	
Water used (adjusted)	170	Fibers	1.5	lb/yd^3

Table B.7 Mix Proportions – Mix D

Date: 4/3/2017	*Mix D - Cylinders, length change prisms*			
	FA	CA	w/c	0.36
Moisture Content	4.9%	1.2%	Slump (in)	3.5
Absorption	2.1%	1.3%	Air content	4.8%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	10	fl oz/cwt
Fine aggregate (FA)	1552	HRWR	8	fl oz/cwt
Coarse aggregate (CA)	1443	SRA	1	gal/yd^3
Water	197	BA	-	
Water used (adjusted)	170	Fibers	1.5	lb/yd^3

Table B.8 Mix Proportions – Trial 1 Mix E

Date: 4/6/2017	*Mix E Trial 1*			
	FA	CA	w/c	0.36
Moisture Content	4.9%	2.1%	Slump (in)	9.0
Absorption	2.1%	1.3%	Air content	24.0%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	10	fl oz/cwt
Fine aggregate (FA)	1552	HRWR	8	fl oz/cwt
Coarse aggregate (CA)	1457	SRA	1	gal/yd^3
Water	185	BA	1 part BA to 3 parts water	
Water used (adjusted)	111	Fibers	0.75	lb/yd^3

Table B.9 Mix Proportions – Trial 2 Mix E

Date: 4/6/2017	*Mix E Trial 2*			
	FA	CA	w/c	0.36
Moisture Content	4.9%	2.1%	Slump (in)	3.5
Absorption	2.1%	1.3%	Air content	7.0%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	-	fl oz/cwt
Fine aggregate (FA)	1552	HRWR	4	fl oz/cwt
Coarse aggregate (CA)	1457	SRA	1	gal/yd^3
Water	185	BA	1 part BA to 3 parts water	
Water used (adjusted)	111	Fibers	0.75	lb/yd^3

Table B.10 Mix Proportions – Mix E

Date: 4/6/2017	*Mix E - Cylinders, length change prisms (2 batches)*			
	FA	CA	w/c	0.36
Moisture Content	4.9%	1.9%	Slump (in)	2.0, 7.0
Absorption	2.1%	1.3%	Air content	8.4%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	-	fl oz/cwt
Fine aggregate (FA)	1552	HRWR	7	fl oz/cwt
Coarse aggregate (CA)	1454	SRA	1	gal/yd^3
Water	187	BA	1 part BA to 3 parts water	
Water used (adjusted)	113	Fibers	0.75	lb/yd^3

Note: The two slump values correspond to the first batch and the second batch respectively.

Air content was only taken for the first batch.

Table B.11 Mix Proportions – Trial Mix F

Date: 4/10/2017	*Mix F Trial*			
	FA	CA	w/c	0.36
Moisture Content	4.9%	1.9%	Slump (in)	6.5
Absorption	2.1%	1.3%	Air content	8.5%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	-	fl oz/cwt
Fine aggregate (FA)	1552	HRWR	5	fl oz/cwt
Coarse aggregate (CA)	1454	SRA	1	gal/yd^3
Water	187	BA	1 part BA to 3 parts water	
Water used (adjusted)	113	Fibers	-	lb/yd^3

Table B.12 Mix Proportions – Mix F

Date: 4/11/2017	*Mix F - Cylinders, length change prisms (2 batches)*			
	FA	CA	w/c	0.36
Moisture Content	4.9%	1.9%	Slump (in)	2.25, 3.5
Absorption	2.1%	1.3%	Air content	6.6%
	lb/yd^3		Dosage	
Cement	528	AC	70	fl oz/cwt
Fly ash	132	AE	-	fl oz/cwt
Fine aggregate (FA)	1552	HRWR	6	fl oz/cwt
Coarse aggregate (CA)	1454	SRA	1	gal/yd^3
Water	187	BA	1 part BA to 3 parts water	
Water used (adjusted)	113	Fibers	-	lb/yd^3

Note: The two slump values correspond to the first batch and the second batch respectively.

Air content was only taken for the first batch.

Appendix C   Aggregate Analysis Results

Table C.1 Coarse Aggregate Gradation

Sieve Size	Tare Weight (a)	Gross Weight (b)	Weight Retained, c = (a-b)	Cum. Percent Retained, d = sum (c)	Percent Passing e = (100-d)
1"					
3/4"					100.0%
5/8"	510.2	518	7.8	0.4%	99.6%
1/2"	614.2	620.6	6.4	0.7%	99.3%
3/8"	794.5	1623	828.5	39.4%	60.6%
No. 8	504.7	1712.8	1208.1	95.9%	4.1%
No. 4	490.5	513	22.5	97.0%	3.0%
Pan	369.1	433.6	64.5	100.0%	0.0%
Total Weight			2137.8		

Table C.2 Fine Aggregate Gradation

Sieve Size	Tare Weight (a)	Gross Weight (b)	Weight Retained, c = (a-b)	Cum. Percent Retained, d = sum (c)	Percent Passing e = (100-d)
3/8”	724.5	724.5			100.0%
No. 4	769.3	777.9	8.6	8.6	99.1%
No. 8	476.4	554.3	77.9	86.5	91.2%
No. 16	421.7	482.7	61	147.5	84.9%
No. 30	358.3	534.6	176.3	323.8	66.9%
No. 50	284.3	575.4	291.1	614.9	37.2%
No. 100	255	559.6	304.6	919.5	6.1%
No. 200	316.7	365.7	49	968.5	1.1%
Pan	497.5	507.8	10.3	978.8	0.0%
Total Weight			978.8		

Table C.3 Bulk Density of Coarse Aggregate

	As Measured	Units	Converted	Units
Weight of 0.1 ft^3 Bucket	2587.8	g	5.705	lb
Weight of Aggregate and Bucket	7110.8	g	15.675	lb
Weight of Aggregate	4523.0	g	9.971	lb
Bulk Density			99.71	lb/ft^3

Table C.4 Bulk Density of Coarse Aggregate

	As Measured	Units	Converted	Units
Weight of 0.1 ft^3 Bucket	2587.8	g	5.705	lb
Weight of Aggregate and Bucket	7107.9	g	15.670	lb
Weight of Aggregate	4520.1	g	9.965	lb
Bulk Density			99.65	lb/ft^3

Table C.5 Apparent Density of Coarse Aggregate

	As Measured	Units	Converted	Units
Tare wt of pycnometer	543.8	g		lb
Wt of pycnometer and water	1703.1	g		lb
Volume of pycnometer (water)	1153.9	g or cm^3	0.041	ft^3
Wt of pycnometer and aggregate	1378.6	g		lb
Wt of aggregate	834.8	g	1.8408	lb
Wt of pycnometer, aggregate and water	2221.5	g		lb
Wt of aggregate and water	1677.7	g		lb
Volume of water less aggregate			0.0298	ft^3
Volume of aggregate			0.0112	ft^3
Apparent density of aggregate			164.3	lb/ft^3

Table C.6 Apparent Density of Fine Aggregate

	As Measured	Units	Converted	Units
Tare wt of pycnometer	543.8	g		lb
Wt of pycnometer and water	1703.1	g		lb
Volume of pycnometer (water)	1153.9	g or cm^3	0.041	ft^3
Wt of pycnometer and aggregate	1484.0	g		lb
Wt of aggregate	940.2	g	2.073	lb
Wt of pycnometer, aggregate and water	2279.1	g		lb
Wt of aggregate and water	1735.3	g		lb
Volume of water less aggregate			0.0281	ft^3
Volume of aggregate			0.0129	ft^3
Apparent density of aggregate			160.7	lb/ft^3

Appendix D Compression Results

Table D.1 Compression Results – Mix A

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date Cast: 4/13/2017						
A-1	4.005	4.012	12.620	103,540	8,205	28
A-2	4.023	4.000	12.639	103,090	8,157	28
A-3	3.993	4.028	12.632	93,070	7,368	28
A-4	4.023	3.981	12.579	98,320	7,816	28
A-5	4.005	4.003	12.592	95,900	7,616	28
A-6	3.991	4.023	12.610	92,710	7,352	28
				Average 28-day	7,752	
				Std. Dev.	374	
A-13	4.015	3.980	12.551	41,660	3,319	1
A-14	3.992	4.020	12.604	38,720	3,072	1
				Average 1-day	3,196	

Table D.2 Compression Results – Mix B

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 3/16/2017						
B-1	4.018	4.020	12.686	101,710	8,017	28
B-2	4.007	4.014	12.632	106,110	8,400	28
B-3	4.002	4.025	12.651	105,650	8,351	28
B-4	4.018	4.015	12.670	110,510	8,722	28
B-5	4.020	4.032	12.730	111,900	8,790	28
B-6	4.008	4.011	12.626	107,870	8,543	28
				Average 28-day	8,471	
				Std. Dev.	281	
B-13	4.012	3.999	12.601	43,220	3,430	1
B-14	4.019	3.986	12.582	43,460	3,454	1
				Average 1-day	3,442	

Table D.3 Compression Results – Mix C

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 3/21/2017						
C-1	4.015	4.002	12.620	94,500	7,488	28
C-2	4.008	4.012	12.629	96,710	7,658	28
C-3	4.027	3.995	12.636	98,210	7,772	28
C-4	3.987	4.030	12.620	101,080	8,010	28
C-5	3.991	4.010	12.570	97,930	7,791	28
C-6	4.036	3.984	12.629	106,600	8,441	28
				Average 28-day	7,860	
				Std. Dev.	332	
C-13	4.003	4.011	12.610	44,900	3,561	1
C-14	4.021	3.990	12.601	44,620	3,541	1
				Average 1-day	3,551	

Table D.4 Compression Results – Mix D

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 4/4/2017						
D-1	4.007	4.037	12.705	104,250	8,205	28
D-2	4.025	3.989	12.610	114,480	9,078	28
D-3	3.991	4.031	12.636	103,290	8,175	28
D-4	4.011	4.006	12.620	109,200	8,653	28
D-5	4.002	4.027	12.658	122,400	9,670	28
D-6	3.993	4.011	12.579	118,310	9,405	28
				Average 28-day	8,864	
				Std. Dev.	623	
D-13	4.020	4.010	12.661	38,280	3,024	1
D-14	4.015	3.999	12.610	39,390	3,124	1
				Average 1-day	3,074	

Table D.5 Compression Results – Mix E

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 4/6/2017						
E-1	4.027	3.976	12.576	98,130	7,803	28
E-2	4.000	4.019	12.626	101,250	8,019	28
E-3	3.997	4.020	12.620	97,810	7,750	28
E-4	4.025	3.994	12.626	93,440	7,401	28
E-5	4.019	4.008	12.651	93,500	7,391	28
E-6	3.979	4.054	12.670	100,070	7,898	28
				Average 28-day	7,710	
				Std. Dev.	260	
E-13	4.001	4.016	12.620	32,220	2,553	1
E-14	4.028	3.990	12.623	32,150	2,547	1
				Average 1-day	2,550	

Table D.6 Compression Results – Mix F

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 4/11/2017						
F-1	3.997	4.020	12.620	105,190	8,335	28
F-2	3.985	3.997	12.510	107,660	8,606	28
F-3	3.987	3.996	12.513	106,620	8,521	28
F-4	4.007	3.994	12.570	95,930	7,632	28
F-5	3.980	4.017	12.557	99,570	7,929	28
F-6	4.006	3.983	12.532	99,520	7,941	28
				Average 28-day	8,161	
				Std. Dev.	385	
F-13	4.013	4.005	12.623	35,740	2,831	1
F-14	4.012	4.004	12.617	32,980	2,614	1
				Average 1-day	2,723	

Table D.7 Compression Results – Interface Bond Precast Set 1

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 7/12/2017						
PC-1	4.022	4.015	12.683	94,470	7,449	28
PC-2	4.004	4.008	12.604	89,920	7,134	28
PC-3	4.019	4.012	12.664	87,240	6,889	28
PC-4	4.017	4.003	12.629	89,650	7,099	28
PC-5	4.025	4.011	12.680	85,170	6,717	28
PC-6	4.019	4.012	12.664	75,970	5,999	28
				Average	6,605	
				Std. Dev.	498	

Table D.8 Compression Results – Interface Bond Precast Set 2

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 7/20/2017						
PC-1	4.018	4.023	12.696	71,750	5,652	28
PC-2	4.013	4.025	12.686	72,700	5,731	28
PC-3	4.016	4.010	12.648	74,890	5,921	28
				Average	5,768	
				Std. Dev.	138	

Table D.9 Compression Results – Headed Bar Pullout

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 9/15/2017						
D-1	3.977	3.972	12.407	107,100	8,632	28
D-2	3.978	3.972	12.410	109,830	8,850	28
D-3	3.974	3.973	12.400	97,680	7,877	28
				Average	8,453	
Date cast: 7/31/2017						
PC-1	4.004	4.000	12.579	64,390	5,119	74
PC-2	3.990	3.983	12.482	69,850	5,596	74
PC-3	3.992	4.020	12.604	63,750	5,058	74
				Average	5,258	

Table D.10 Compression Results – LB-1, LB-2

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 10/30/2017						
D-1	3.982	3.965	12.400	101,270	8,167	28
D-2	3.956	3.980	12.366	104,500	8,451	28
D-3	3.968	3.980	12.404	101,360	8,172	28
				Average	8,263	
Date cast: 7/31/2017						
PC-1	4.010	3.994	12.579	60,120	4,779	119
PC-2	4.010	3.991	12.570	63,720	5,069	119
PC-3	4.003	3.998	12.570	60,550	4,817	119
PC-4	4.005	4.011	12.617	62,290	4,937	119
				Average	4,901	

Table D.11 Compression Results – LB-3, LB-4

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 11/1/2017						
D-1	3.982	3.978	12.441	105,910	8,513	28
D-2	3.970	3.974	12.391	102,840	8,300	28
D-3	3.983	3.970	12.419	107,730	8,675	28
				Average	8,496	
Date cast: 7/31/2017						
PC-1	4.000	4.017	12.620	63,370	5,021	121
PC-2	3.983	3.996	12.500	60,260	4,821	121
PC-3	3.996	4.013	12.595	60,730	4,822	121
PC-4	3.988	3.988	12.491	63,680	5,098	121
				Average	4,940	

Table D.12 Compression Results – LB-5, LB-6

Sample	D_1 (in)	D_2 (in)	Area (in^2)	Max Load (lb)	Compressive Strength (psi)	Age (day)
Date cast: 11/3/2017						
D-1	3.982	3.982	12.454	108,320	8,698	28
D-2	3.976	3.971	12.400	101,850	8,213	28
D-3	3.970	3.965	12.363	98,860	7,996	28
				Average	8,303	
Date cast: 7/31/2017						
PC-1	4.010	3.986	12.554	65,450	5,214	123
PC-2	4.002	4.000	12.573	63,530	5,053	123
PC-3	3.992	3.987	12.500	60,310	4,825	123
PC-4	4.004	3.977	12.507	64,630	5,168	123
				Average	5,065	

Appendix E Split Tensile Results

Table E.1 Splitting Tensile Results – Mix A

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 4/13/2017								
A-7	3.988	4.014	4.040	8.042	8.041	38,060	751	28
A-8	4.040	4.000	4.034	8.154	8.107	38,620	751	28
A-9	4.011	4.034	4.007	8.093	8.095	41,120	805	28
A-10	3.995	4.006	4.007	8.070	8.083	37,900	746	28
A-11	4.007	3.974	4.019	8.133	8.128	37,810	740	28
A-12	3.991	4.007	3.997	8.097	8.043	40,750	804	28
						Average	766	
						Std. Dev.	30	

Table E.2 Splitting Tensile Results – Mix B

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 3/16/2017								
B-7	4.000	4.024	4.033	8.015	8.020	33,320	658	28
B-8	4.033	4.005	4.060	8.002	8.064	31,350	616	28
B-9	4.037	4.060	4.001	8.012	7.977	26,480	523	28
B-10	3.981	3.995	4.018	8.093	8.088	35,670	702	28
B-11	4.001	3.991	4.025	7.984	8.033	30,640	608	28
B-12	4.006	4.018	4.000	8.055	8.031	34,870	689	28
						Average	633	
						Std. Dev.	66	

Table E.3 Splitting Tensile Results – Mix C

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 3/21/2017								
C-7	3.994	4.017	4.043	7.987	7.966	34,400	683	28
C-8	4.043	3.980	3.998	8.050	8.042	38,660	763	28
C-9	3.992	3.998	4.063	8.044	8.024	35,160	693	28
C-10	3.993	4.035	4.009	8.085	8.107	36,940	724	28
C-11	4.063	3.995	4.068	8.010	8.034	37,090	728	28
C-12	4.003	4.009	3.996	8.099	8.088	40,760	801	28
						Average	732	
						Std. Dev.	44	

Table E.4 Splitting Tensile Results – Mix D

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 4/4/2017								
D-7	3.996	4.001	4.010	8.035	8.043	43,840	867	28
D-8	4.010	3.992	4.041	8.031	8.032	38,800	766	28
D-9	4.007	4.041	4.024	8.092	8.128	46,540	908	28
D-10	3.992	4.009	4.038	8.124	8.164	40,400	787	28
D-11	4.024	4.007	4.036	8.097	8.097	43,730	855	28
D-12	4.032	4.038	4.043	7.997	8.020	42,110	829	28
						Average	835	
						Std. Dev.	53	

Table E.5 Splitting Tensile Results – Mix E

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 4/6/2017								
E-7	3.983	3.995	4.006	8.046	8.034	37,520	744	28
E-8	4.006	3.992	4.042	8.053	8.066	37,490	738	28
E-9	4.009	4.042	4.052	8.023	8.044	35,160	691	28
E-10	3.990	4.027	4.019	7.977	7.981	38,080	757	28
E-11	4.052	3.980	3.974	8.060	8.050	40,510	800	28
E-12	4.001	4.019	4.063	8.083	8.072	38,560	755	28
						Average	747	
						Std. Dev.	35	

Table E.6 Splitting Tensile Results – Mix F

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 4/11/2017								
F-7	3.982	4.008	4.016	7.974	7.973	37,780	754	28
F-8	4.016	3.986	4.021	8.015	7.995	39,570	785	28
F-9	4.000	4.021	3.978	8.046	8.104	36,220	714	28
F-10	3.979	3.972	4.013	8.057	8.040	39,110	776	28
F-11	3.978	3.988	4.045	8.063	8.063	38,900	767	28
F-12	4.004	4.013	4.022	8.080	8.066	40,250	791	28
						Average	764	
						Std. Dev.	28	

Table E.7 Splitting Tensile Results – Interface Bond Precast Set 1

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 7/12/2017								
PC-7	4.002	4.022	4.044	8.066	8.052	31,310	615	28
PC-8	4.044	3.993	4.033	8.138	8.086	35,180	686	28
PC-9	4.018	4.033	4.023	8.061	8.043	37,100	729	28
PC-10	3.990	4.011	4.097	7.969	8.176	31,830	622	28
PC-11	4.023	3.989	3.994	8.023	8.064	34,270	678	28
PC-12	4.030	4.097	4.026	8.043	8.092	35,930	700	28
						Average	672	
						Std. Dev.	45	

Table E.8 Splitting Tensile Results – Interface Bond Precast Set 2

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 7/20/2017								
PC-4	4.025	4.012	4.002	8.041	8.140	30,990	608	28
PC-5	4.002	4.037	3.991	8.054	8.021	33,250	657	28
PC-6	4.018	3.991	4.006	8.142	8.061	29,970	588	28
						Average	617	
						Std. Dev.	35	

Table E.9 Splitting Tensile – Headed Bar Pullout

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 9/15/2017								
D-5	3.970	3.973	3.979	7.998	8.033	38,160	763	28
D-6	3.982	3.981	3.969	8.007	8.014	38,530	770	28
D-7	3.974	3.985	3.984	8.022	8.038	38,670	770	28
						Average	768	
Date Cast: 7/31/2017								
PC-4	3.991	4.042	4.068	8.043	8.096	31,350	613	74
PC-5	3.981	3.999	4.015	8.107	8.08	31,440	619	74
PC-6	3.963	4.004	4.032	8.047	8.005	30,830	611	74
						Average	614	

Table E.10 Splitting Tensile - LB1, LB2

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 10/30/2017								
D-4	3.982	3.972	3.98	8.018	8.001	41,200	823	28
D-5	3.980	3.974	3.976	8.015	8.037	35,450	707	28
D-6	3.977	3.985	3.987	8.020	8.020	37,790	753	28
						Average	761	
Date Cast: 7/31/2017								
PC-5	3.964	4.012	4.073	8.000	8.024	30,410	602	119
PC-6	3.962	4.031	4.062	7.983	8.051	29,640	586	119
PC-7	3.964	3.998	4.019	8.033	8.032	30,920	614	119
						Average	600	

Table E.11 Splitting Tensile - LB-3, LB-4

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 11/1/2017								
D-4	3.973	3.980	3.969	8.050	8.047	38,940	775	28
D-5	3.983	3.986	3.988	8.052	8.043	36,700	728	28
D-6	3.971	3.979	3.972	8.034	8.053	40,310	803	28
						Average	769	
Date Cast: 7/31/2017								
PC-5	3.967	4.002	4.042	7.993	7.991	31,690	631	121
PC-6	3.971	3.991	4.033	8.015	7.990	30,670	610	121
PC-7	3.952	3.992	4.033	7.978	7.964	29,740	595	121
						Average	612	

Table E.12 Splitting Tensile - LB-5, LB-6

Sample	D_1 (in)	D_2 (in)	D_3 (in)	L_1 (in)	L_2 (in)	Max Load (lb)	Split Tensile Strength (psi)	Age (day)
Date Cast: 11/3/2017								
D-4	3.981	3.978	3.969	8.051	8.035	43,780	872	28
D-5	3.982	3.979	3.985	7.973	8.041	38,460	768	28
D-6	3.967	3.966	3.965	8.017	8.012	36,340	728	28
						Average	789	
Date Cast: 7/31/2017								
PC-5	3.972	4.004	4.034	8.006	7.958	28,330	564	123
PC-6	3.958	3.997	4.018	8.025	8.039	27,740	551	123
PC-7	3.974	3.987	4.019	7.995	8.003	30,630	610	123
						Average	575	

Appendix F Length Change Results

Table F.1 Shrinkage Measurements – Mix A

Date	4/14/17	5/11	5/15	5/18	5/25	6/8	6/22	7/6	8/3
	Initial	End curing	Air drying (days after moist curing)						
Day	1-day	0	4	7	14	28	42	56*	84
A-1	0.2883	0.2893	0.2872	0.2861	0.2848	0.2839	0.2833	0.2830	0.2828
A-2	0.3307	0.3317	0.3295	0.3280	0.3269	0.3259	0.3256	0.3253	0.3250
A-3	0.2290	0.2298	0.2277	0.2260	0.2251	0.2239	0.2237	0.2234	0.2231
A-4	-0.0815	-0.0808	-0.0830	-0.0832	-0.0831	-0.0838	-0.0838	-0.0838	-0.0838
A-5	-0.0235	-0.0227	-0.0251	-0.0264	-0.0279	-0.0291	-0.0293	-0.0296	-0.0299
A-6	-0.0260	-0.0255	-0.0277	-0.0289	-0.0303	-0.0316	-0.0318	-0.0321	-0.0324

Note: Units = inches
*Linear interpolation

Date	8/31	9/28	10/26	11/23	12/21	1/18/18	2/15	3/15	4/12
Day	112	140	168	196	224	252	280	308	336
A-1	0.2828	0.2821	0.2820	0.2820	0.2816	0.2815	0.2815	0.2814	0.2816
A-2	0.3247	0.3242	0.3240	0.3244	0.3238	0.3236	0.3235	0.3235	0.3235
A-3	0.2232	0.2226	0.2222	0.2221	0.2219	0.2217	0.2217	0.2216	0.2217
A-4	-0.0838	-0.0836	-0.0835	-0.0832	-0.0825	-0.0828	-0.0828	-0.0887	-0.0887
A-5	-0.0301	-0.0304	-0.0307	-0.0310	-0.0310	-0.0312	-0.0312	-0.0312	-0.0313
A-6	-0.0324	-0.0329	-0.0333	-0.0335	-0.0336	-0.0337	-0.0337	-0.0337	-0.0338

Table F.2 Shrinkage Measurements – Mix B

Date	3/17/17	4/13	4/17	4/20	4/27	5/11	5/25	6/8	7/6
	Initial	End curing	Air drying (days after moist curing)						
Day	1-day	0	4	7	14	28	42	56	84
B-1	-0.0240	-0.0228	-0.0259	-0.0267	-0.0278	-0.0287	-0.0290	-0.0296	-0.0296
B-2	-0.0508	-0.0495	-0.0528	-0.0537	-0.0547	-0.0557	-0.0561	-0.0565	-0.0565
B-3	-0.0260	-0.0250	-0.0278	-0.0288	-0.0297	-0.0306	-0.0310	-0.0316	-0.0315
B-4	0.1161	0.1173	0.1149	0.1142	0.1129	0.1119	0.1114	0.1109	0.1108
B-5	0.3131	0.3142	0.3119	0.3110	0.3100	0.3090	0.3085	0.3080	0.3080
B-6	0.3755	0.3765	0.3746	0.3735	0.3724	0.3715	0.3710	0.3706	0.3705

Note: Units = inches

Date	8/3	8/31	9/28	10/26	11/23	12/21	1/18/18	2/15	3/15
Day	112	140	168	196	224	252	280	308	336
B-1	-0.0297	-0.0299	-0.0301	-0.0305	-0.0305	-0.0306	-0.0307	-0.0308	-0.0308
B-2	-0.0567	-0.0569	-0.0572	-0.0575	-0.0573	-0.0576	-0.0576	-0.0577	-0.0577
B-3	-0.0316	-0.0318	-0.0321	-0.0323	-0.0323	-0.0324	-0.0325	-0.0325	-0.0325
B-4	0.1108	0.1105	0.1103	0.1098	0.1103	0.1098	0.1098	0.1096	0.1096
B-5	0.3078	0.3077	0.3074	0.3070	0.3071	0.3070	0.3069	0.3068	0.3068
B-6	0.3705	0.3703	0.3699	0.3697	0.3697	0.3696	0.3694	0.3693	0.3692

Table F.3 Shrinkage Measurements – Mix C

Date	3/22/17	4/18	4/22	4/25	5/2	5/16	5/30	6/13	7/11
	Initial	End curing	Air drying (days after moist curing)						
Day	1-day	0	4	7	14	28	42	56	84
C-1	-0.0185	-0.0178	-0.0201	-0.0209	-0.0227	-0.0237	-0.0238	-0.0241	-0.0244
C-2	0.2681	0.2689	0.2663	0.2655	0.2639	0.2630	0.2630	0.2628	0.2626
C-3	-0.0634	-0.0626	-0.0652	-0.0660	-0.0675	-0.0683	-0.0686	-0.0688	-0.0691
C-4	-0.0184	-0.0177	-0.0203	-0.0211	-0.0226	-0.0235	-0.0237	-0.0240	-0.0242
C-5	0.3313	0.3321	0.3298	0.3290	0.3278	0.3268	0.3266	0.3265	0.3262
C-6	0.4107	0.4116	0.4095	0.4086	0.4074	0.4066	0.4063	0.4061	0.4059

Note: Units = inches

Date	8/8	9/5	10/3	10/31	11/28	12/26	1/23/18	2/20	3/20
Day	112	140	168	196	224	252	280	308	336
C-1	-0.0246	-0.0249	-0.0253	-0.0255	-0.0253	-0.0255	-0.0256	-0.0257	-0.0257
C-2	0.2623	0.2620	0.2617	0.2612	0.2620	0.2614	0.2612	0.2612	0.2613
C-3	-0.0693	-0.0696	-0.0699	-0.0703	-0.0697	-0.0701	-0.0703	-0.0702	-0.0702
C-4	-0.0245	-0.0246	-0.0250	-0.0255	-0.0248	-0.0251	-0.0254	-0.0253	-0.0253
C-5	0.3259	0.3256	0.3255	0.3250	0.3255	0.3253	0.3248	0.3249	0.3248
C-6	0.4055	0.4051	0.4050	0.4046	0.4051	0.4048	0.4044	0.4046	0.4045

Table F.4 Shrinkage Measurements – Mix D

Date	4/5/17	5/2	5/6	5/9	5/16	5/30	6/13	6/27	7/25
	Initial	End curing	Air drying (days after moist curing)						
Day	1-day	0	4*	7	14	28	42	56	84
D-1	-0.0255	-0.0243	-0.0258	-0.0269	-0.0280	-0.0285	-0.0290	-0.0296	-0.0296
D-2	-0.0719	-0.0711	-0.0725	-0.0735	-0.0745	-0.0750	-0.0754	-0.0760	-0.0761
D-3	-0.0218	-0.0209	-0.0223	-0.0233	-0.0240	-0.0248	-0.0252	-0.0257	-0.0260
D-4	0.3269	0.3281	0.3268	0.3259	0.3252	0.3243	0.3241	0.3236	0.3234
D-5	0.2724	0.2735	0.2724	0.2715	0.2707	0.2699	0.2697	0.2692	0.2690
D-6	0.3880	0.3892	0.3881	0.3872	0.3863	0.3855	0.3853	0.3848	0.3845

Note: Units = inches
*Linear interpolation

Date	8/22	9/19	10/17	11/14	12/12	1/9/18	2/6	3/6	4/3
Day	112	140	168	196	224	252	280	308	336
D-1	-0.0303	-0.0300	-0.0303	-0.0302	-0.0304	-0.0307	-0.0308	-0.0309	-0.0309
D-2	-0.0768	-0.0764	-0.0768	-0.0767	-0.0768	-0.0772	-0.0774	-0.0774	-0.0774
D-3	-0.0266	-0.0262	-0.0265	-0.0267	-0.0268	-0.0270	-0.0273	-0.0273	-0.0274
D-4	0.3227	0.3232	0.3228	0.3227	0.3225	0.3222	0.3219	0.3219	0.3219
D-5	0.2683	0.2687	0.2683	0.2684	0.2682	0.2678	0.2675	0.2675	0.2674
D-6	0.3839	0.3843	0.3840	0.3840	0.3839	0.3835	0.3832	0.3832	0.3832

Table F.5 Shrinkage Measurements – Mix E

Date	4/7/17	5/4	5/8	5/11	5/18	6/1	6/15	6/29	7/27
	Initial	End curing	Air drying (days after moist curing)						
Day	1-day	0	4	7	14	28	42	56	84
E-1	0.3188	0.3193	0.3183	0.3177	0.3163	0.3156	0.3151	0.3149	0.3149
E-2	0.3216	0.3223	0.3207	0.3202	0.3191	0.3181	0.3176	0.3174	0.3173
E-3	0.2395	0.2403	0.2388	0.2384	0.2371	0.2362	0.2357	0.2354	0.2354
E-4	-0.0232	-0.0226	-0.0240	-0.0244	-0.0258	-0.0266	-0.0272	-0.0273	-0.0276
E-5	-0.0742	-0.0737	-0.0752	-0.0749	-0.0770	-0.0778	-0.0784	-0.0786	-0.0788
E-6	-0.0256	-0.0252	-0.0263	-0.0269	-0.0284	-0.0291	-0.0297	-0.0299	-0.0300

Note: Units = inches

Date	8/24	9/21	10/19	11/16	12/14	1/11/18	2/8	3/8	4/5
Day	112	140	168	196	224	252	280	308	336
E-1	0.3144	0.3144	0.3145	0.3144	0.3140	0.3139	0.3138	0.3137	0.3137
E-2	0.3168	0.3167	0.3169	0.3166	0.3162	0.3161	0.3159	0.3159	0.3158
E-3	0.2347	0.2349	0.2350	0.2347	0.2343	0.2343	0.2342	0.2341	0.2341
E-4	-0.0280	-0.0279	-0.0277	-0.0279	-0.0283	-0.0285	-0.0286	-0.0287	-0.0287
E-5	-0.0792	-0.0792	-0.0792	-0.0793	-0.0796	-0.0797	-0.0799	-0.0800	-0.0801
E-6	-0.0306	-0.0304	-0.0304	-0.0308	-0.0309	-0.0310	-0.0312	-0.0312	-0.0312

Table F.6 Shrinkage Measurements – Mix F

Date	4/12/17	5/9	5/13	5/16	5/23	6/6	6/20	7/4	8/1
	Initial	End curing	Air drying (days after moist curing)						
Day	1-day	0	4	7	14	28	42	56	84
F-1	-0.0246	-0.0241	-0.0247	-0.0255	-0.0269	-0.0276	-0.0282	-0.0284	-0.0285
F-2	-0.0251	-0.0244	-0.0255	-0.0265	-0.0280	-0.0287	-0.0293	-0.0296	-0.0295
F-3	-0.0761	-0.0753	-0.0762	-0.0771	-0.0786	-0.0794	-0.0801	-0.0804	-0.0804
F-4	0.3233	0.3241	0.3233	0.3224	0.3211	0.3203	0.3197	0.3194	0.3194
F-5	0.3249	0.3257	0.3248	0.3240	0.3225	0.3220	0.3212	0.3210	0.3209
F-6	0.2191	0.2198	0.2190	0.2180	0.2171	0.2166	0.2157	0.2154	0.2154

Note: Units = inches

Date	8/29	9/26	10/24	11/21	12/19	1/16/18	2/13	3/13	4/10
Day	112	140	168	196	224	252	280	308	336
F-1	-0.0292	-0.0293	-0.0292	-0.0292	-0.0294	-0.0296	-0.0296	-0.0298	-0.0297
F-2	-0.0304	-0.0304	-0.0303	-0.0304	-0.0306	-0.0307	-0.0309	-0.0309	-0.0309
F-3	-0.0811	-0.0810	-0.0809	-0.0812	-0.0813	-0.0815	-0.0816	-0.0817	-0.0818
F-4	0.3188	0.3187	0.3187	0.3187	0.3184	0.3184	0.3183	0.3183	0.3183
F-5	0.3203	0.3203	0.3202	0.3202	0.3199	0.3198	0.3198	0.3198	0.3198
F-6	0.2148	0.2150	0.2149	0.2147	0.2145	0.2145	0.2143	0.2142	0.2141

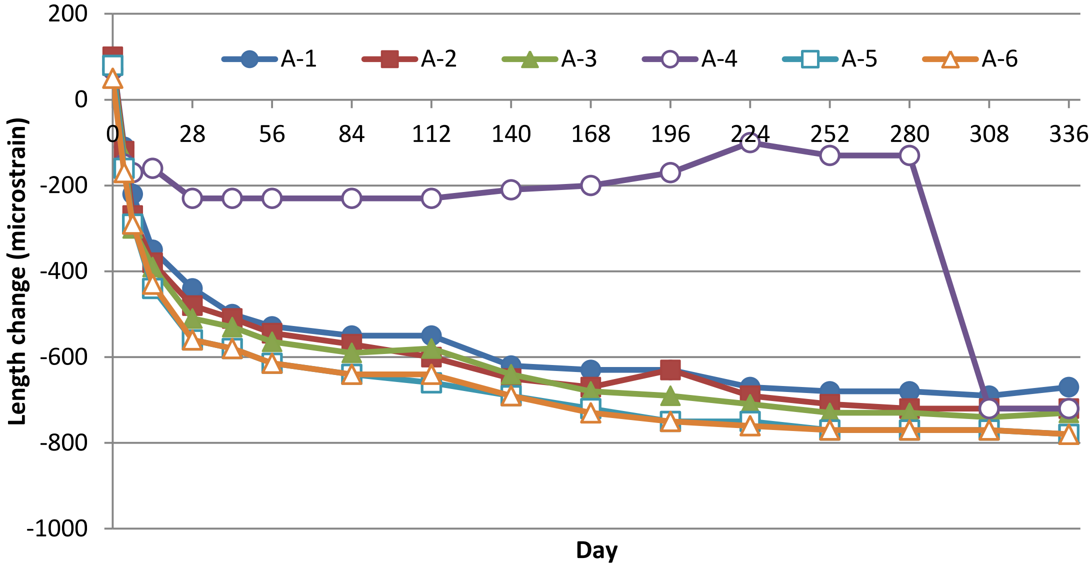


Figure F.1 Shrinkage - Mix A

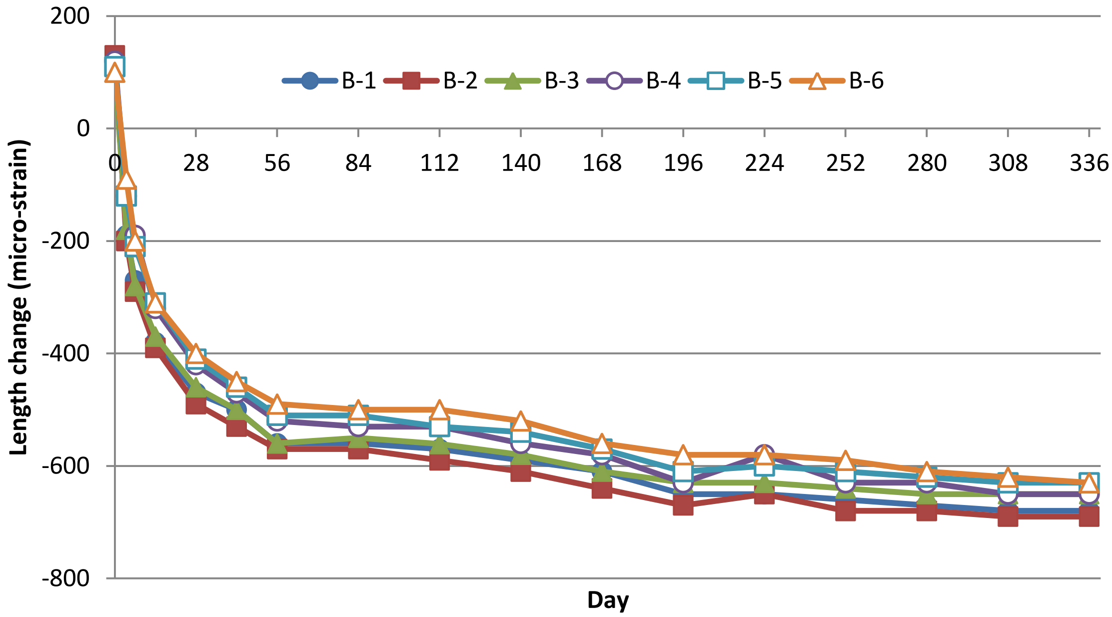


Figure F.2 Shrinkage - Mix B

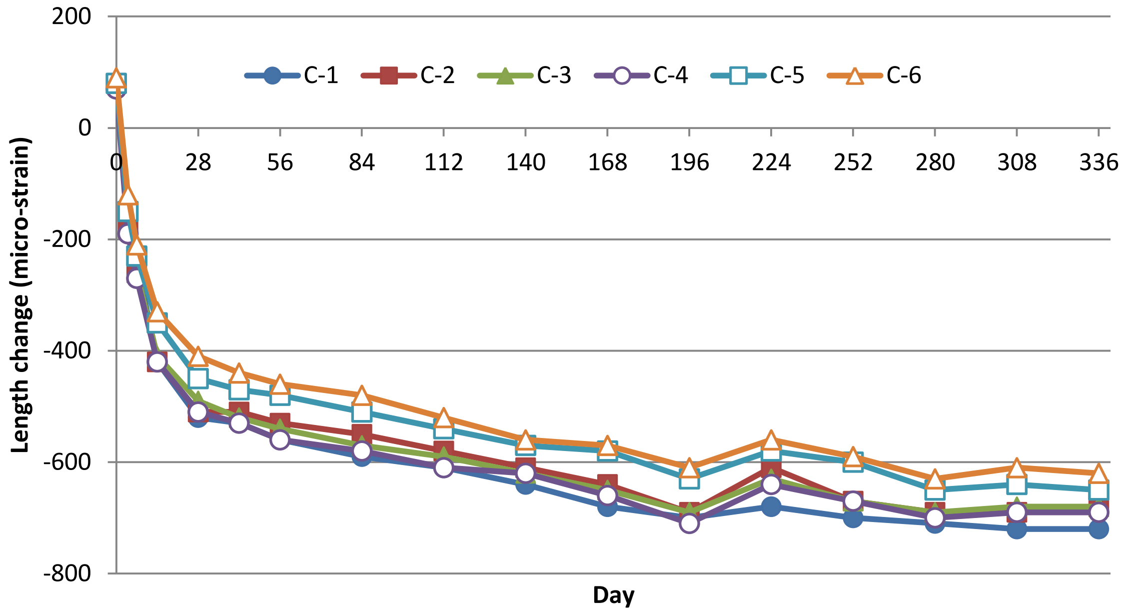


Figure F.3 Shrinkage - Mix C

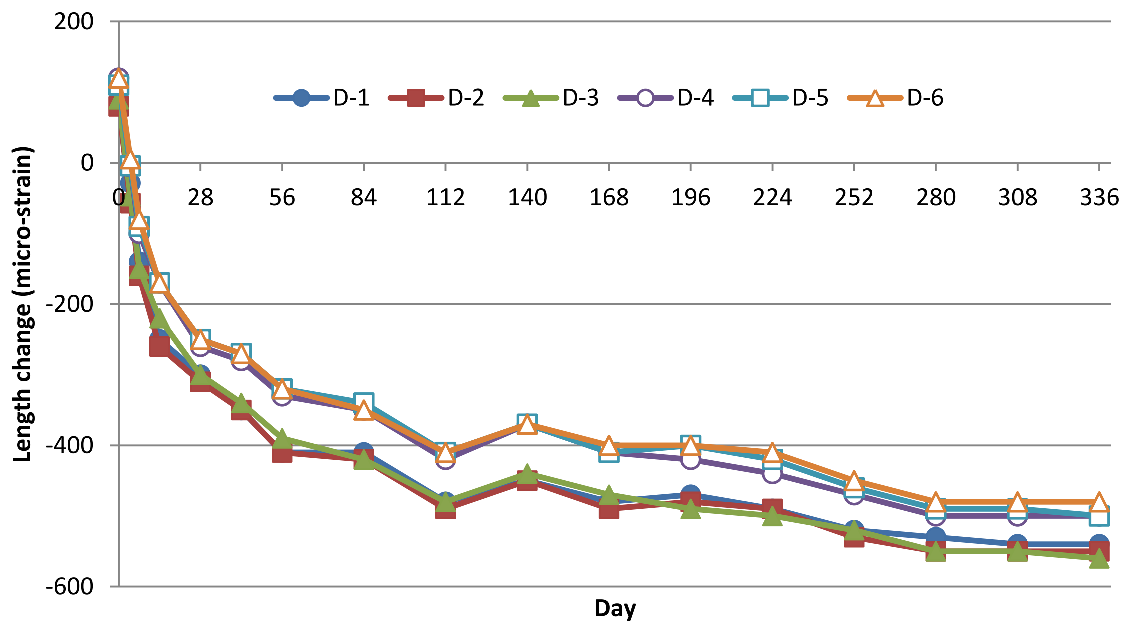


Figure F.4 Shrinkage - Mix D

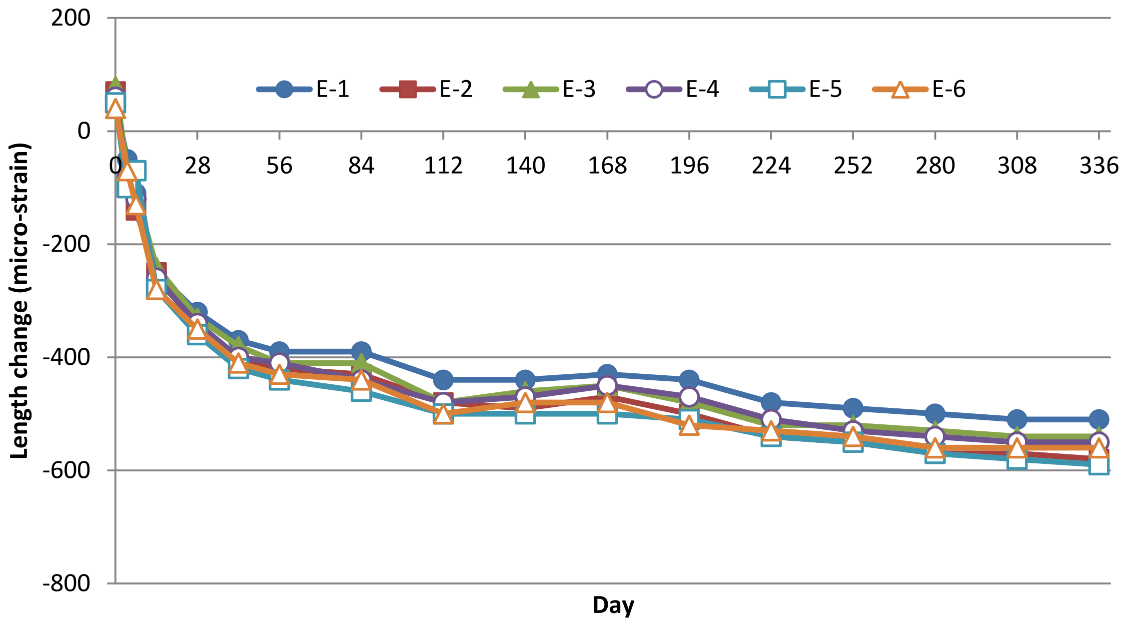


Figure F.5 Shrinkage - Mix E

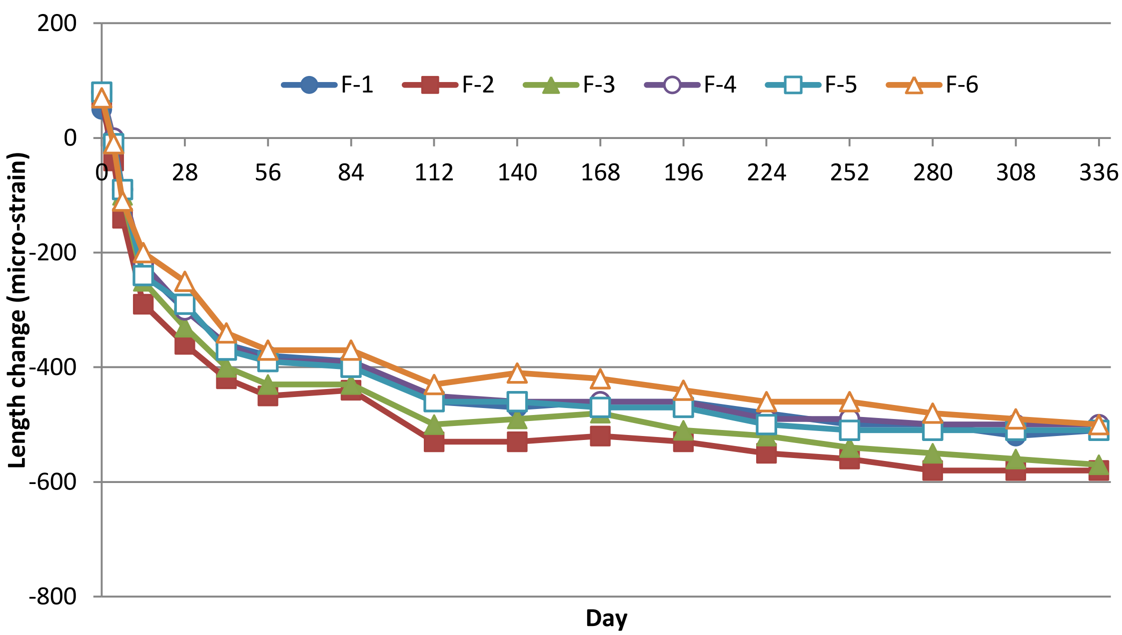


Figure F.6 Shrinkage - Mix F

Table F.7 Average Shrinkage Summary

	Initial	End curing	Air drying (days after moist curing)						
Day	1-day	0	4	7	14	28	42	56	84
A (Control)	0	82	-138	-274	-398	-510	-540	-573	-598
B	0	113	-150	-240	-347	-442	-485	-535	-537
C	0	78	-163	-245	-392	-482	-500	-522	-547
D	0	107	-23	-120	-207	-278	-310	-363	-382
E	0	58	-77	-113	-260	-342	-397	-417	-428
F	0	72	-13	-103	-238	-305	-375	-402	-403

Note: Units = microstrain

Day	112	140	168	196	224	252	280	308	336
A (Control)	-606	-658	-686	-690	-716	-732	-734	-738	-736
B	-547	-567	-595	-628	-615	-635	-643	-653	-655
C	-575	-603	-630	-672	-617	-650	-678	-672	-673
D	-448	-408	-443	-443	-458	-492	-517	-518	-522
E	-480	-473	-463	-487	-520	-530	-545	-552	-555
F	-472	-470	-468	-478	-500	-510	-520	-527	-528

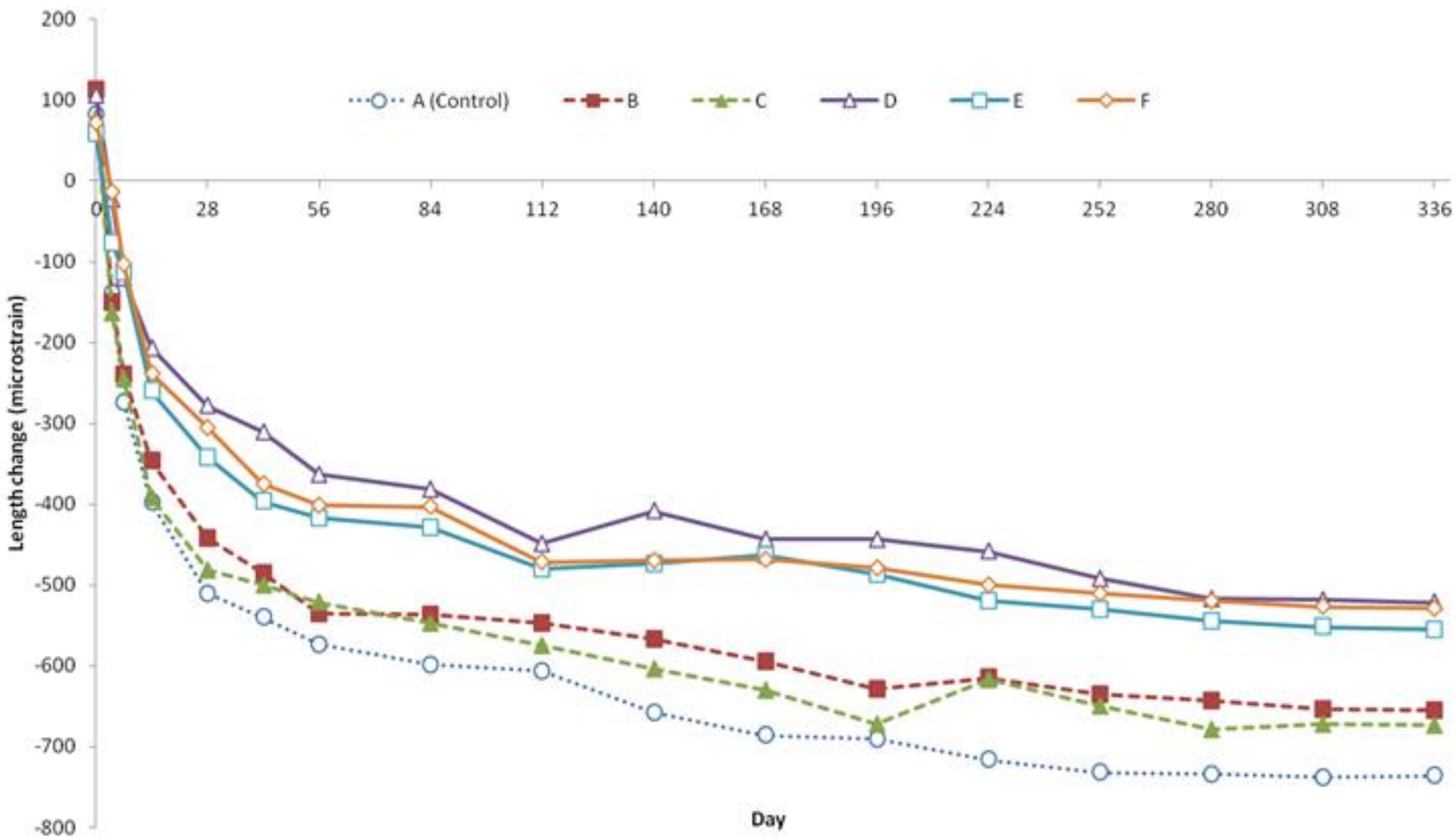


Figure F.7 Shrinkage Summary

Appendix G   Modulus of Elasticity and Poisson's Ratio Results

Table G.1 Modulus of Elasticity and Poisson's Ratio Results – Headed Bar Pullout

Specimen	$g_{long.\ initial}$ (in)	$g_{tran.\ initial}$ (in)	F_1 (lb)	$g_{tran.1}$ (in)	F_2 (lb)	$g_{long.2}$ (in)	$g_{tran.2}$ (in)	E (psi)	μ
Date cast: 9/15/2017; Date tested: 10/13/2017									
D-2A	0.0544	0.0506	3,840	0.0506	44,110	0.0631	0.0493	4,444,338	0.190
D-2B	0.0545	0.0314	3,510	0.0314	44,140	0.0634	0.0301	4,376,568	0.193
D-3A	0.0329	0.1112	3,490	0.1112	39,130	0.0409	0.1100	4,306,558	0.192
D-3B	0.0330	0.0915	3,330	0.0915	39,160	0.0413	0.0902	4,161,758	0.200
							Avg.	4,322,306	0.194
							Std. Dev.	120,913	0.004
Date cast: 7/31/2017; Date tested: 10/13/2017									
PC-1A	0.0417	0.1012	2,790	0.1012	25,800	0.0481	0.1003	3,491,588	0.182
PC-1B	0.0424	0.0820	2,780	0.0820	25,770	0.0491	0.0808	3,318,170	0.222*
PC-2A	0.0342	0.0615	2,810	0.0615	27,690	0.0411	0.0611	3,504,815	0.075*
PC-2B	0.0347	0.0433	2,840	0.0433	27,980	0.0419	0.0422	3,381,495	0.197
PC-3A	0.0548	0.1119	2,420	0.1119	25,210	0.0618	0.1110	3,129,873	0.165
PC-3B	0.0557	0.0824	2,430	0.0824	25,660	0.0632	0.0816	2,960,532	0.136
							Avg.	3,297,745	0.170
							Std. Dev.	214,373	0.026

*Not included in average

Table G.2 Modulus of Elasticity and Poisson's Ratio Results – LB-1, LB-2

Specimen	$g_{long.\ initial}$ (in)	$g_{tran.\ initial}$ (in)	F_1 (lb)	$g_{tran.1}$ (in)	F_2 (lb)	$g_{long.2}$ (in)	$g_{tran.2}$ (in)	E (psi)	μ
Date cast: 10/20/2017; Date tested: 11/27/2017									
D-1B	0.07340	0.02160	3,460	0.02160	40,810	0.08160	0.02050	4,395,077	0.172
D-2A	0.03910	0.08000	3,510	0.08000	42,010	0.04710	0.07920	4,665,052	0.128
D-2B	0.03920	0.03860	3,010	0.03860	41,900	0.04760	0.03750	4,471,958	0.167
D-3A	0.04950	0.06130	2,890	0.06130	40,770	0.05800	0.06060	4,287,998	0.105*
D-3B	0.04980	0.04210	3,180	0.04210	40,580	0.05840	0.04130	4,181,021	0.119
							Avg.	4,400,221	0.146
							Std. Dev.	184,357	0.027
Date cast: 7/31/2017; Date tested: 11/27/2017									
PC-2A	0.05840	0.06150	2,390	0.06150	25,800	0.06540	0.06070	3,223,867	0.147
PC-2B	0.05910	0.04310	2,450	0.04310	25,480	0.06620	0.04200	3,123,060	0.199
PC-3A	0.04930	0.06160	2,030	0.06160	24,110	0.05610	0.06070	3,138,129	0.171
PC-3B	0.04940	0.04170	2,130	0.04170	24,090	0.05630	0.04110	3,071,865	0.112
PC-4A	0.05620	0.04000	2,520	0.04000	24,930	0.06270	0.03960	3,333,316	0.080*
PC-4B	0.05640	0.02590	2,640	0.02590	25,190	0.06300	0.02540	3,298,630	0.098*
							Avg.	3,198,144	0.157
							Std. Dev.	104,131	0.037

*Not included in average

Table G.3 Modulus of Elasticity and Poisson's Ratio Results – LB-3, LB-4

Specimen	$g_{long.\ initial}$ (in)	$g_{tran.\ initial}$ (in)	F_1 (lb)	$g_{tran.1}$ (in)	F_2 (lb)	$g_{long.2}$ (in)	$g_{tran.2}$ (in)	E (psi)	μ
Date cast: 11/1/2017; Date tested: 11/29/2017									
D-1B	0.05590	0.03150	3,420	0.03150	41,990	0.06420	0.03020	4,465,396	0.200
D-1C	0.05620	0.01190	3,090	0.01190	42,460	0.06490	0.01060	4,334,100	0.190
D-2A	0.03970	0.06180	3,350	0.06180	41,330	0.04770	0.06080	4,592,779	0.160
D-2B	0.03970	0.03180	3,380	0.03180	41,220	0.04790	0.03060	4,456,101	0.187
D-3A	0.07010	0.07190	3,360	0.07180	43,100	0.07840	0.07050	4,608,954	0.200
D-3B	0.05900	0.01250	3,080	0.01250	43,180	0.06750	0.01120	4,533,595	0.195
							Avg.	4,498,488	0.189
							Std. Dev.	102,217	0.015
Date cast: 7/31/2017; Date tested: 11/29/2017									
PC-2A	0.03550	0.07760	2,220	0.07760	24,060	0.04190	0.07680	3,334,850	0.163
PC-2B	0.03560	0.05920	2,400	0.05920	24,030	0.04200	0.05860	3,302,784	0.122
PC-3A	0.03830	0.06150	2,260	0.06150	24,440	0.04490	0.06070	3,250,180	0.157
PC-3B	0.03840	0.04210	2,140	0.04210	24,160	0.04500	0.04170	3,226,734	0.078*
PC-4A	0.09350	0.05110	2,260	0.05110	25,550	0.10070	0.05020	3,130,301	0.161
PC-4B	0.09360	0.03110	2,280	0.03110	25,480	0.10080	0.03040	3,110,401	0.125
							Avg.	3,225,875	0.145
							Std. Dev.	90,389	0.020

*Not included in average

Table G.4 Modulus of Elasticity and Poisson's Ratio Results – LB-5, LB-6

Specimen	$g_{long.\ initial}$ (in)	$g_{tran.\ initial}$ (in)	F_1 (lb)	$g_{tran.1}$ (in)	F_2 (lb)	$g_{long.2}$ (in)	$g_{tran.2}$ (in)	E (psi)	μ
Date cast: 11/3/2017; Date tested: 12/1/2017									
D-1A	0.05980	0.06220	3,330	0.06220	43,450	0.06830	0.06100	4,523,335	0.180
D-1B	0.06000	0.04050	3,310	0.04050	43,410	0.06860	0.03920	4,464,865	0.192
D-2A	0.06180	0.06150	3,420	0.06150	40,700	0.07010	0.06030	4,330,180	0.185
D-2B	0.06190	0.04100	3,220	0.04100	40,690	0.07030	0.03960	4,296,753	0.213
D-3A	0.04410	0.06200	3,170	0.06200	39,310	0.05240	0.06080	4,210,472	0.185
D-3B	0.04420	0.04160	3,000	0.04160	39,060	0.05260	0.04040	4,147,582	0.183
							Avg.	4,328,865	0.185
							Std. Dev.	144,380	0.005
Date cast: 7/31/2017; Date tested: 12/1/2017									
PC-2A	0.05380	0.08190	2,140	0.08180	25,760	0.06120	0.08090	3,061,867	0.156
PC-2B	0.05400	0.04180	2,210	0.04180	25,190	0.06140	0.04060	2,978,904	0.208
PC-3A	0.03260	0.09120	2,510	0.09120	24,090	0.03880	0.09050	3,411,948	0.147
PC-3B	0.03260	0.06240	2,380	0.06240	24,130	0.03920	0.06170	3,211,181	0.137
PC-4A	0.04010	0.08100	2,350	0.08100	25,990	0.04710	0.07960	3,271,878	0.258*
PC-4B	0.04030	0.04170	2,450	0.04170	26,030	0.04730	0.04040	3,263,574	0.239*
							Avg.	3,199,892	0.162
							Std. Dev.	156,345	0.031

*Not included in average

Appendix H Interface Bond Results

Table H.1 Interface Bond Results – Set 1 (without Bonding Agent)

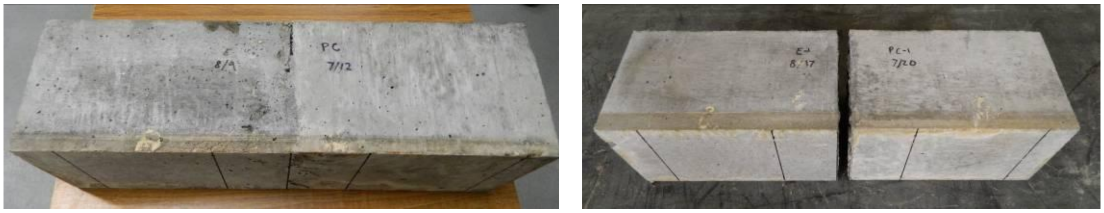
Sample	W_1 (in)	W_2 (in)	W_3 (in)	D_1 (in)	D_2 (in)	D_3 (in)	Max Load (lb)	Strength (psi)
Date Cast: Precast - 7/12/2017, Closures - 8/9/2017; Date Tested: 9/6/2017								
D-1	6.055	6.016	6.019	6.012	6.015	6.049	6,710	552
D-2	5.958	5.971	5.935	6.077	6.067	6.088	8,600	704
D-3	6.028	6.032	6.027	6.042	6.074	6.119	8,040	650
D-4	6.029	6.042	6.063	6.092	6.152	6.202	6,890	543
							Average	612
							Std. Dev.	78
E-1	6.070	6.103	6.107	6.049	6.082	6.125	8,070	644
E-2	6.018	6.041	6.031	6.063	6.056	6.089	6,650	539
E-3	6.015	6.026	6.013	6.019	6.053	6.085	6,320	516
E-4	6.041	6.081	6.061	6.045	6.080	6.127	6,800	546
							Average	561
							Std. Dev.	57

Table H.2 Interface Bond Results – Set 2 (with Bonding Agent)

Sample	W_1 (in)	W_2 (in)	W_3 (in)	D_1 (in)	D_2 (in)	D_3 (in)	Max Load (lb)	Strength (psi)
Date Cast: Precast - 7/20/2017, Closures - 8/17/2017; Date Tested: 9/14/2017								
D-1(BG)	6.111	6.113	6.081	6.051	6.092	6.137	5,880	467
D-2(BG)	6.079	6.051	6.056	6.093	6.142	6.197	5,100	401
D-3(BG)	6.013	6.040	6.044	6.007	6.040	6.087	5,630	460
D-4(BG)	5.978	6.017	6.010	6.081	6.090	6.134	5,160	416
							Average	436
							Std. Dev.	32
E-1(BG)	6.060	6.104	6.118	6.032	6.061	6.110	4,900	393
E-2(BG)	6.104	6.122	6.124	6.047	6.077	6.131	4,950	393
E-3(BG)	6.140	6.049	6.005	6.055	6.067	6.091	4,680	377
E-4(BG)	6.040	6.096	6.112	6.019	6.059	6.105	4,720	380
							Average	386
							Std. Dev.	9

Table H.3 Interface Bond Results – Fractured Aggregate

Specimen	Total Number of Aggregates	# Fractured	Percent Aggregate Fractured	Average
WITHOUT bonding agent				
D-1	121	11	9%	
D-2	115	13	11%	
D-3	108	15	14%	
D-4	132	12	9%	11%
E-1	127	8	6%	
E-2	123	7	6%	
E-3	125	10	8%	
E-4	135	7	5%	6%
WITH bonding agent				
D-1	102	11	11%	
D-2	100	9	9%	
D-3	94	8	9%	
D-4	93	9	10%	9%
E-1	99	7	7%	
E-2	92	7	8%	
E-3	98	6	6%	
E-4	101	6	6%	7%



E-1 with BG before and after testing

D-4 without BG

D-1 with BG

BG = Bonding Agent

Figure H.1 Interface Beam Specimen Failures

(a) Closure Concrete

(b) Precast Concrete

Figure H.2 Interface Failure – D-1 without BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.3 Interface Failure – D-2 without BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.4 Interface Failure – D-3 without BG


(a) Closure Concrete


(b) Precast Concrete

Figure H.5 Interface Failure – D-4 without BG


(a) Closure Concrete


(b) Precast Concrete

Figure H.6 Interface Failure – E-1 without BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.7 Interface Failure – E-2 without BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.8 Interface Failure – E-3 without BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.9 Interface Failure – E-4 without BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.10 Interface Failure – D-1 with BG

(a) Closure Concrete

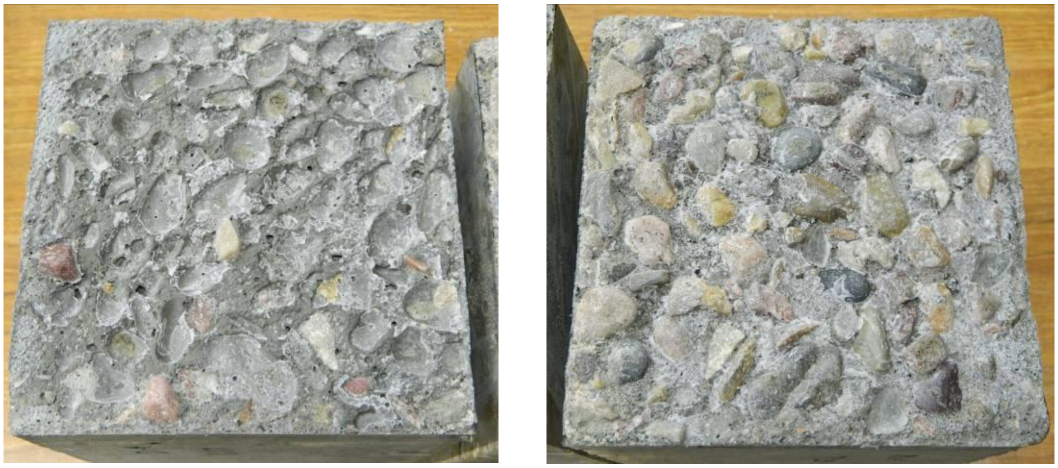
(b) Precast Concrete

Figure H.11 Interface Failure – D-2 with BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.12 Interface Failure – D-3 with BG



(a) Closure Concrete (b) Precast Concrete

Figure H.13 Interface Failure – D-4 with BG

(a) Closure Concrete (b) Precast Concrete

Figure H.14 Interface Failure – E-1 with BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.15 Interface Failure – E-2 with BG

(a) Closure Concrete

(b) Precast Concrete

Figure H.16 Interface Failure – E-3 with BG

(a) Closure Concrete (b) Precast Concrete

Figure H.17 Interface Failure – E-4 with BG

Appendix I Headed Bar Pullout Results

Table I.1 United Machine Force-Displacement Data – HB-1, HB-2, and HB-3

HB-1		HB-2		HB-3	
Machine Force (lb)	**Machine Displacement (in)**	**Machine Force (lb)**	**Machine Displacement (in)**	**Machine Force (lb)**	**Machine Displacement (in)**
0	0.0000	0	0.0000	0	0.0000
110	0.0094	545	0.0114	541	0.0112
536	0.0210	910	0.0197	919	0.0195
925	0.0293	1,612	0.0279	1,648	0.0278
1,645	0.0376	2,728	0.0362	2,862	0.0362
2,812	0.0459	4,044	0.0446	4,149	0.0445
3,762	0.0542	4,810	0.0528	5,422	0.0527
4,841	0.0624	5,987	0.0611	6,162	0.0610
5,981	0.0706	7,143	0.0693	7,393	0.0692
7,138	0.0789	8,249	0.0775	8,645	0.0774
8,274	0.0872	9,454	0.0859	9,911	0.0859
9,493	0.0955	10,635	0.0943	11,129	0.0941
10,739	0.1038	11,618	0.1025	12,230	0.1049
11,941	0.1121	9,487	0.1108	11,745	0.1107
13,120	0.1202	9,542	0.1191	12,238	0.1190
14,297	0.1286	2,839	0.1273	8,723	0.1291
911	0.1391				

Note: Data was extracted from video of test.

Table I.2 United Machine Force-Displacement Data – HB-4, HB-5, and HB-6

HB-4		HB-5		HB-6	
Machine Force (lb)	**Machine Displacement (in)**	**Machine Force (lb)**	**Machine Displacement (in)**	**Machine Force (lb)**	**Machine Displacement (in)**
0	0.0000	0	0.0000	0	0.0000
538	0.0287	668	0.0154	600	0.0256
939	0.0369	1,003	0.0233	950	0.0339
1,597	0.0453	1,753	0.0319	1,570	0.0422
2,741	0.0536	2,876	0.0403	2,725	0.0505
3,976	0.0618	4,105	0.0486	4,020	0.0588
5,171	0.0702	5,353	0.0569	5,348	0.0671
6,453	0.0785	6,127	0.0650	6,602	0.0752
7,055	0.0866	7,329	0.0733	7,640	0.0818
8,306	0.0949	8,560	0.0816	8,345	0.0918
9,552	0.1032	9,784	0.0899	9,610	0.1001
10,759	0.1115	10,940	0.0983	10,817	0.1084
11,992	0.1198	11,984	0.1064	12,022	0.1167
13,148	0.1280	11,963	0.1143	12,835	0.1242
10,044	0.1364	5,826	0.1252	11,308	0.1334
				11,637	0.1417
				10,018	0.1495

Note: Data was extracted from video of test.

Force vs. Rebar Strain

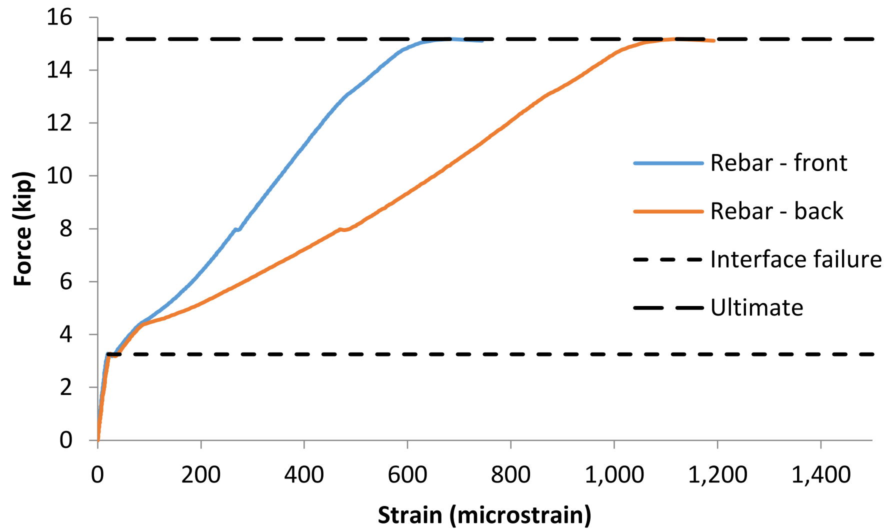


Figure I.1 Force vs. Rebar Strain – HB-1

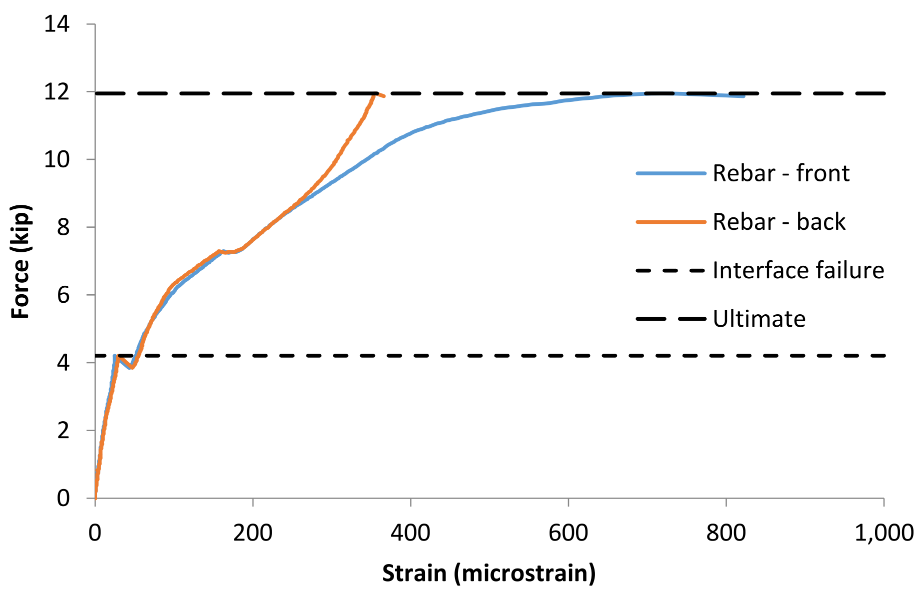


Figure I.2 Force vs. Rebar Strain – HB-2

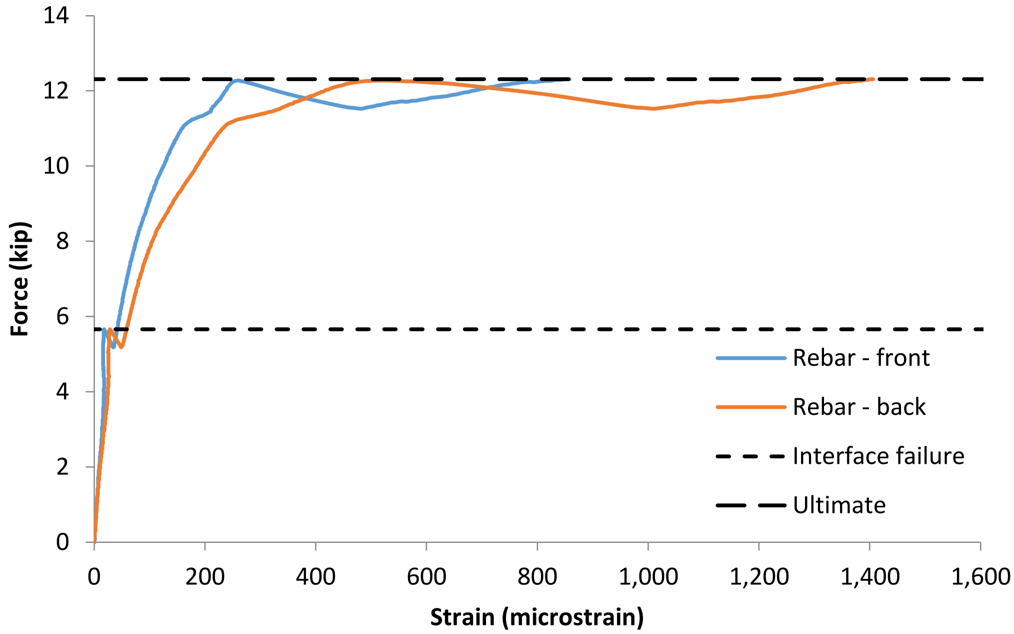


Figure I.3 Force vs. Rebar Strain – HB-3

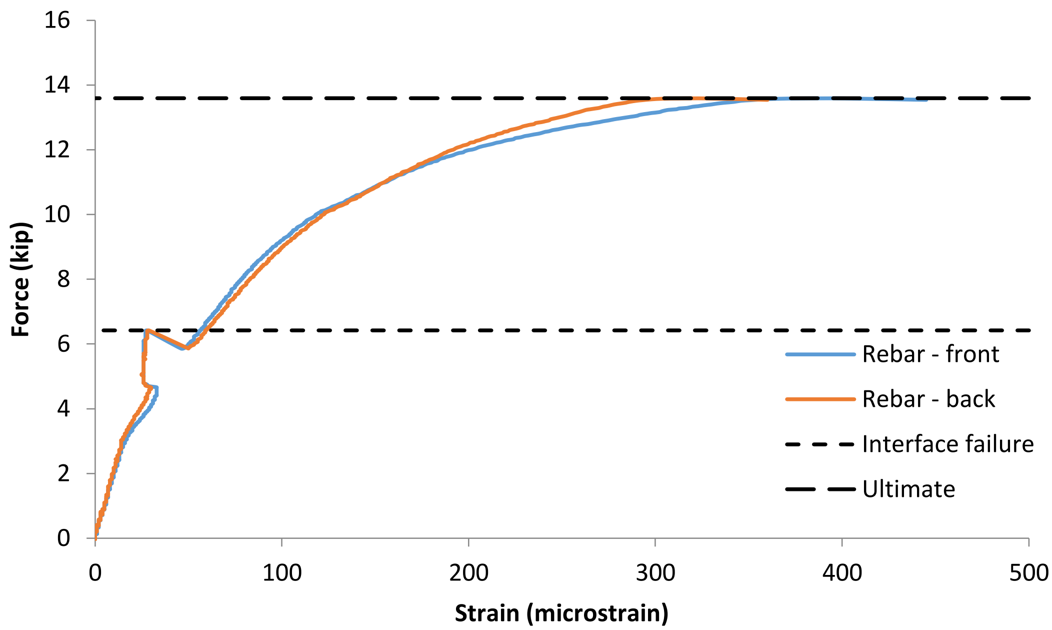


Figure I.4 Force vs. Rebar Strain – HB-4

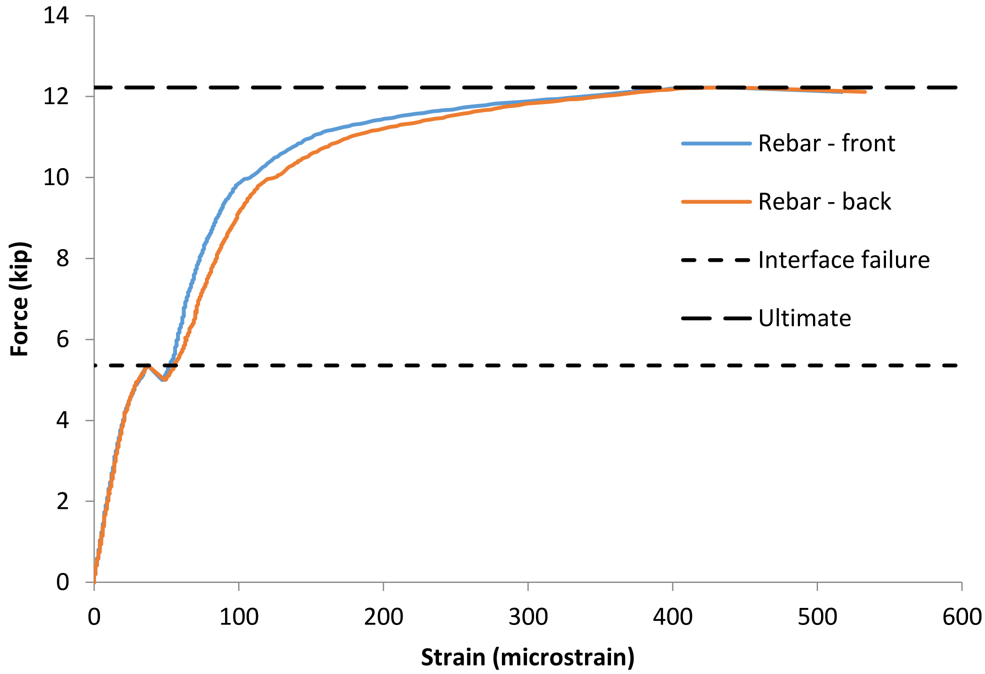


Figure I.5 Force vs. Rebar Strain – HB-5

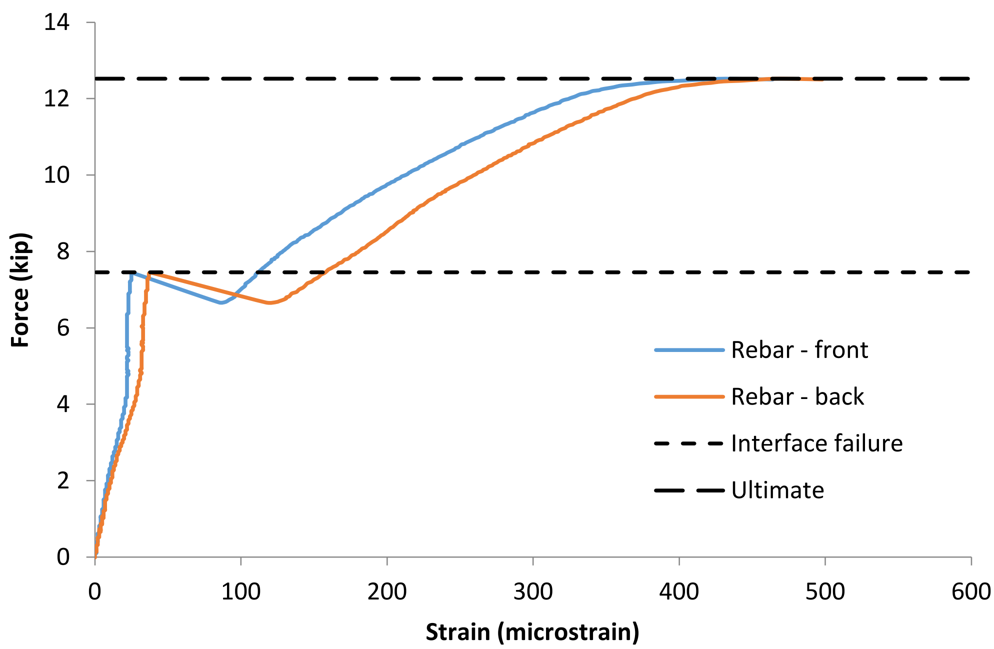


Figure I.6 Force vs. Rebar Strain – HB-6

Force vs. Crack Expansion

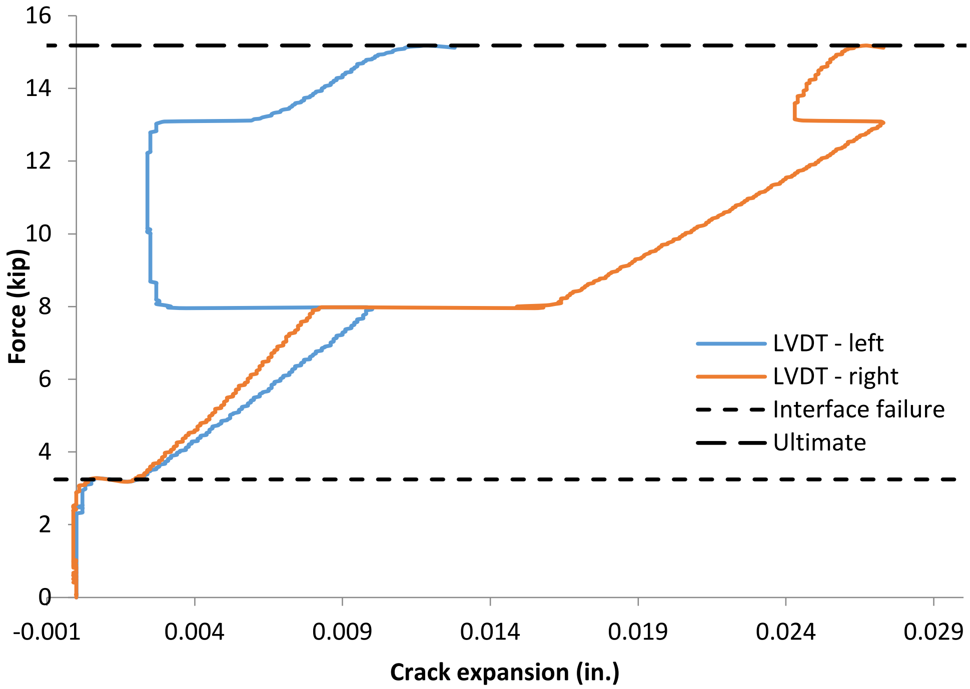


Figure I.7 Force vs. Crack Expansion – HB-1

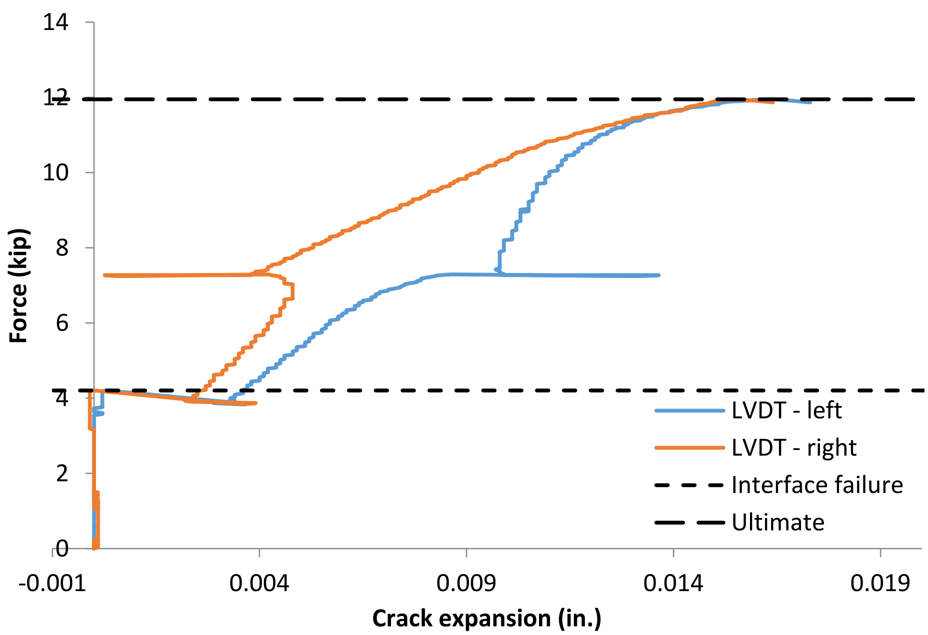


Figure I.8 Force vs. Crack Expansion – HB-2

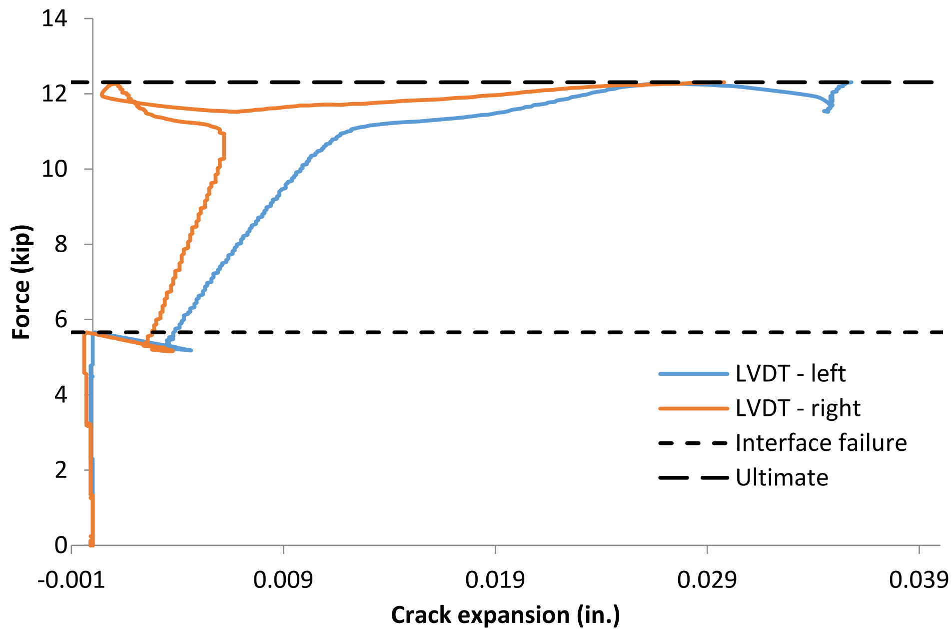


Figure I.9 Force vs. Crack Expansion – HB-3

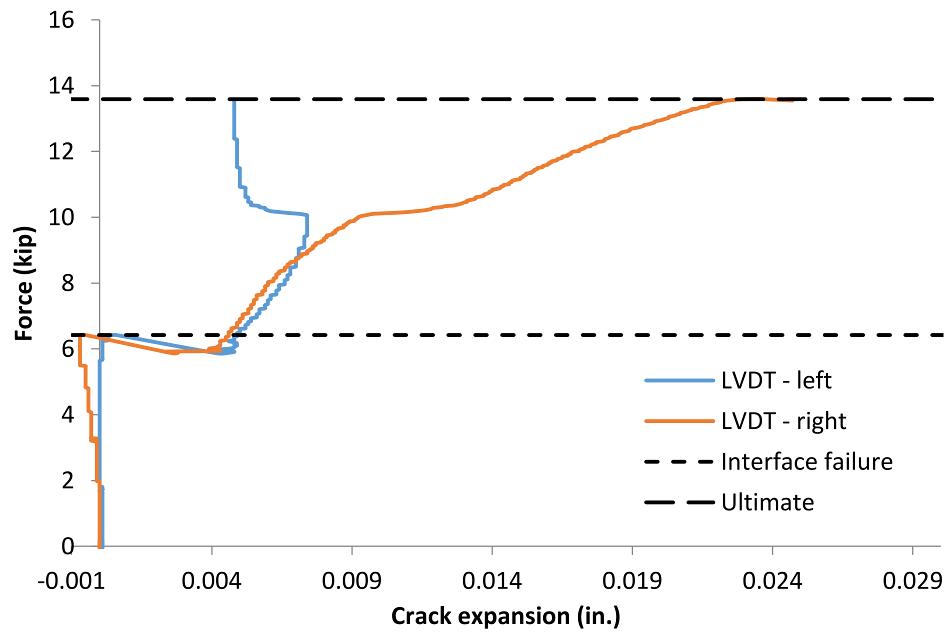


Figure I.10 Force vs. Crack Expansion – HB-4

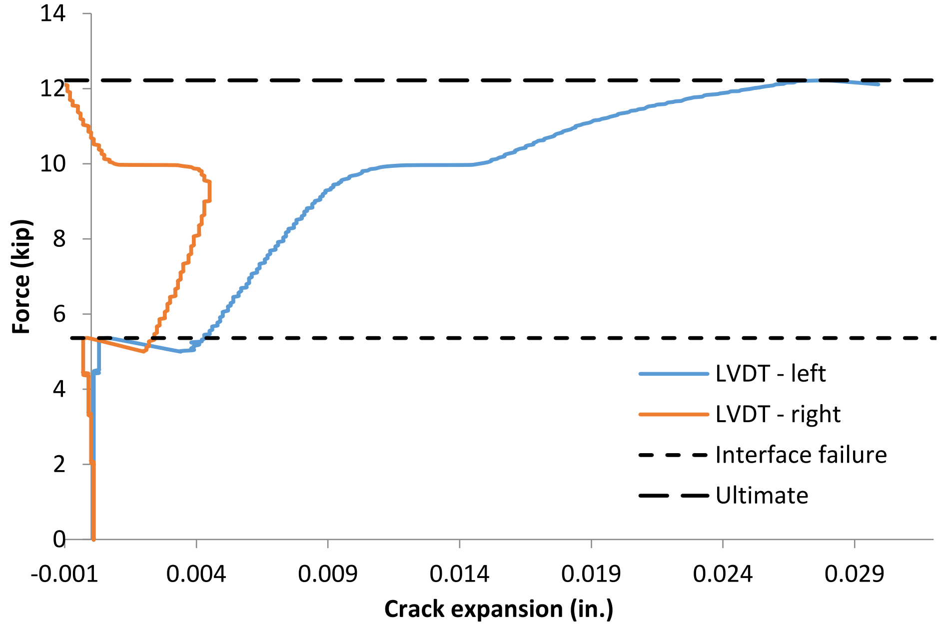


Figure I.11 Force vs. Crack Expansion – HB-5

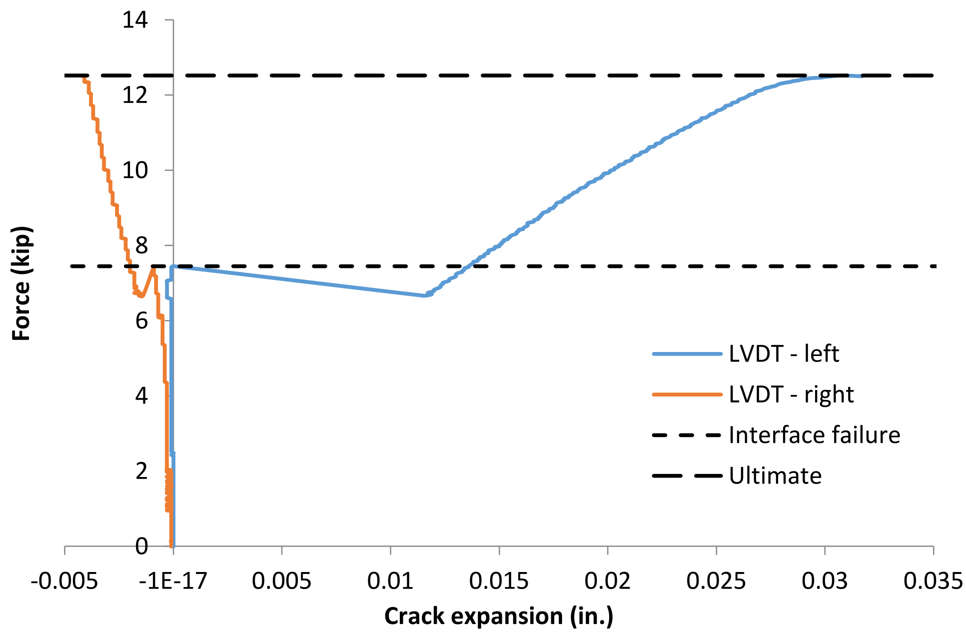


Figure I.12 Force vs. Crack Expansion – HB-6

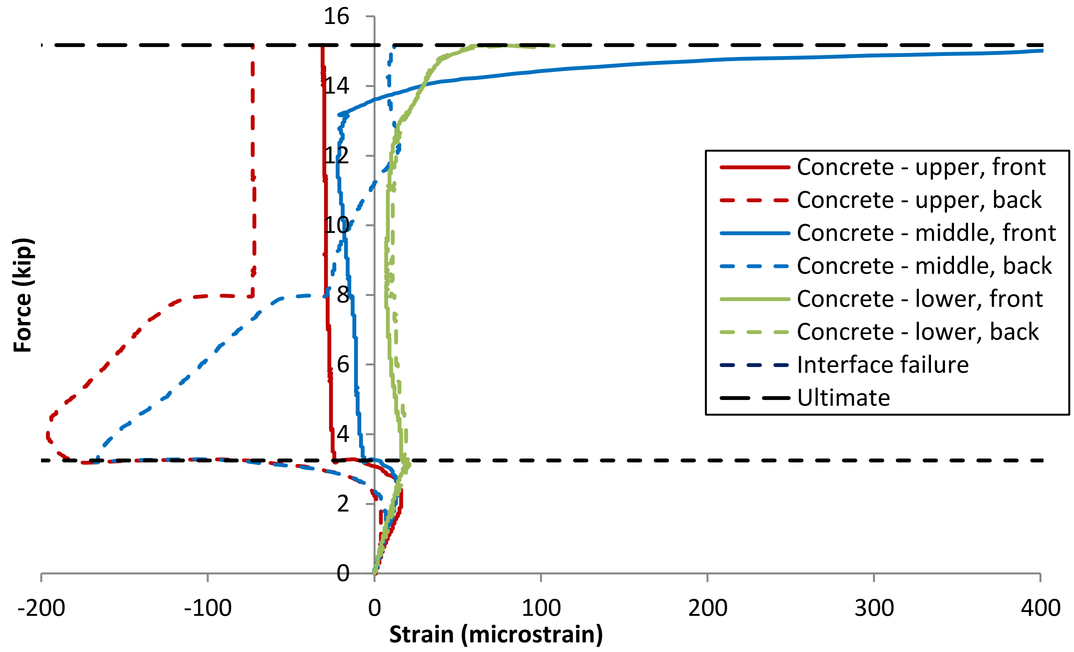


Figure I.13 Force vs. Concrete Strain – HB-1

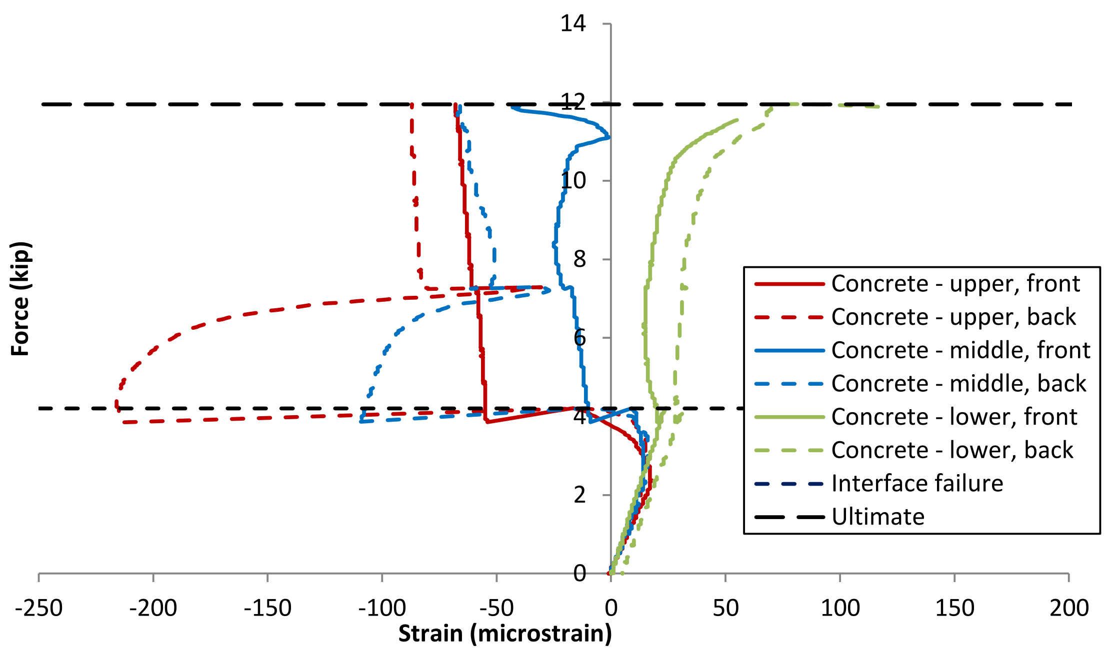


Figure I.14 Force vs. Concrete Strain – HB-2

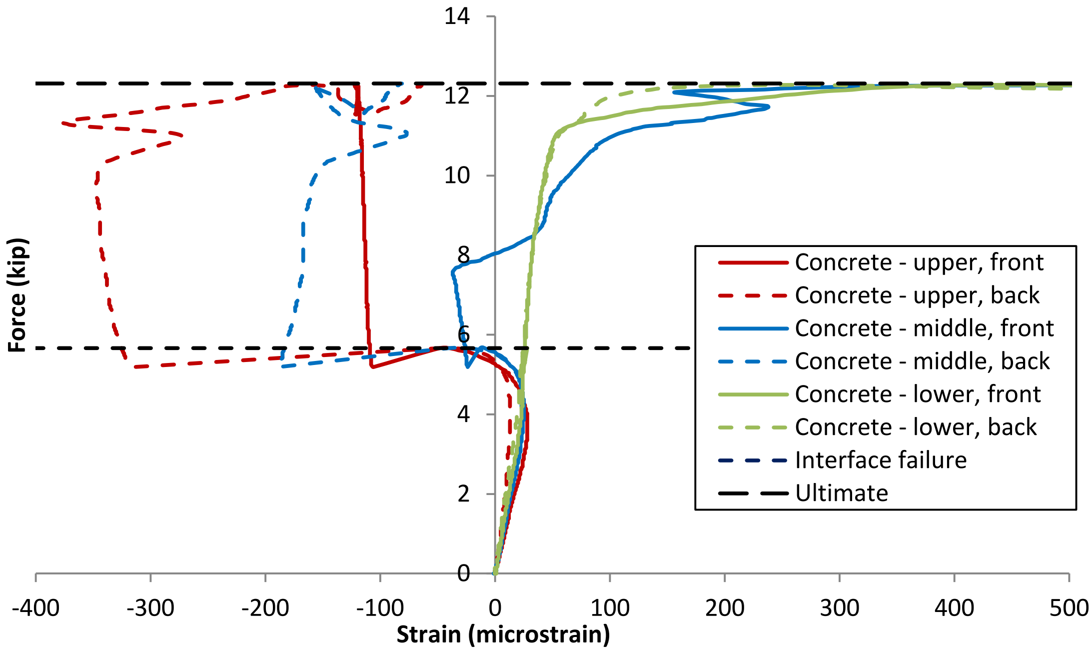


Figure I.15 Force vs. Concrete Strain – HB-3

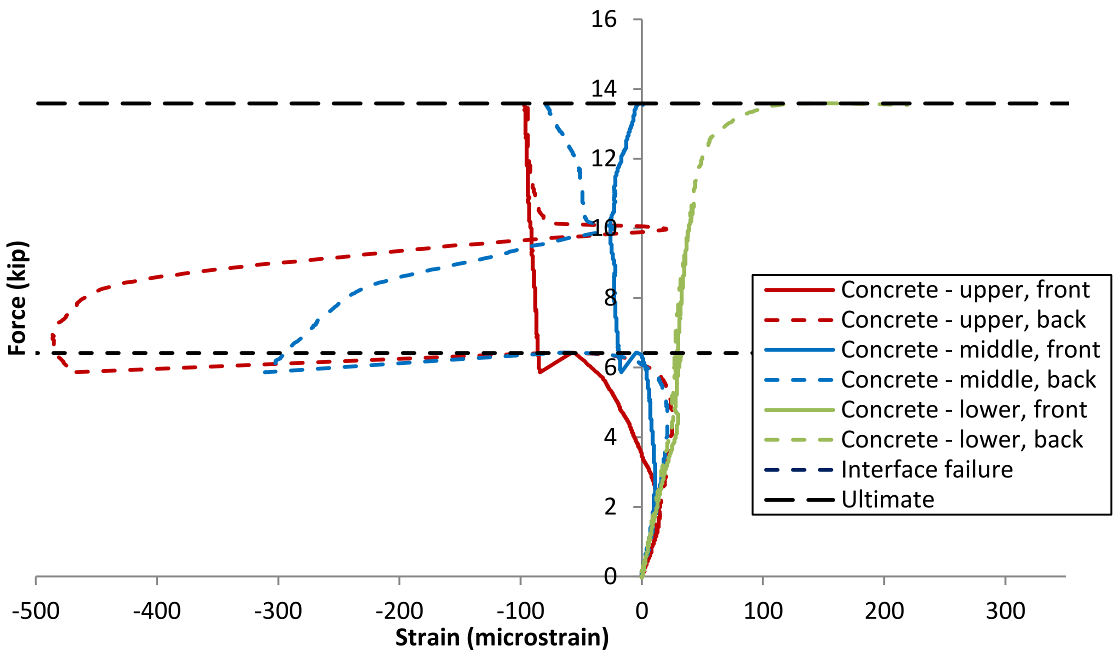


Figure I.16 Force vs. Concrete Strain – HB-4

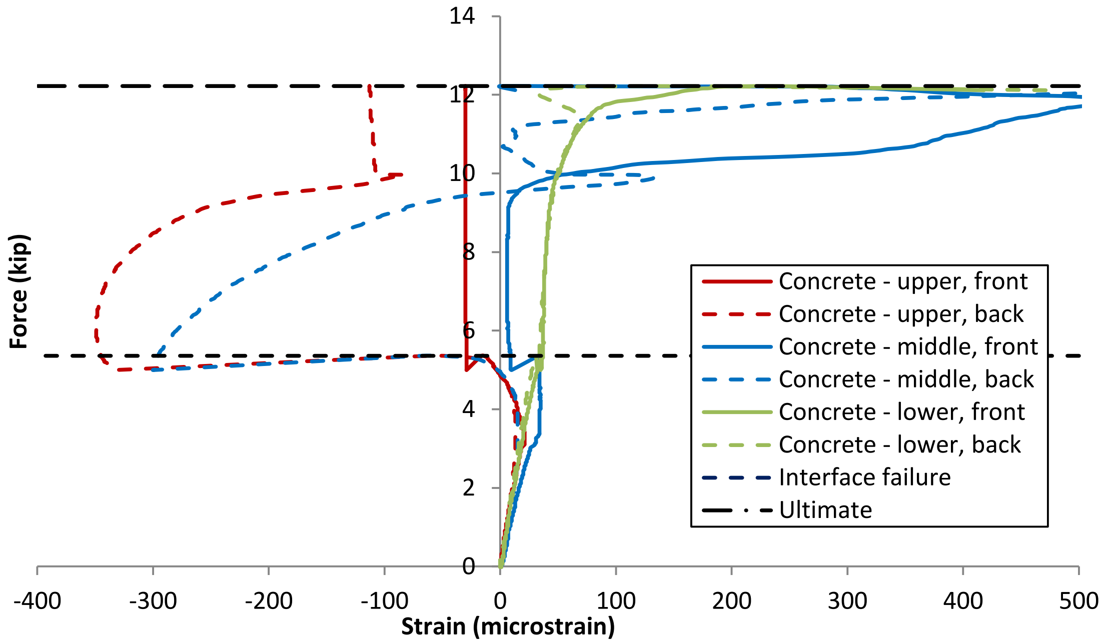


Figure I.17 Force vs. Concrete Strain – HB-5

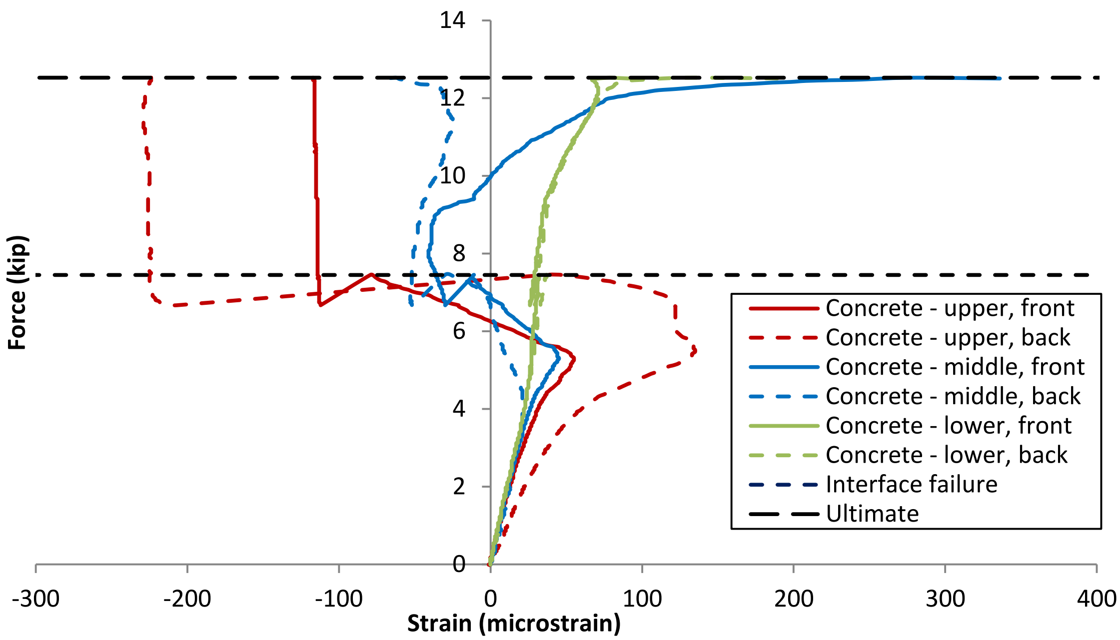
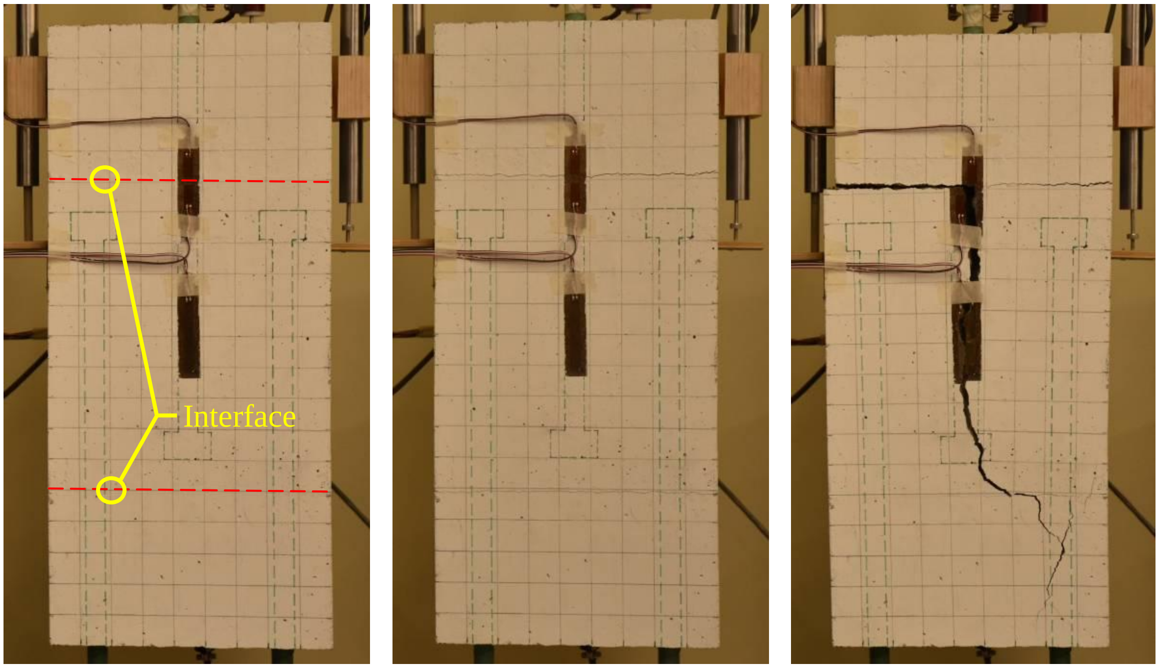
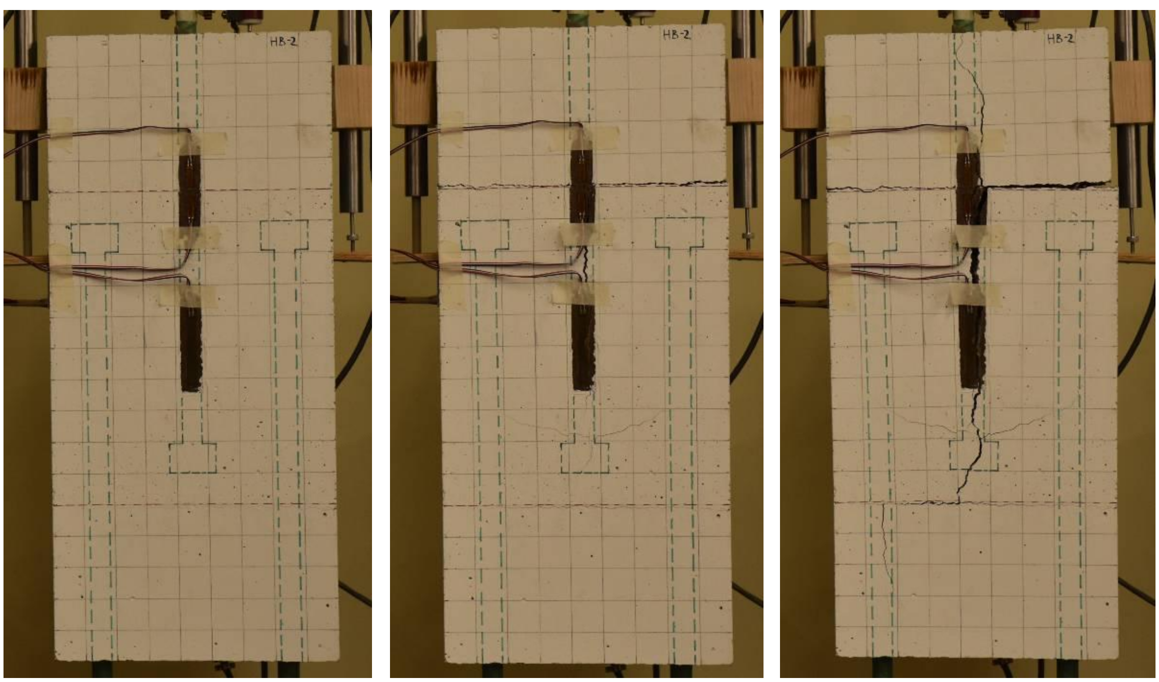


Figure I.18 Force vs. Concrete Strain – HB-6



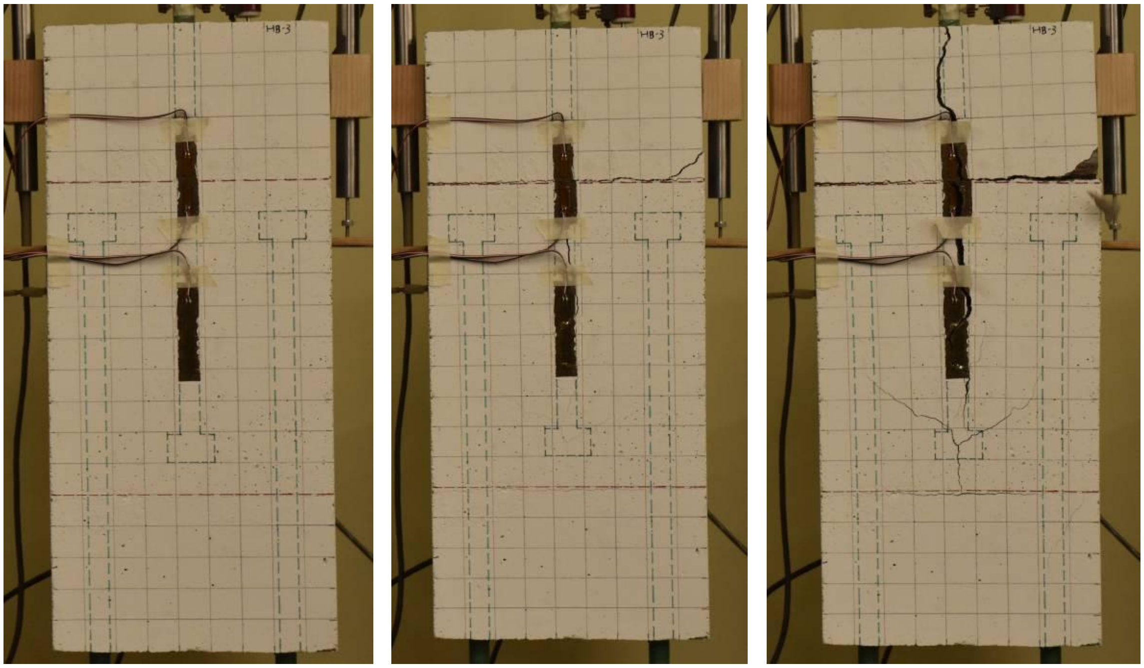
(a) At start of test (b) Instant before failure (c) Failure

Figure I.19 Pullout Specimen Before and After Failure – HB-1



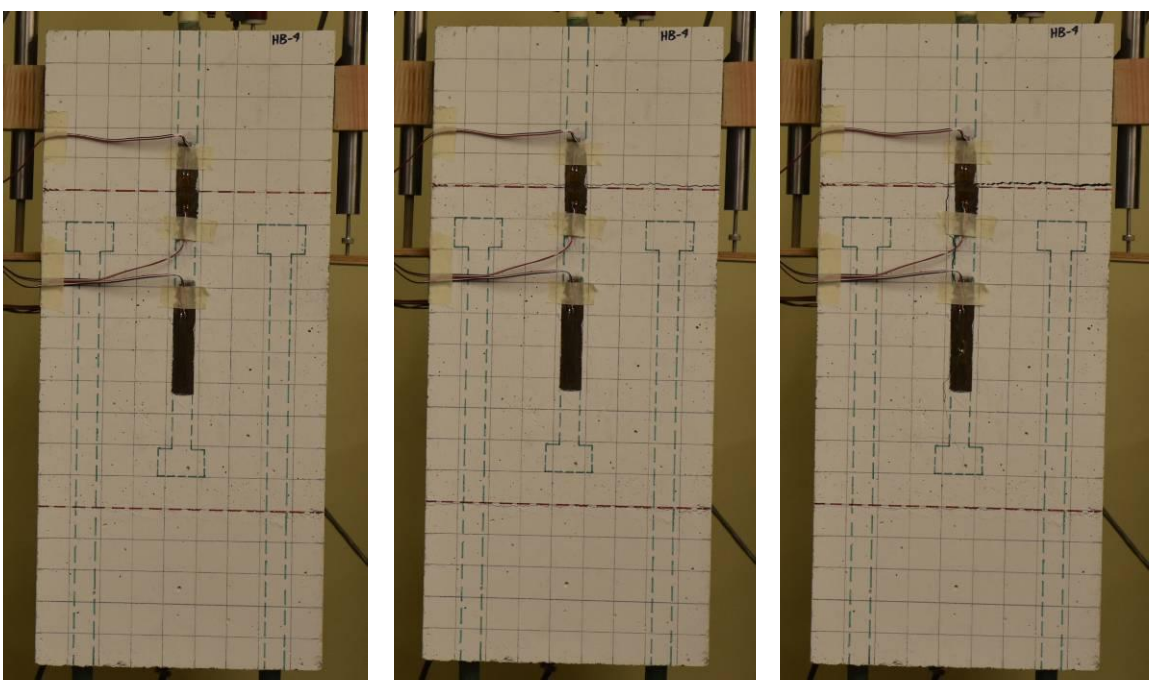
(a) At start of test (b) Instant before failure (c) Failure

Figure I.20 Pullout Specimen Before and After Failure – HB-2



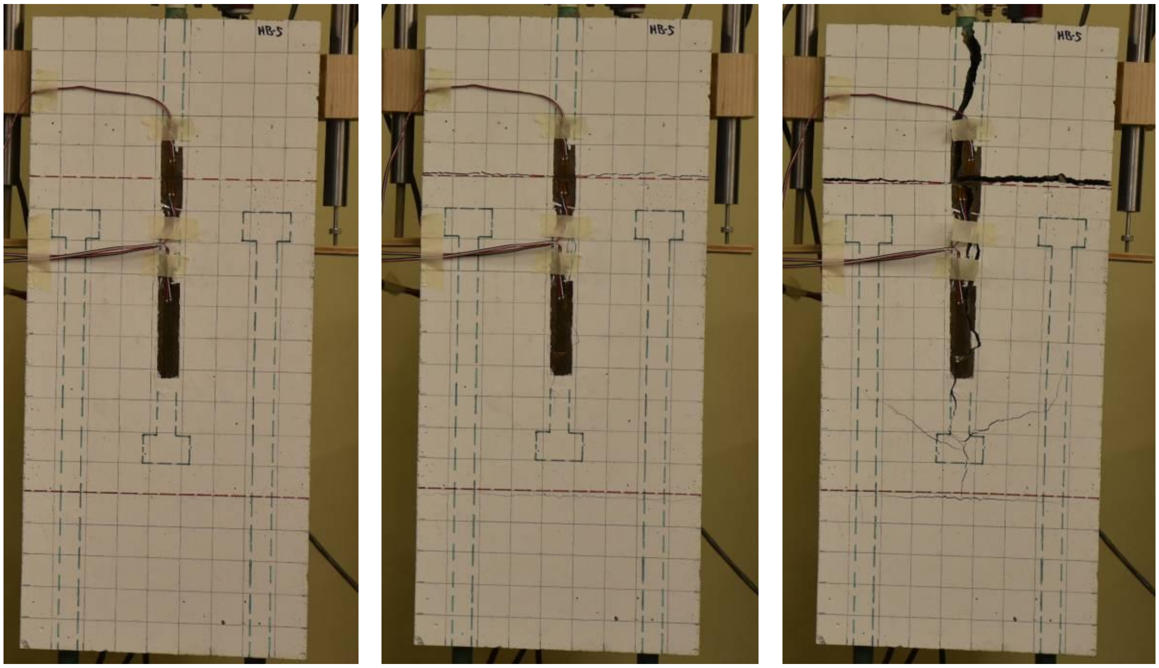
(a) At start of test (b) Instant before failure (c) Failure

Figure I.21 Pullout Specimen Before and After Failure – HB-3



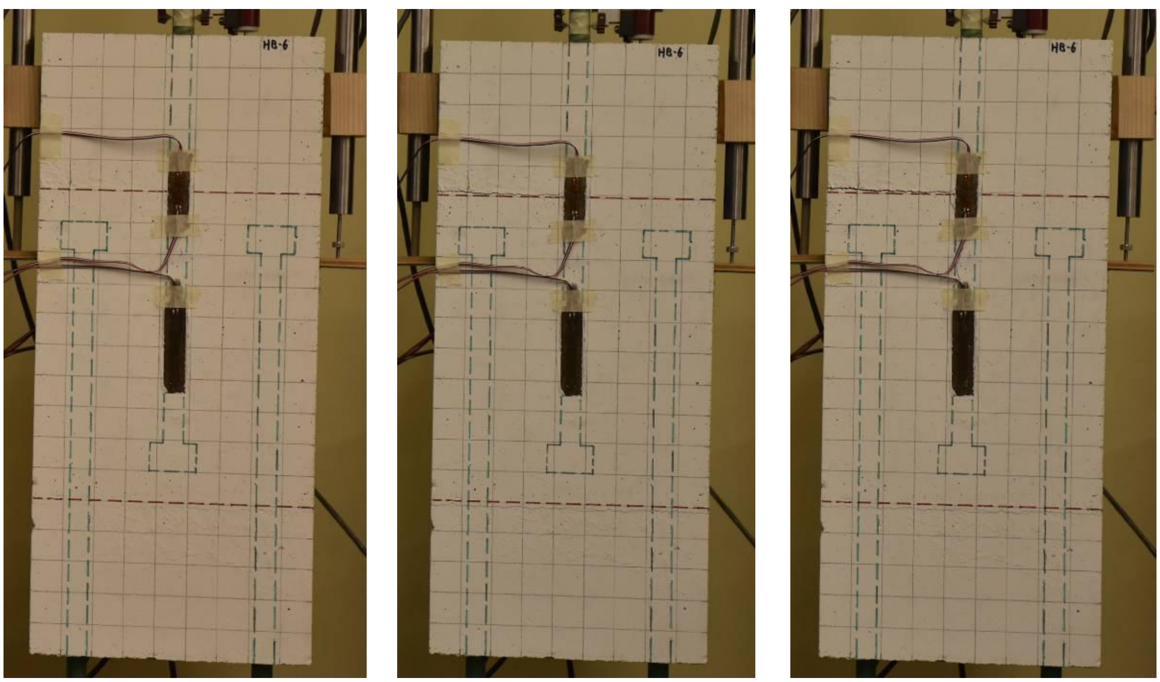
(a) At start of test (b) Instant before failure (c) Failure

Figure I.22 Pullout Specimen Before and After Failure – HB-4



(a) At start of test (b) Instant before failure (c) Failure

Figure I.23 Pullout Specimen Before and After Failure – HB-5



(a) At start of test (b) Instant before failure (c) Failure

Figure I.24 Pullout Specimen Before and After Failure – HB-6

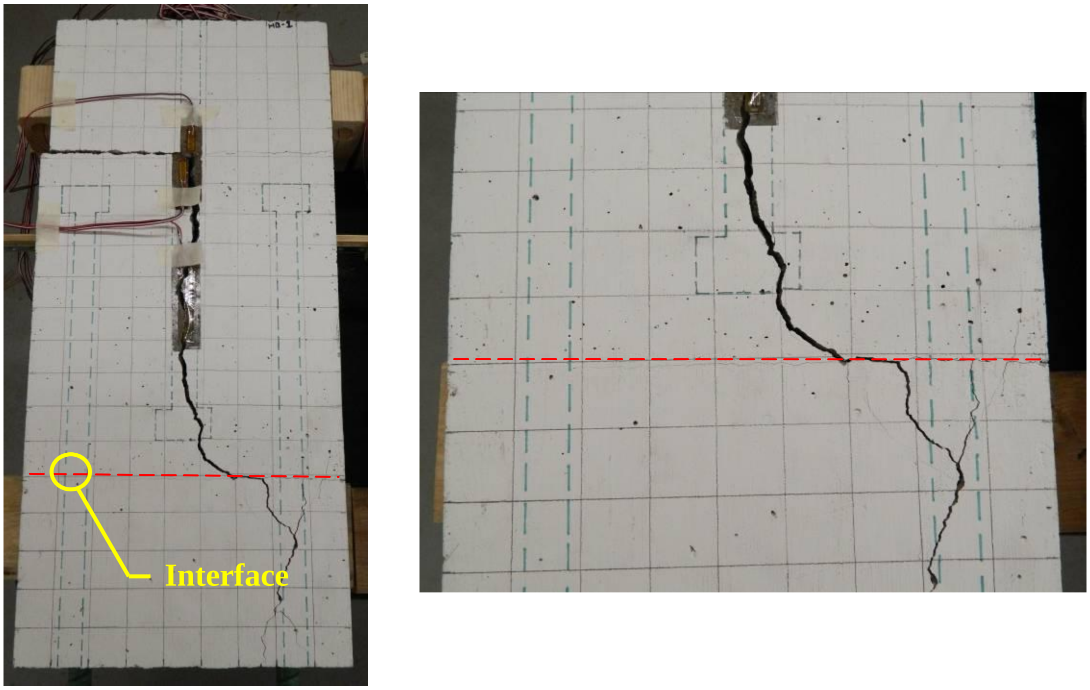


Figure I.25 Pullout Specimen Cracks – HB-1

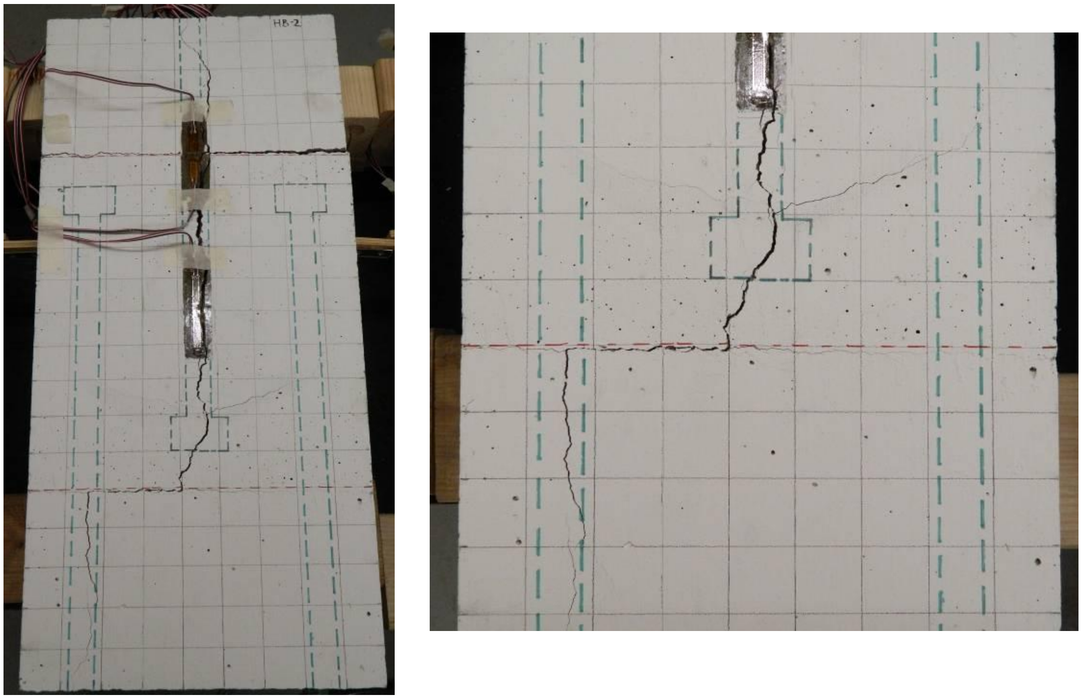


Figure I.26 Pullout Specimen Cracks – HB-2

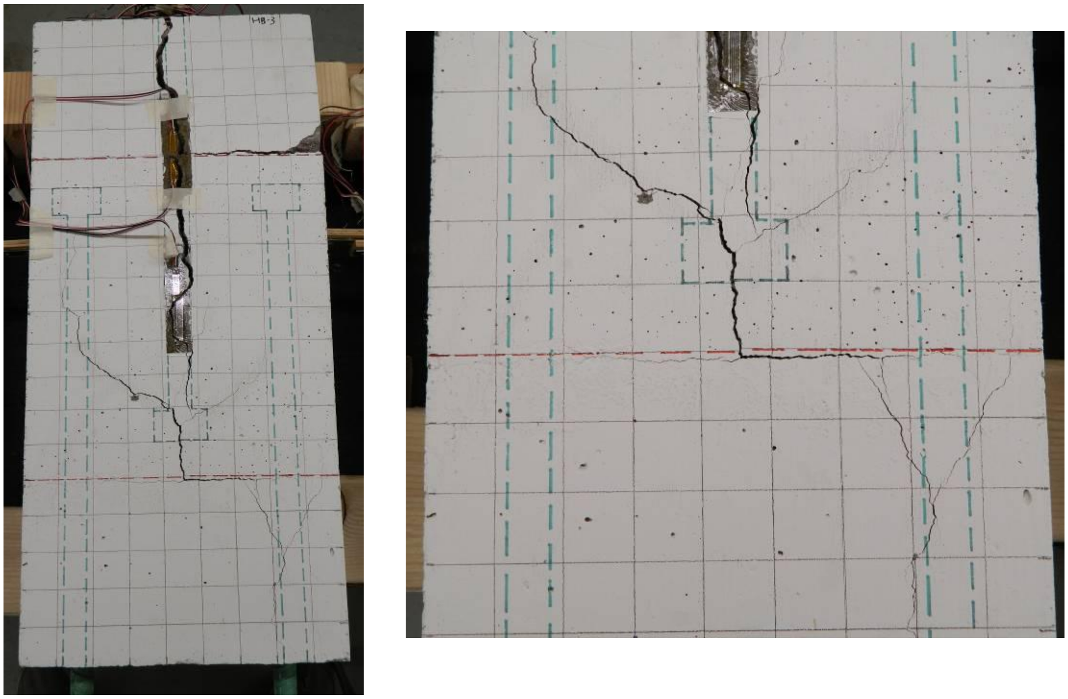


Figure I.27 Pullout Specimen Cracks – HB-3

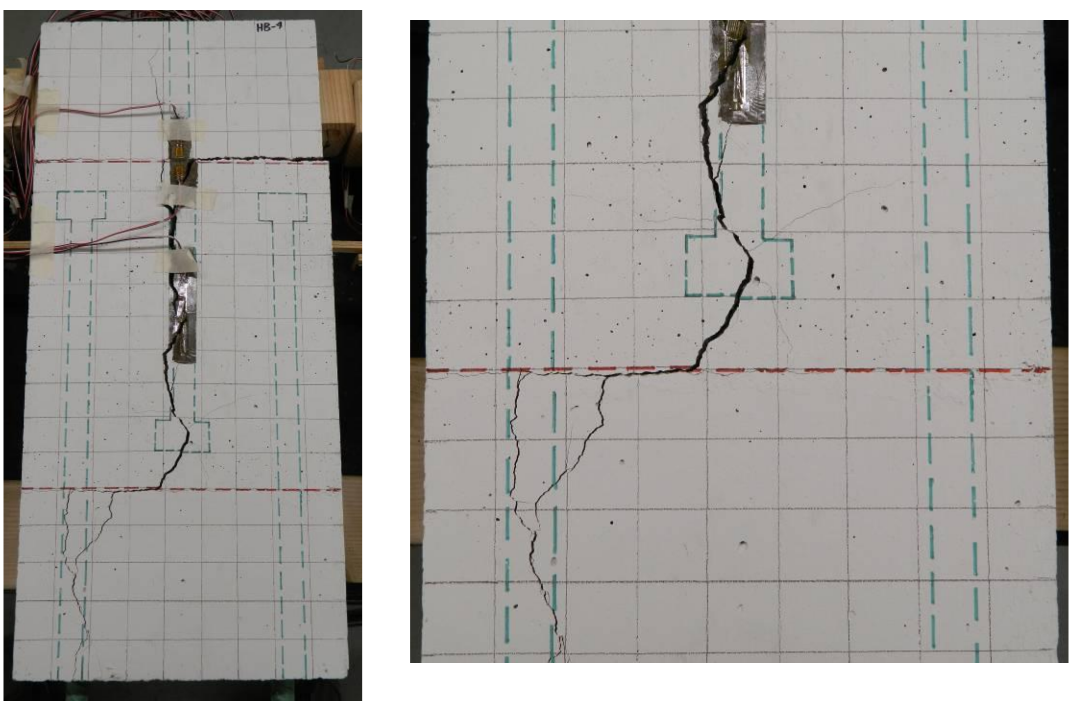


Figure I.28 Pullout Specimen Cracks – HB-4

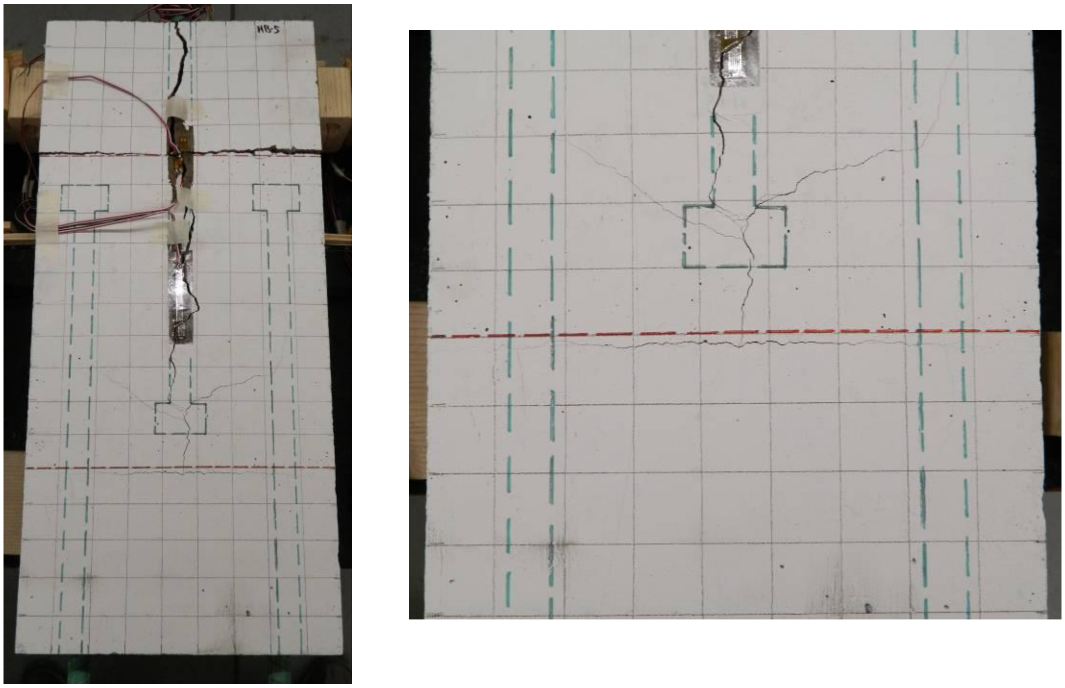


Figure I.29 Pullout Specimen Cracks – HB-5

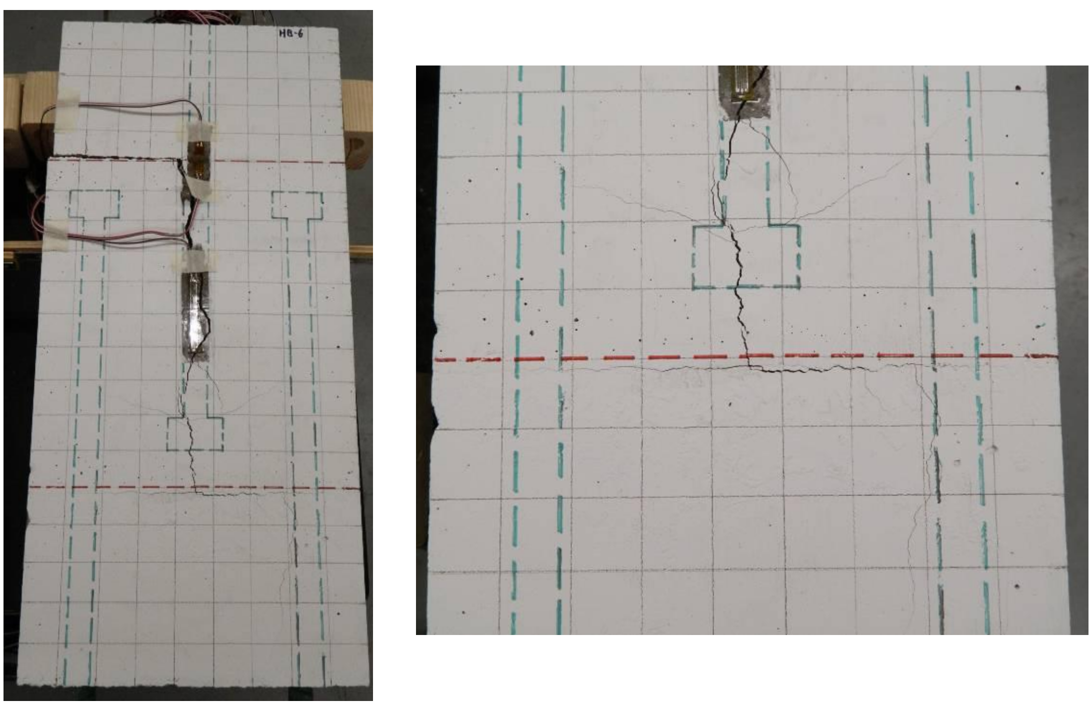


Figure I.30 Pullout Specimen Cracks – HB-6

Appendix J Beam Results

Force vs. Displacement

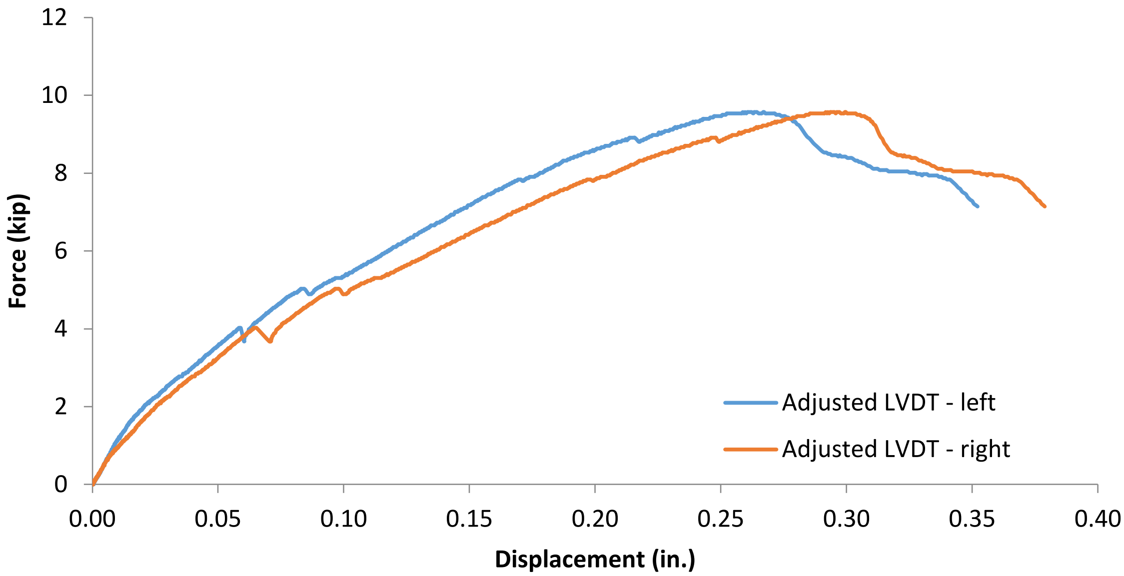


Figure J.1 Force vs. Displacement – LB-1

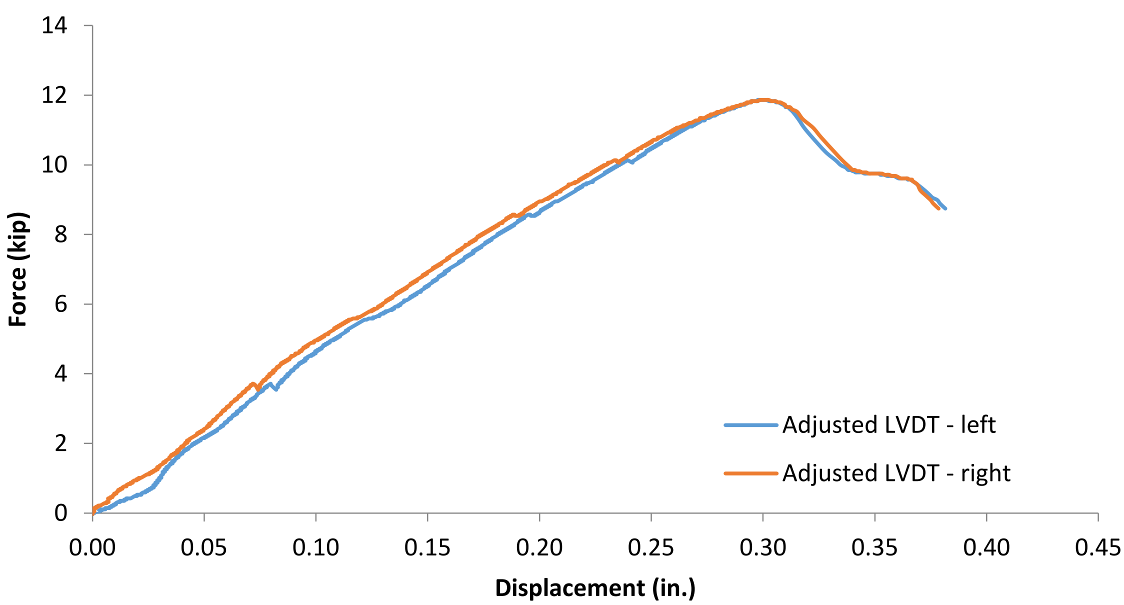


Figure J.2 Force vs. Displacement – LB-2

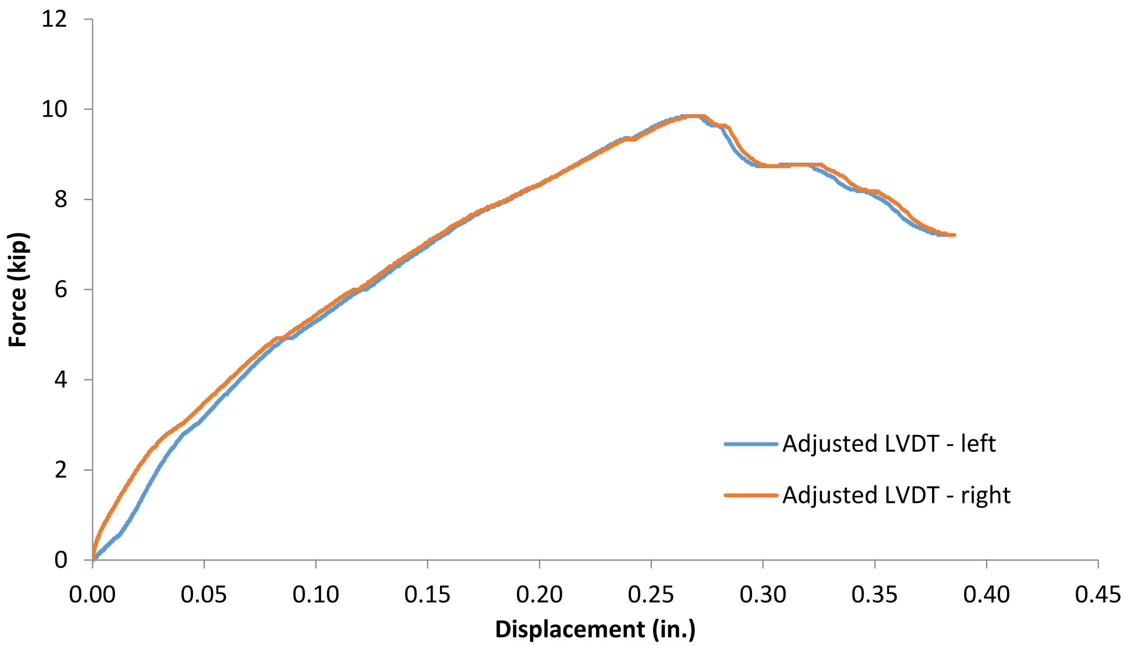


Figure J.3 Force vs. Displacement – LB-3

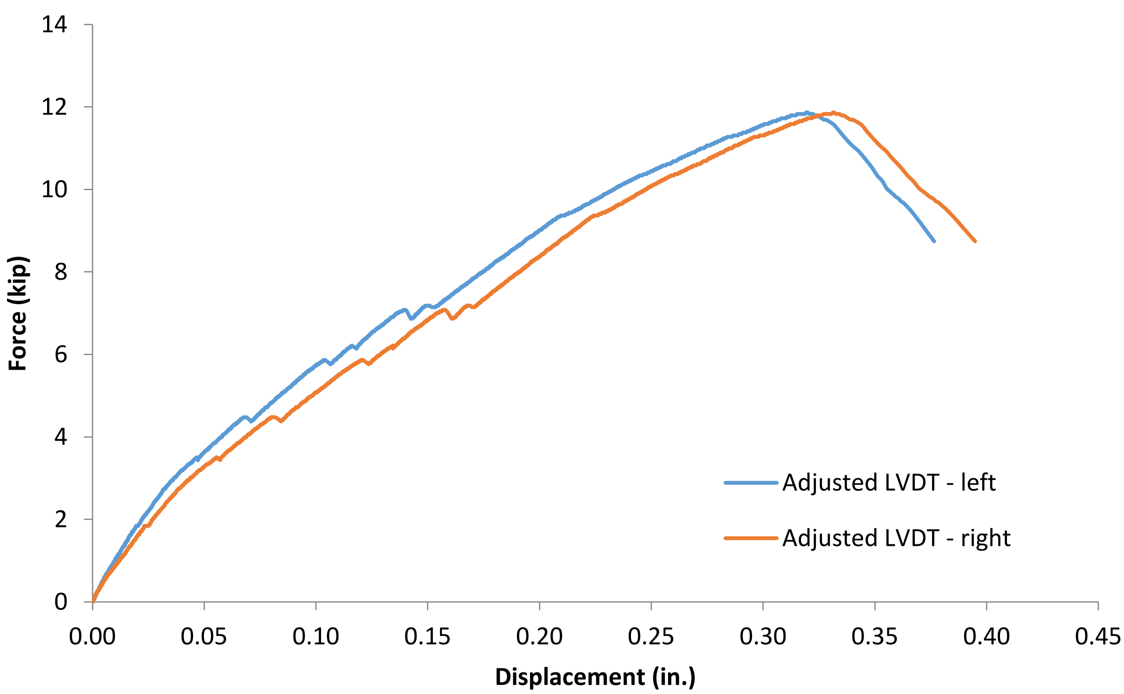


Figure J.4 Force vs. Displacement – LB-4

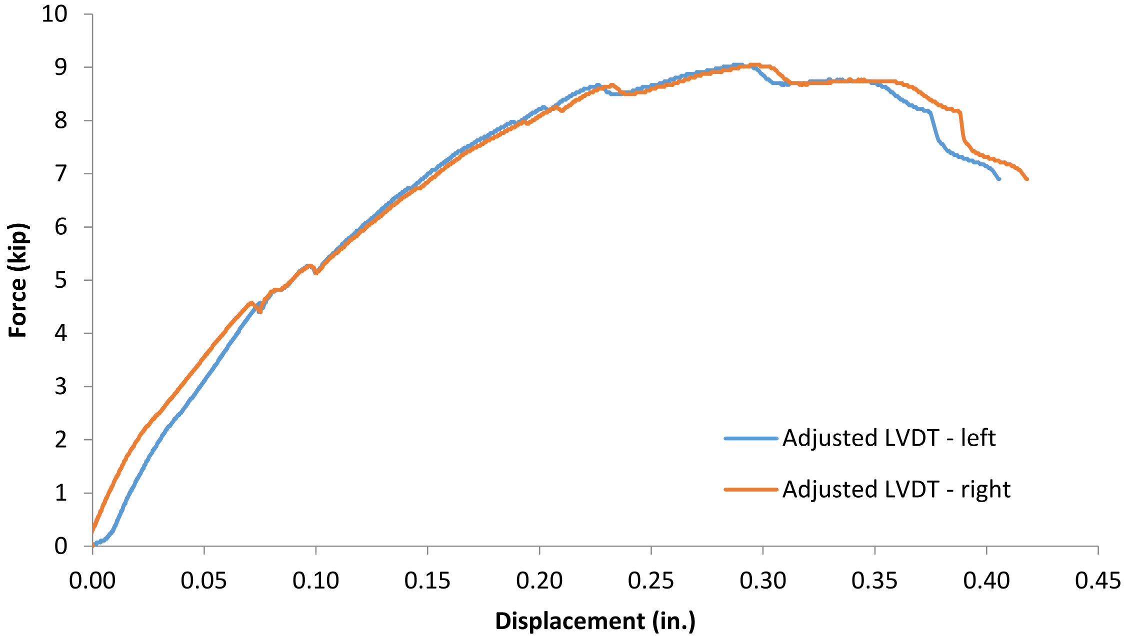


Figure J.5 Force vs. Displacement – LB-5

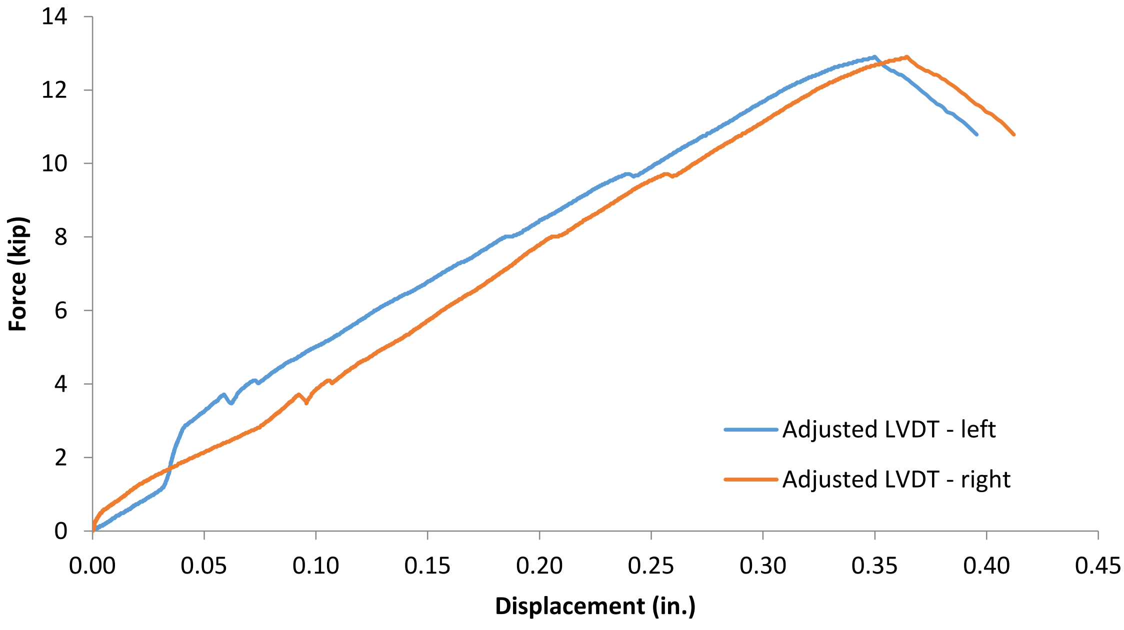


Figure J.6 Force vs. Displacement – LB-6

Moment vs. Rebar Stress

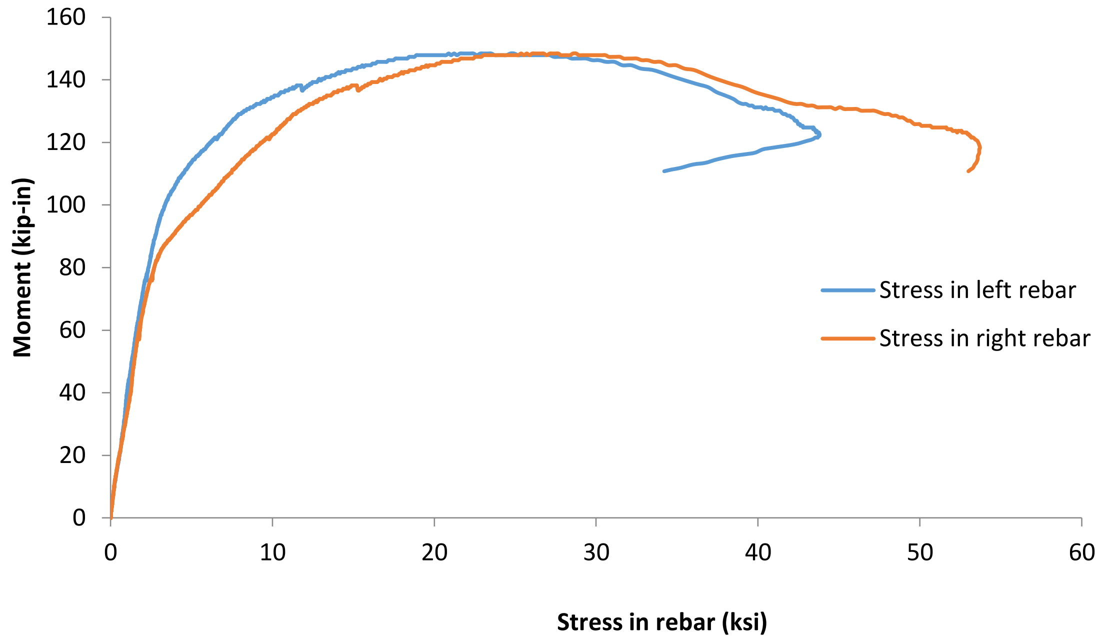


Figure J.7 Moment vs. Rebar Stress – LB-1

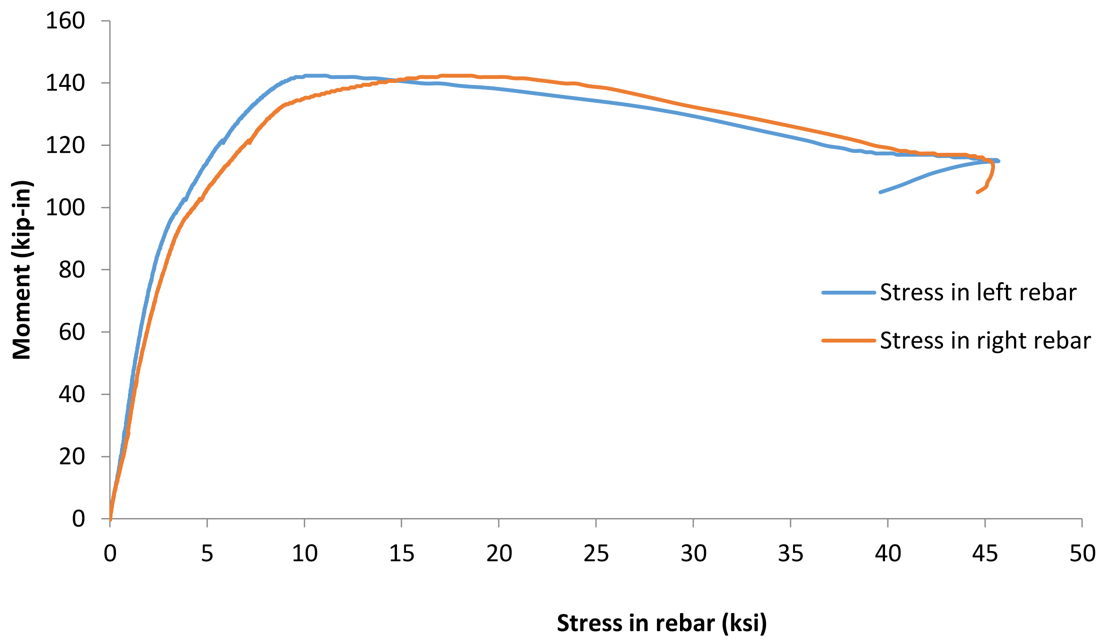


Figure J.8 Moment vs. Rebar Stress – LB-2

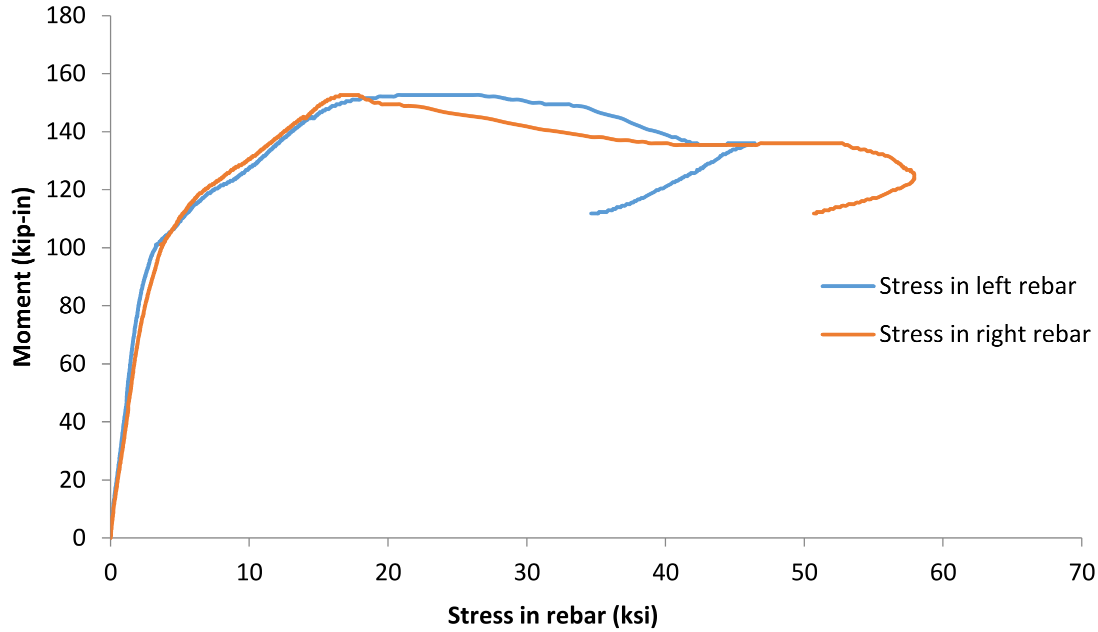


Figure J.9 Moment vs. Rebar Stress – LB-3

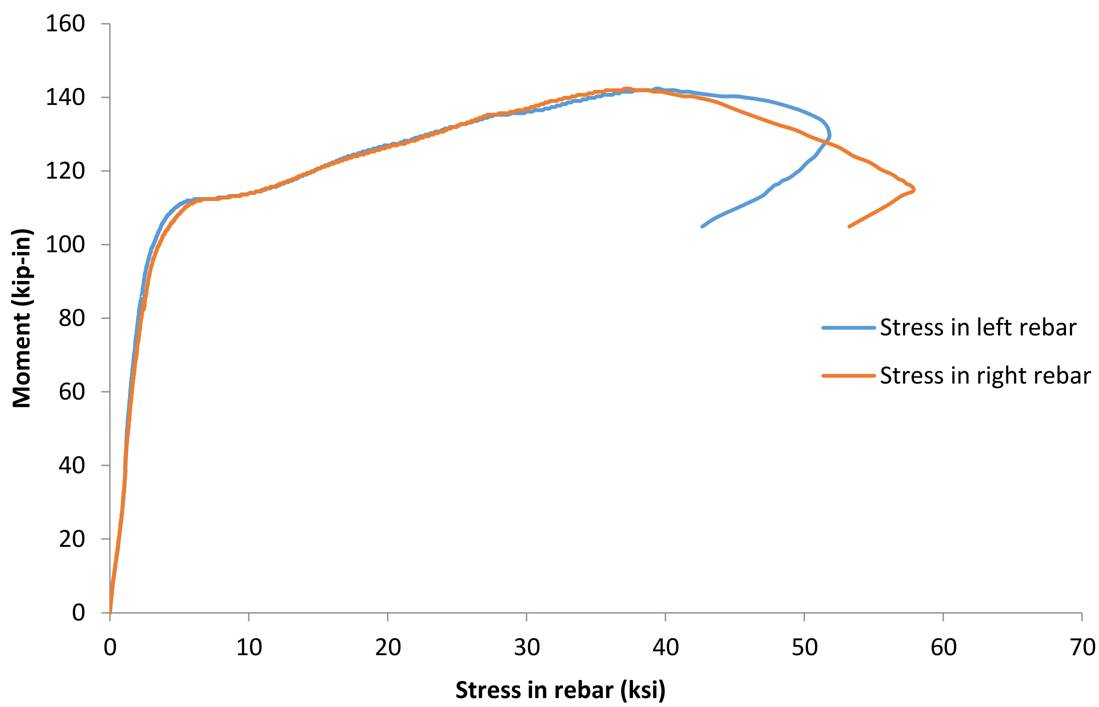


Figure J.10 Moment vs. Rebar Stress – LB-4

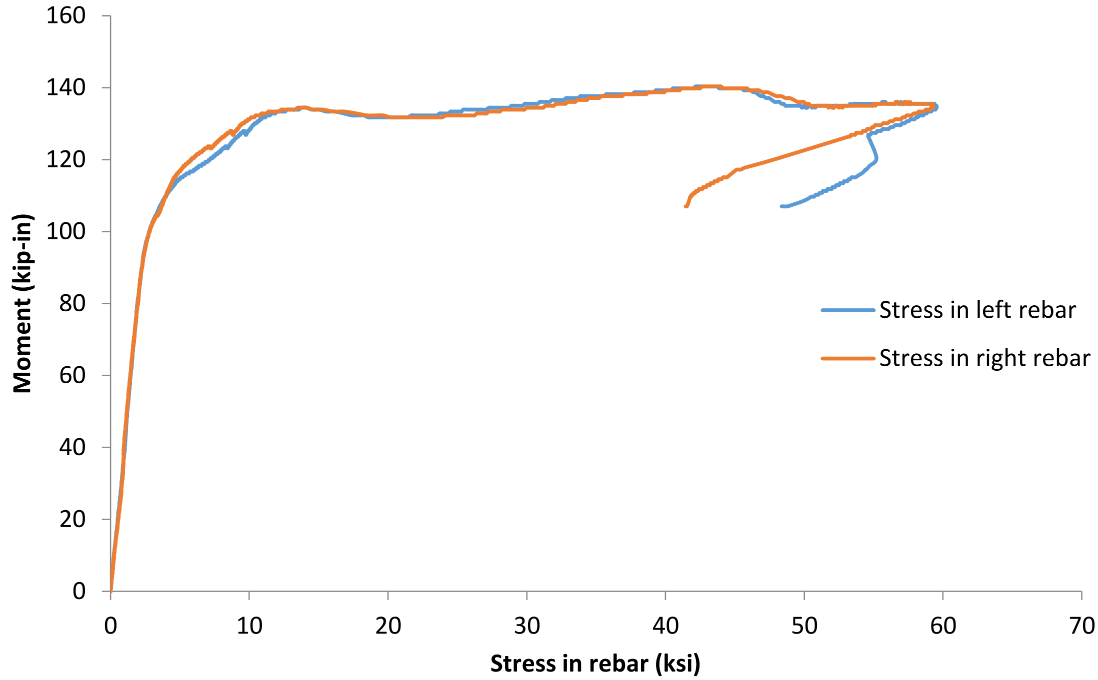


Figure J.11 Moment vs. Rebar Stress – LB-5

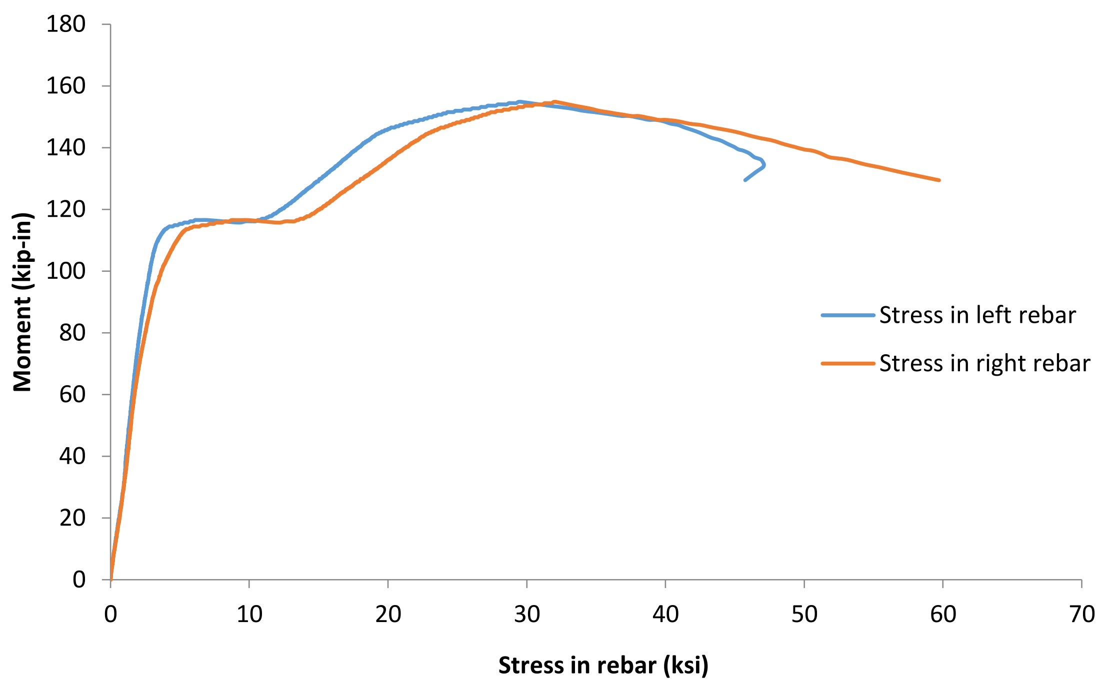


Figure J.12 Moment vs. Rebar Stress – LB-6

Moment vs. Concrete Strain

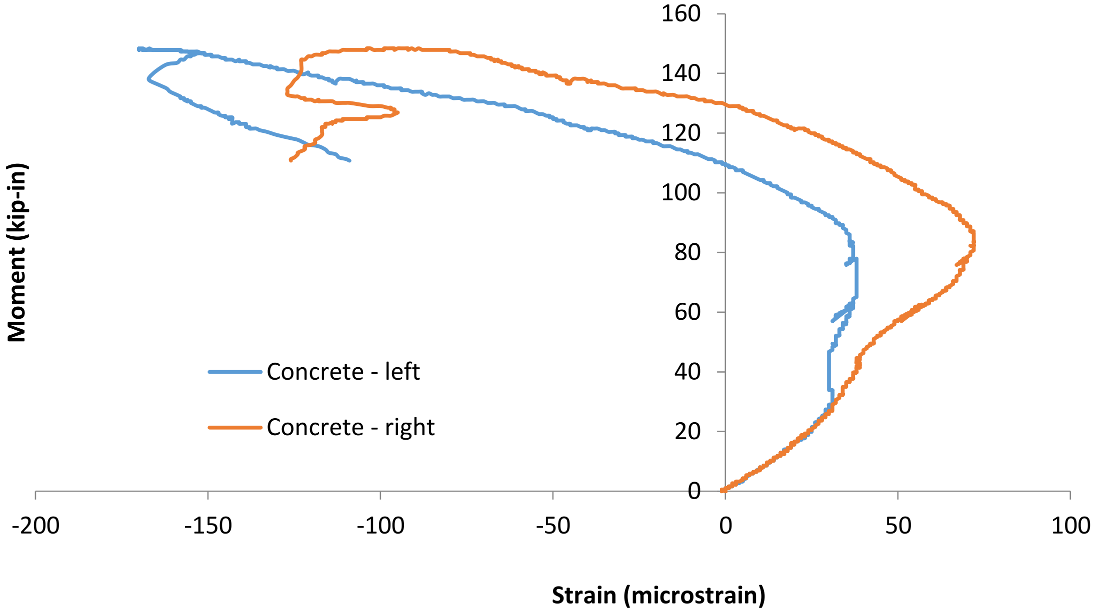


Figure J.13 Moment vs. Concrete Strain – LB-1

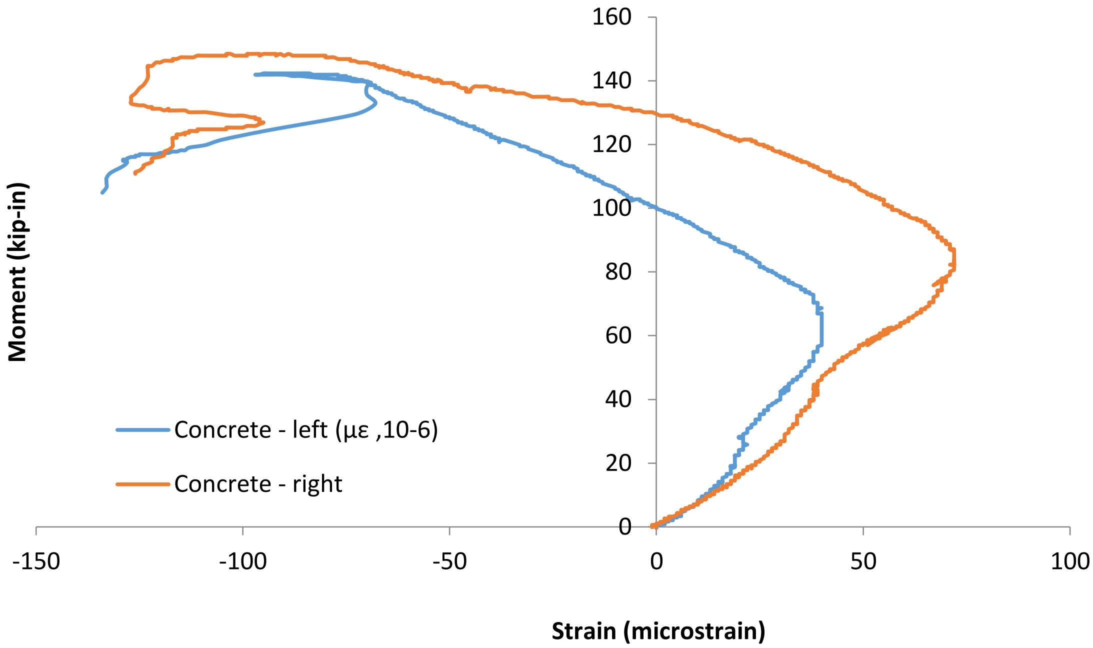


Figure J.14 Moment vs. Concrete Strain – LB-2

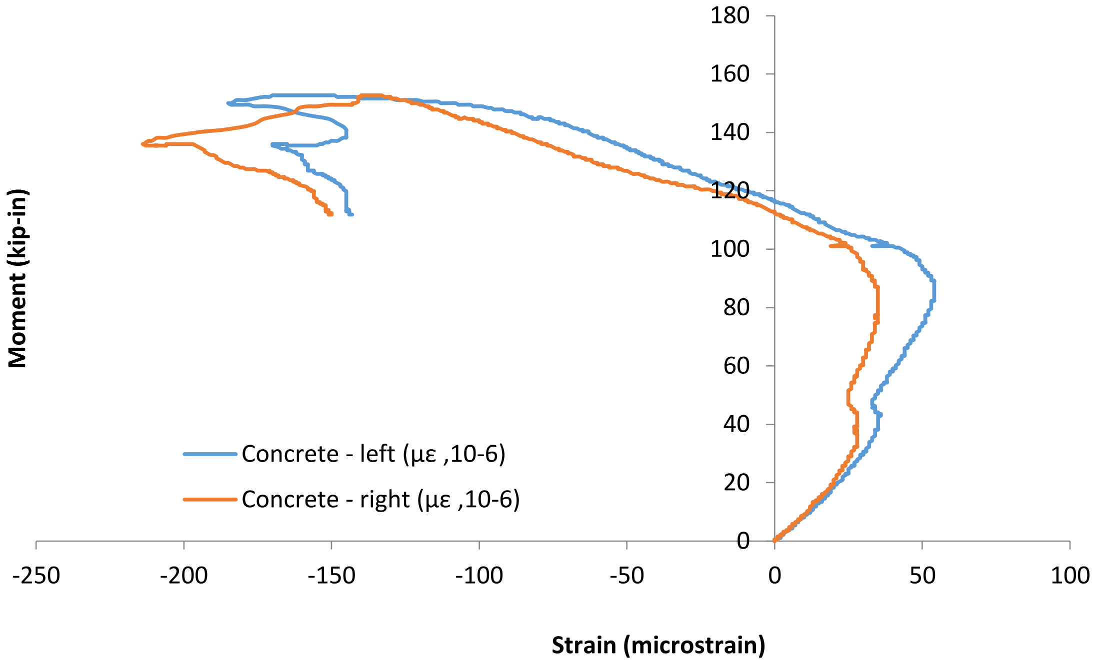


Figure J.15 Moment vs. Concrete Strain – LB-3

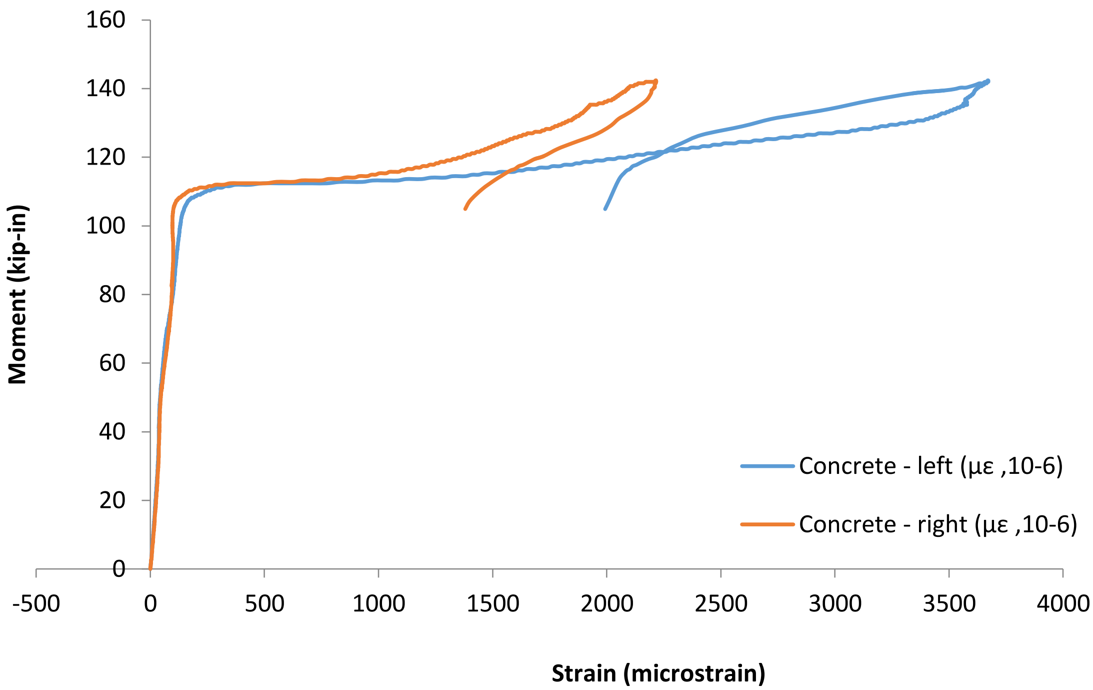


Figure J.16 Moment vs. Concrete Strain – LB-4

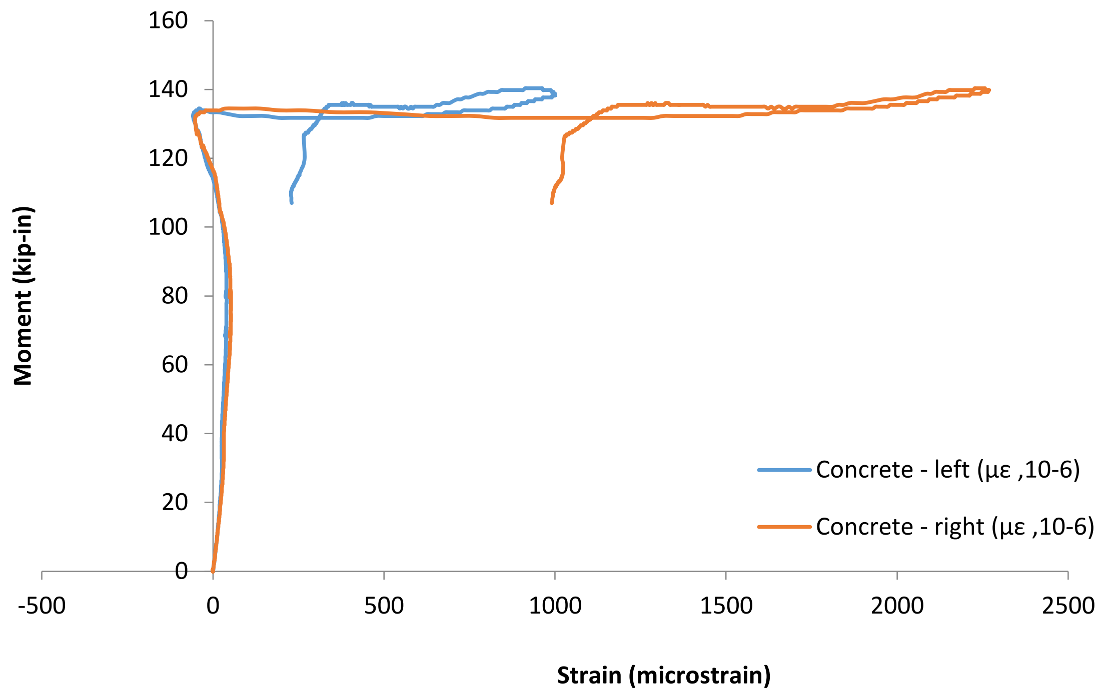


Figure J.17 Moment vs. Concrete Strain – LB-5

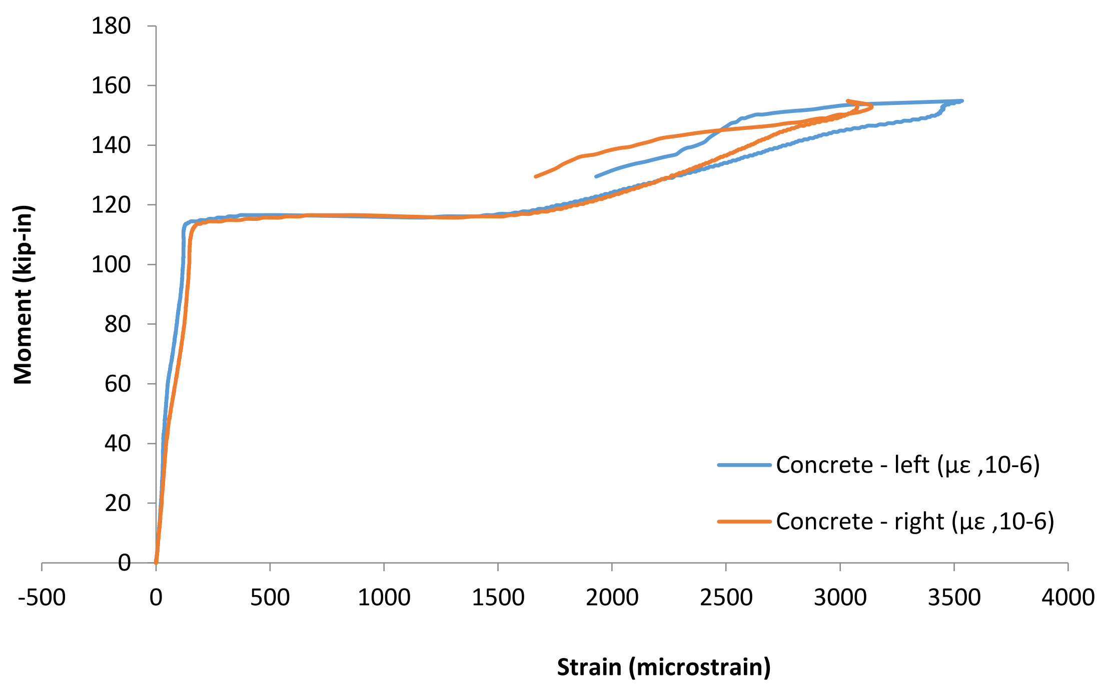


Figure J.18 Moment vs. Concrete Strain – LB-6

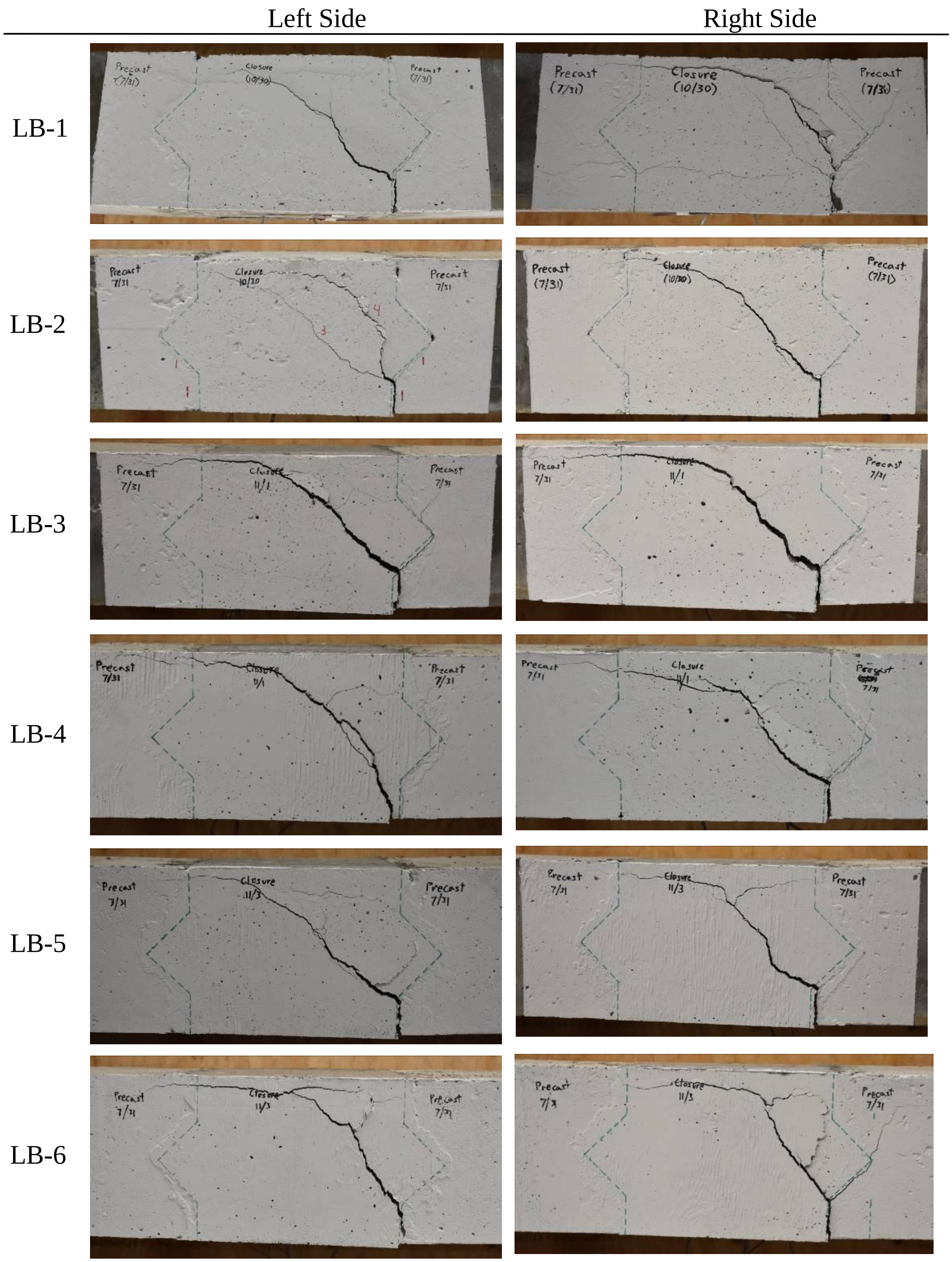


Figure J.19 Beam Specimen Cracks

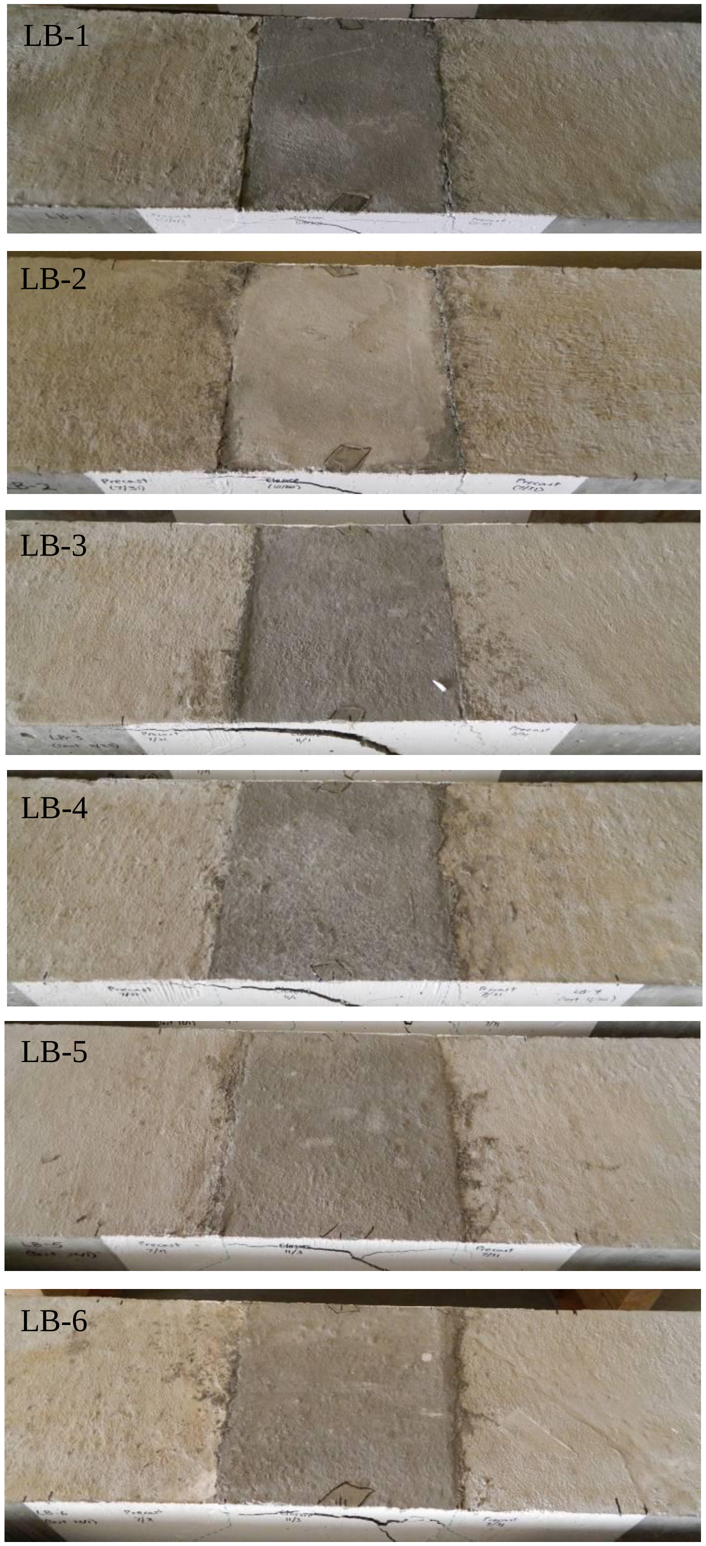
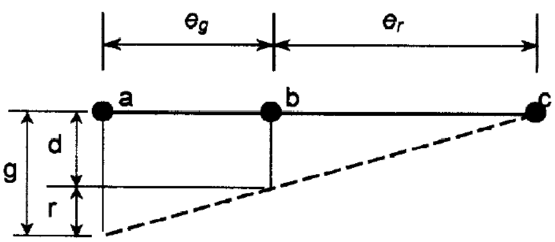


Figure J.20 Beam Specimen Top Surface

Appendix K   Modulus of Elasticity and Poisson's Ratio Procedure

Procedure:

1. Prior to testing for Modulus of Elasticity and Poisson's Ratio determine the compressive strength of a companion specimen to determine the ultimate load.
2. Measure two diameters perpendicular from each at the center of the specimen.
3. Measure compressometer/extensometer
   a. Measure distance from dial gage to pivot rod
   b. Measure distance from contact screw to pivot rod



d = displacement due to specimen deformation
r = displacement due to rotation of the yoke about the pivot rod
a = location of gage
b = support point of the rotating yoke
c = location of pivot rod
g = gage reading

g = gauge length (in)

e_r = perpendicular distance from the pivot rod to the vertical plane passing through the two support points of the rotating yoke (in)

e_g = perpendicular distance from the gauge to the vertical plane passing through the two support points of the rotating yoke (in)

e_h = perpendicular distance from the hinge to the vertical plane passing through of the middle yoke (in)

e'_g = perpendicular distance from the gauge to the vertical plane passing through the support points of the middle yoke (in)

4. Check that the compressometer/extensometer is straight and that all yolks align. If not, loosen the necessary bracing screws to adjust yolks.
5. Check pivot rod for proper placement between bottom and top yolks.
6. Tighten bracing screws (longitudinal and transverse).
7. Unscrew the seven contact screws (2 on the upper ring, 3 on the middle ring, and 3 on the lower ring) until the points are flush with the inside surface of the rings.
8. Place spacers under the lower ring to provide the correct height for compressometer/extensometer to align the center ring with the center of the cylinder.
9. Center the cylinder in the compressometer.
10. Hand-tighten the bottom anchor screws to contact the cylinder. Anchor screws should be tightened sufficiently to prevent the sample from moving within the compressometer.

    Note*: Avoid placing anchor screws into voids in the sample. If voids are visible rotate the sample so that no anchor screws will set into a void.*
11. Tighten the top anchor screws to contact the cylinder.
12. Screw middle anchor screws until contact is made with the cylinder. Do not tighten screw into concrete cylinder.
13. Place test specimen and compressometer into testing machine by carrying the assembly by the specimen and not the compressometer to avoid slipping of the anchor screws.
14. Unscrew all bracing screws on top and bottom rings and remove brace.
15. Unscrew one transverse bracing screw from the center ring.
16. Check anchor screws on top and bottom rings to ensure they are secured.
17. Squeeze transverse ring together to ensure adequate anchorage in the specimen.

18. Screw the middle anchor screws simultaneously until at least one turn on the dial gauge has been achieved for adequate anchorage in the specimen. Make sure transverse dial gauge reacts to this tightening, if not, repeat from previous step.
19. Ensure specimen is aligned in the center of the loading platen and load to 40% of ultimate load at a rate of 440 +/- 35 lbs/sec to seat the anchor screws into the specimen. Ensure dial gauges are moving during this loading process.
20. After unloading, squeeze middle yolk and turn middle anchor screws simultaneously until at least one full turn on the dial gauge has been achieved to ensure the anchor screws are in contact with specimen
21. First measurement loading: Load specimen up to 40% percent of ultimate load.
    a. Record: applied load and transverse strain when the longitudinal strain is 50 millionths
    b. Record: longitudinal strain and transverse strain when the applied load is 40% of the ultimate load.
22. Repeat steps 20 and 21 for a subsequent loading.
23. Remove specimen and compressometer from testing machine by carrying the specimen and not the compressometer.
24. Screw in all bracing screws (longitudinal and transverse).
25. Unscrew all anchor screws until the points are flush with the inside surface of the rings.
26. Remove the specimen.
27. Deterine the compressive strength, following ASTM C39.

The following equations are used in the excel file,

$$d = \frac{ge_r}{(e_r+e_g)} \quad \text{(K.1)}$$

$$d_t = \frac{ge_h}{(e_h+e'_g)} \quad \text{(K.2)}$$

$$g_{50} = \frac{ge_r}{(e_r+e_g)} \quad \text{(K.3)}$$

Where

d = total deformation of the specimen throughout the effective gage length (in)

d_t = transverse deformation of the specimen diameter (in)

E = chord modulus of elasticity (psi)

g = gauge reading (in)

g_{50} = gauge reading corresponding to a longitudinal strain of 0.000050 (in)

e_r = perpendicular distance from the pivot rod to the vertical plane passing through the two support points of the rotating yoke (in)

e_g = perpendicular distance from the gauge to the vertical plane passing through the two support points of the rotating yoke (in)

e_h = perpendicular distance from the hinge to the vertical plane passing through of the middle yoke (in)

e'_g = perpendicular distance from the gauge to the vertical plane passing through the support points of the middle yoke (in)

$$\varepsilon = \frac{\Delta}{G} \quad \text{(K.4)}$$

$$E = \frac{(S_2-S_1)}{(\varepsilon_2-0.000050)} \quad \text{(K.5)}$$

$$\mu = \frac{(\varepsilon_{t2} - \varepsilon_{t1})}{(\varepsilon_2 - 0.000050)} \quad \text{(K.6)}$$

Where

Δ = change in length

G = gage length

E = chord modulus of elasticity (psi)

S_1 = stress corresponding to a longitudinal strain, ε_{l1}, of 50 millionths (psi)

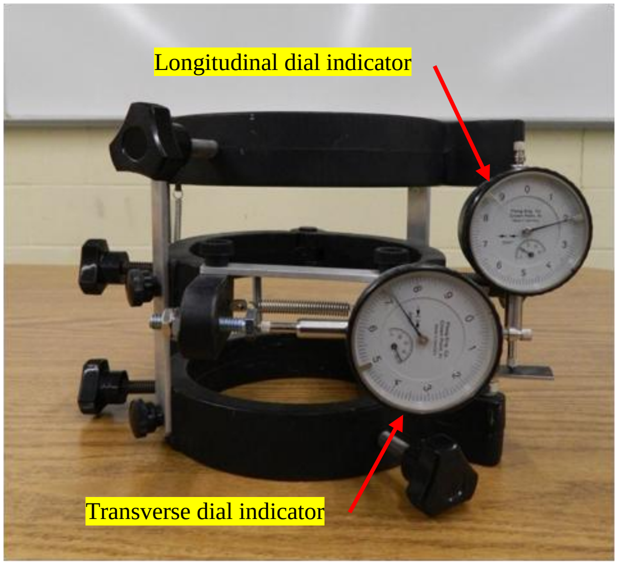
S_2 = stress corresponding to 40% of ultimate load (psi)

ε_{l2} = longitudinal strain produced by stress S_2

μ = Poisson's ratio

ε_{t1} = transverse strain at midheight of the sample produced by stress S_1

ε_{t2} = transverse strain at midheight of the sample produced by stress S_2



Appendix L Instrumentation

L.1 Strain gage specifications

Figure L.1 Quarter Inch Strain Gage Specifications

Figure L.2 Half Inch Strain Gage Specifications

Figure L.3 Two Inch Strain Gage Specifications

L.2 Rebar Measuring Procedure

The following section describes the procedure for measuring rebar diameters in preparation for strain gage installation. The process of installing strain gages on rebar changes the diameter and needs to be measured. Converting strain to force requires an accurate measurement of the cross-sectional area.

Gage installation requires smooth, flat, clean surfaces in order to achieve good adhesion. Reinforcing steel has ribs that need to be removed to provide a suitable surface for the strain gage. An example of an epoxy coated rebar before and after grinding is shown in Figure L.4.

Measurements are taken after the main surface preparation has been completed (this includes grinding off the ribs and the initial sanding/cleaning process) and before the gages are attached. Details about the installation process are included in the next section.

Figure L.4 Rebar Before (Bottom) and After (Top) Grinding

If measurements are taken after strain gages have been glued the thicknesses of additional materials, including masking tape, strain gage, and/or rebar epoxy, need to be taken into account. The thickness of adhesives is considered to be insignificant, trial measurements confirmed this, and won't be included. Thicknesses of the materials to consider are:

- Masking tape = 0.005 inch

- Strain gage = 0.003 inch
- Rebar epoxy = 0.007 inch

Using calipers, four diameter measurements are taken according to the diagram. Also note the number of instances a material was included for a measurement.

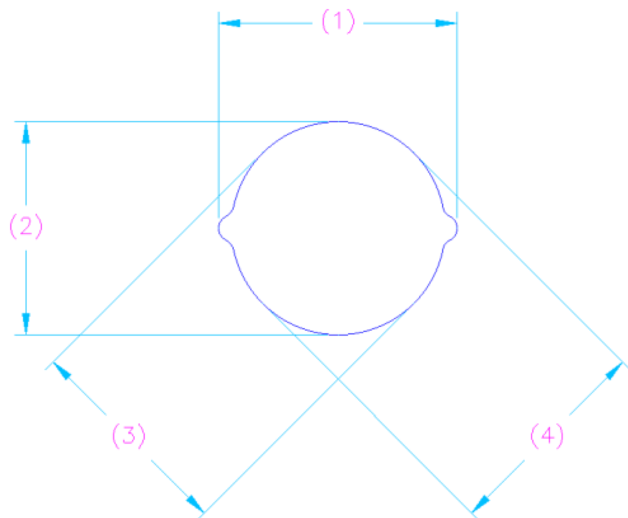


Figure L.5 Rebar Measurement Diagram

Measurements need to be adjusted to account for the additional thickness of materials, which is accomplished using the following equation:

$$Adjusted\ Dia. = Measured - 0.005 \times n_{tape} - 0.003 \times n_{S.G.} - 0.007 \times n_{epoxy}$$

Where,

n = number of occurrences for a measurement for masking tape, strain gage (S.G.), and rebar epoxy.

After averaging the adjusted diameters the cross-sectional area is calculated using the equation for a circle. The areas are then used to calculate stress or force in the rebar, refer to Chapter 4.5.1.

L.3 Rebar Measurements

Table L.1 Rebar Diameter Measurements – Headed Bar Pullout Specimens

Specimen	Measurement (in)	Tape	Strain gage	Rebar Epoxy	Adjusted Dia. (in)	Avg. Dia. (in)	Area (in^2)
HB-1	1) 0.682	1	0	2	0.663		
	2) 0.576	2	2	0	0.560		
	3) 0.563	2	0	0	0.553		
	4) 0.585	2	0	0	0.575	0.588	0.2713
HB-2	1) 0.681	0	0	2	0.667		
	2) 0.554	2	2	0	0.538		
	3) 0.579	0	0	0	0.579		
	4) 0.554	0	0	0	0.554	0.585	0.2683
HB-3	1) 0.694	0	0	1	0.687		
	2) 0.546	0	2	0	0.540		
	3) 0.561	0	0	0	0.561		
	4) 0.557	0	0	0	0.557	0.586	0.2699
HB-4	1) 0.682	0	0	2	0.668		
	2) 0.566	2	2	0	0.550		
	3) 0.581	0	0	0	0.581		
	4) 0.607	0	0	0	0.607	0.602	0.2842
HB-5	1) 0.672	0	0	2	0.658		
	2) 0.582	2	2	0	0.566		
	3) 0.586	2	0	0	0.576		
	4) 0.641	2	0	0	0.631	0.608	0.2901
HB-6	1) 0.692	0	0	2	0.678		
	2) 0.554	0	1	0	0.551		
	3) 0.583	0	0	0	0.583		
	4) 0.563	0	0	0	0.563	0.594	0.2769

Table L.2 Rebar Diameter Measurements – LB-1, LB-2, and LB-3

Specimen	Measurement (in.)		Tape	Strain gage	Rebar Epoxy	Adjusted Dia. (in.)	Avg. Dia. (in.)	Area (in.^{2})
LB-1A	1)	0.680	0	0	2	0.666		
	2)	0.558	2	2	0	0.542		
	3)	0.578	2	0	0	0.568		
	4)	0.580	2	0	0	0.570	0.587	0.2702
LB-1B	1)	0.666	0	0	2	0.652		
	2)	0.562	0	0	0	0.562		
	3)	0.561	0	0	0	0.561		
	4)	0.560	0	0	0	0.560	0.584	0.2676
LB-2A	1)	0.701	0	2	0	0.695		
	2)	0.553	1	1	0	0.545		
	3)	0.564	1	0	0	0.559		
	4)	0.579	1	0	0	0.574	0.593	0.2764
LB-2B	1)	0.693	0	0	2	0.679		
	2)	0.561	2	2	0	0.545		
	3)	0.573	2	0	0	0.563		
	4)	0.571	2	0	0	0.561	0.587	0.2706
LB-3A	1)	0.664	0	0	1	0.657		
	2)	0.567	2	2	0	0.551		
	3)	0.581	2	0	0	0.571		
	4)	0.590	2	0	0	0.580	0.590	0.2732
LB-3B	1)	0.675	0	0	2	0.661		
	2)	0.562	2	2	0	0.546		
	3)	0.571	2	0	0	0.561		
	4)	0.590	2	0	0	0.580	0.587	0.2706

A = rebar on left side, B = rebar on right side (refer to Figure 5.24 Top View of the Flexural Beam Test)

Table L.3 Rebar Diameter Measurements – LB-4, LB-5, and LB-6

Specimen	Measurement (in.)		Tape	Strain gage	Rebar Epoxy	Adjusted Dia. (in.)	Avg. Dia. (in.)	Area (in.^{2})
LB-4A	1)	0.661	0	0	1	0.654		
	2)	0.561	2	2	0	0.545		
	3)	0.570	2	0	0	0.560		
	4)	0.555	2	0	0	0.545	0.576	0.2606
LB-4B	1)	0.684	0	0	1	0.677		
	2)	0.563	2	2	0	0.547		
	3)	0.567	2	0	0	0.557		
	4)	0.573	2	0	0	0.563	0.586	0.2697
LB-5A	1)	0.700	0	0	2	0.686		
	2)	0.562	2	2	0	0.546		
	3)	0.574	2	0	0	0.564		
	4)	0.564	2	0	0	0.554	0.588	0.2711
LB-5B	1)	0.660	0	0	2	0.646		
	2)	0.558	2	2	0	0.542		
	3)	0.589	2	0	0	0.579		
	4)	0.579	2	0	0	0.569	0.584	0.2679
LB-6A	1)	0.692	0	0	1	0.685		
	2)	0.558	2	2	0	0.542		
	3)	0.572	2	0	0	0.562		
	4)	0.577	2	0	0	0.567	0.589	0.2725
LB-6B	1)	0.689	0	0	2	0.675		
	2)	0.549	2	2	0	0.533		
	3)	0.556	2	0	0	0.546		
	4)	0.563	2	0	0	0.553	0.577	0.2613

A = rebar on left side, B = rebar on right side (refer to Figure 5.24 Top View of the Flexural Beam Test)

L.4 Strain gage installation

Rebar strain gage installation

(Procedure uses #5 rebar and 0.25" strain gages.)

Using an angle grinder with a flap disc grind the epoxy and ribs on the top and bottom of the rebar where strain gages will be attached were grinded. Grind enough material to achieve a smooth surface. Measurements were taken to determine the new diameter as described in Section L.2.

Strain gages were installed following the "Gage Installation Procedure" as prescribed in Instruction Bulletin B-127-14 or B-137. Instruction Bulletin B-127-14 uses M-Bond 200 adhesive while B-137 used M-Bond AE-10 adhesive. Stain gages were installed so that the wires lead away from the head.

After applying the adhesive and placing strain gage onto surface take a length of installation tape and wrap the tape around the rebar to cover the gage. This will ensure that the entire gage is in contact with the rebar. Otherwise one side of the gage may not adhere properly.

Allow adequate time for the adhesion to cure then apply another strain gage on the opposite side. Align the strain gage to be opposite of the first. After both strain gages are attached check to make sure each gage is completely attached. Now, the lead wires will be attached.

Cut approximately the same length of wire for each strain gage. Separate and strip both ends of the wire. Take one end and twist the exposed wires of the black and white wires together. Tape around the strain gage with masking tape and leave the soldering terminals exposed as shown.

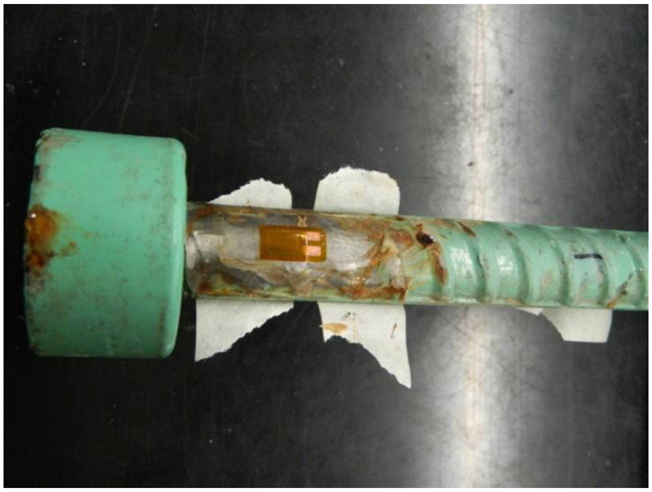


Figure L.6 SG on Rebar

Apply flux to the terminals and tin the tabs. Tin the end of the wires as well, the red and the twisted black and white wires.

Figure L.7 Masking Tape Wrapped Around SG

Tape the wire to the rebar. Trim excess wire. Tape the wire as shown in Figure 6 to help with soldering. Solder the wires to the gage.

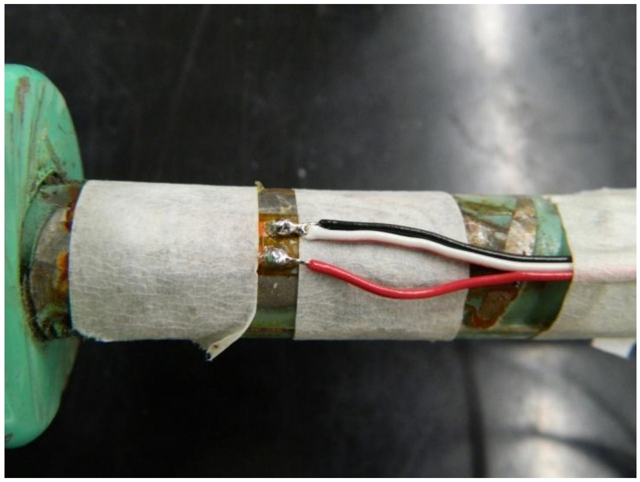


Figure L.8 Wires Soldered to SG

Use the multimeter to check the strain gage resistance. Check the red-black pair and the red-white pair. Remove the masking tape covering the strain gage then label the wires. Leave the tape that is under the wires to prevent the wires from contacting the rebar.

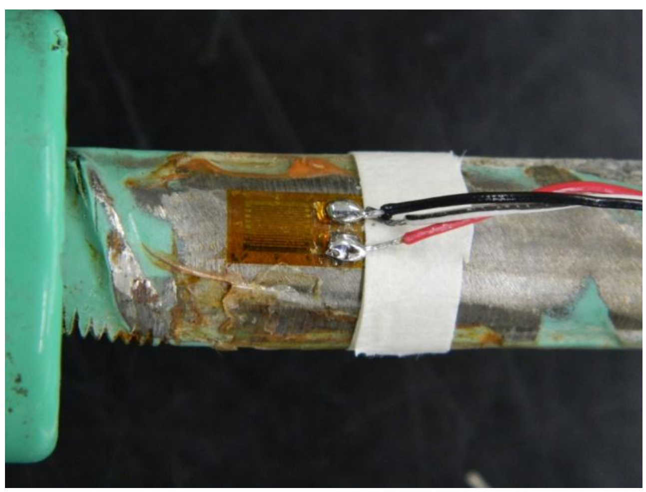


Figure L.9 Removed Masking Tape

After wires were attached M-Coat F protective coating was applied over the gages. The procedure is described next.

M-Coat F Procedure (*Instruction Bulletin B-134-5*)

a) Cut 2.5 inches of M-Coat FT Teflon tape. Wrap the tape around the rebar to completely cover the strain gage and solder connections. Electrical tape was also used instead of the Teflon tape. Wrap the electrical tape around the rebar several times to completely cover the strain gage and solder connections.

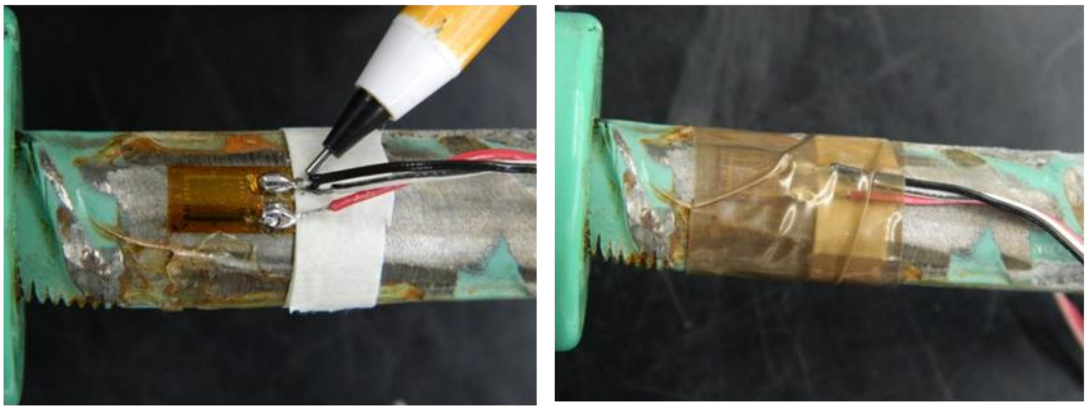


Figure L.10 Wrapping Teflon Tape

b) Cut a piece of M-Coat FB butyl rubber 1” x 2.75”

c) Remove the protective paper on one side of the butyl rubber and wrap it around the strain gage.

d) Remove the other protective paper then press and mold the rubber to the rebar.

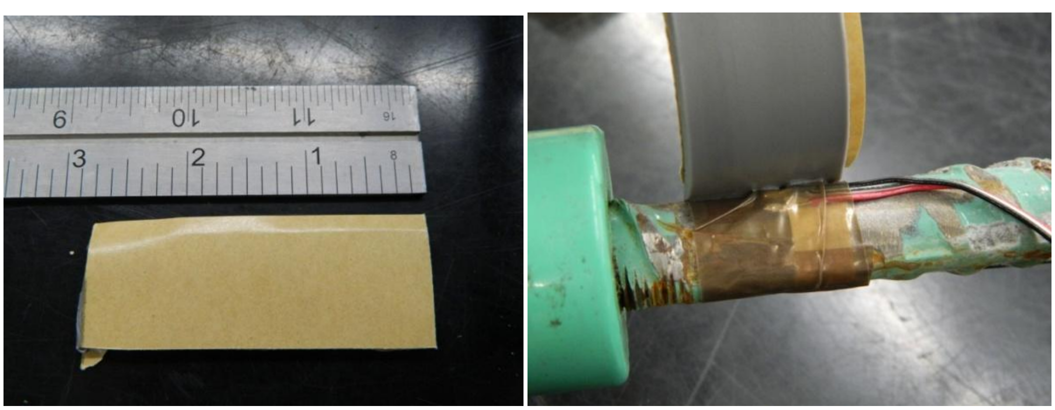


Figure L.11 Butyl Rubber

Figure L.12 Applying Rubber

e) Lift the lead wire then use a metal pick (dental probe) to form the rubber around the wires as shown.

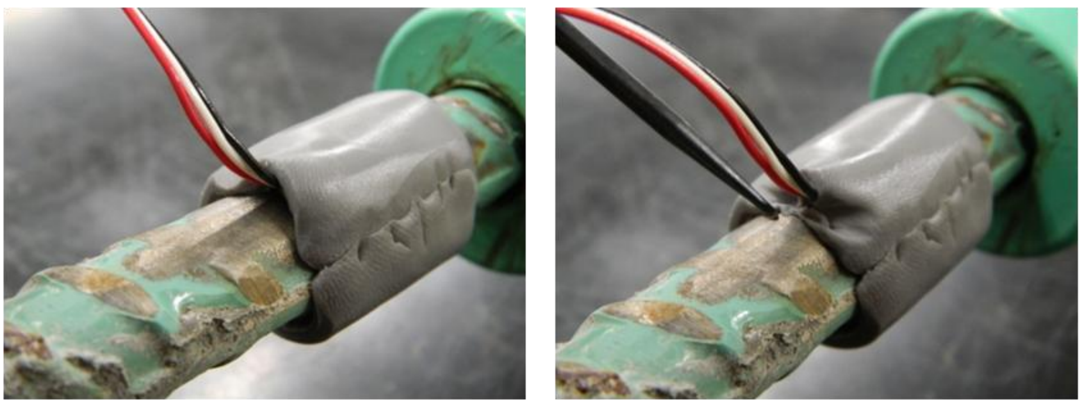


Figure L.13 Molding Rubber Around Wire

f) Press down all the edges of the butyl rubber against the rebar to form a seal.

g) Vinyl tubing was used to protect the wires from damage while casting the concrete.

h) Cut a 6 inch length of 0.25 inch diameter vinyl tube. From one end slice the tube along the length about 0.75 inches.

i) Feed the lead wires through the vinyl tube with the spliced end closer to the rebar.

j) Wrap the wires to meet at the bottom of the rebar (the wires will be extending through the bottom of the beam) as shown below.

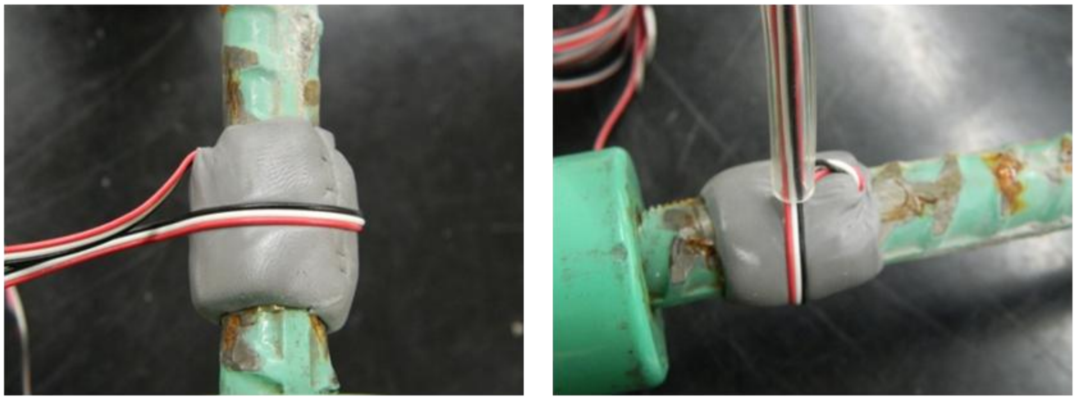


Figure L.14 Wrapping Wires Around Rubber

k) Slide the vinyl tube down and press into the butyl rubber.

Figure L.15 Pressing Vinyl Tube into Rubber

l) Cut a 1.25” x 3.25” piece of aluminum tape.

m) Cut a slit halfway through the tape as shown below.

n) Place the aluminum tape around the patch and press down the edges. If there are areas not covered cut a strip of aluminum tape and cover it.

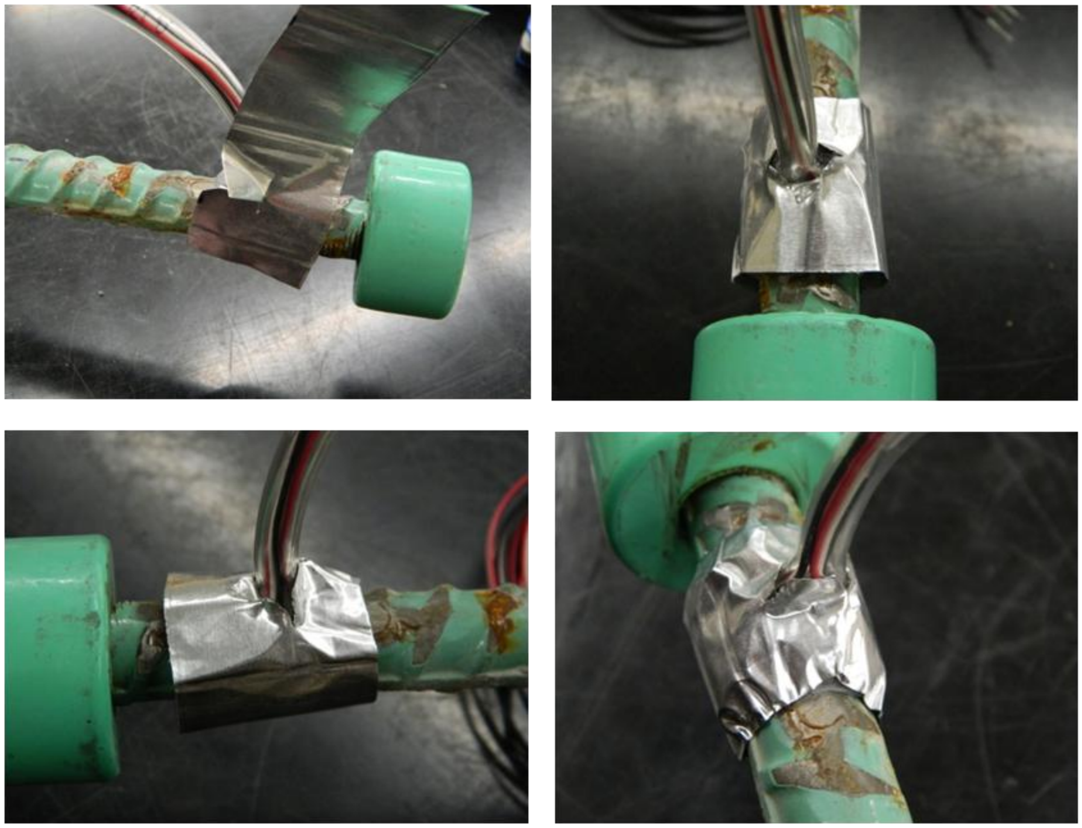


Figure L.16 Applying Aluminum Tape

L.5 DAQ Wiring Board

Sensors were wired according to the Idaho State University lab manual for the StrainSmart DAQ. The wiring boards for HB and LB tests were set up with the following channel assignments.

Table L.4 HB Pullout Test Channel Assignments

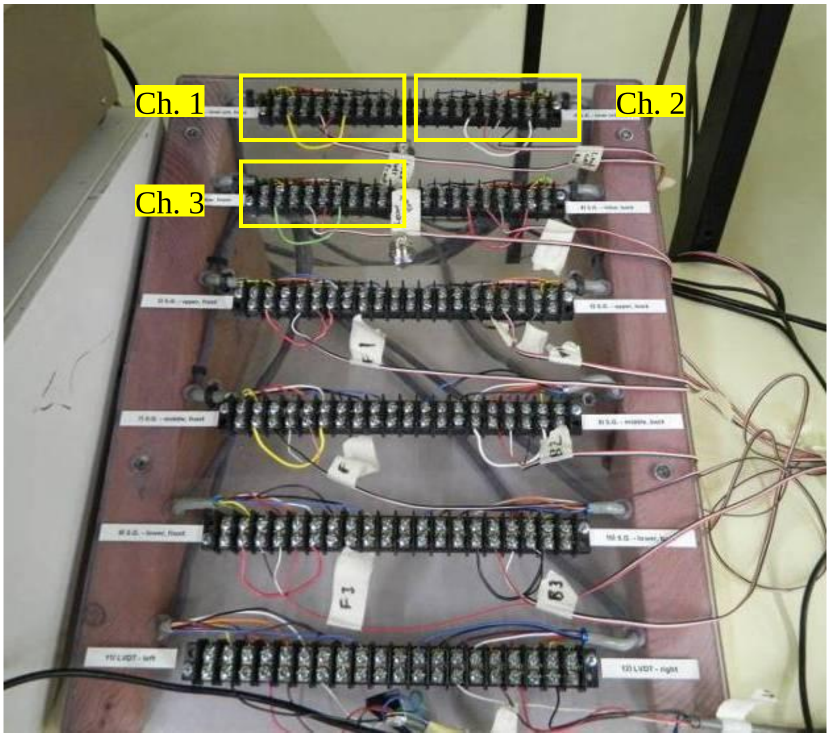
Ch. 1)	SG - load cell, front	**Ch. 2)**	SG - load cell, back
Ch. 3)	SG - rebar, front	**Ch. 4)**	SG - rebar, back
Ch. 5)	SG - concrete, upper, front	**Ch. 6)**	SG - concrete, upper, back
Ch. 7)	SG - concrete, middle, front	**Ch. 8)**	SG - concrete, middle, back
Ch. 9)	SG - concrete, lower, front	**Ch. 10)**	SG - concrete, lower, back
Ch. 11)	LVDT - left	**Ch. 12)**	LVDT - right

Note: SG = strain gage, LVDT = linear variable differential transducer

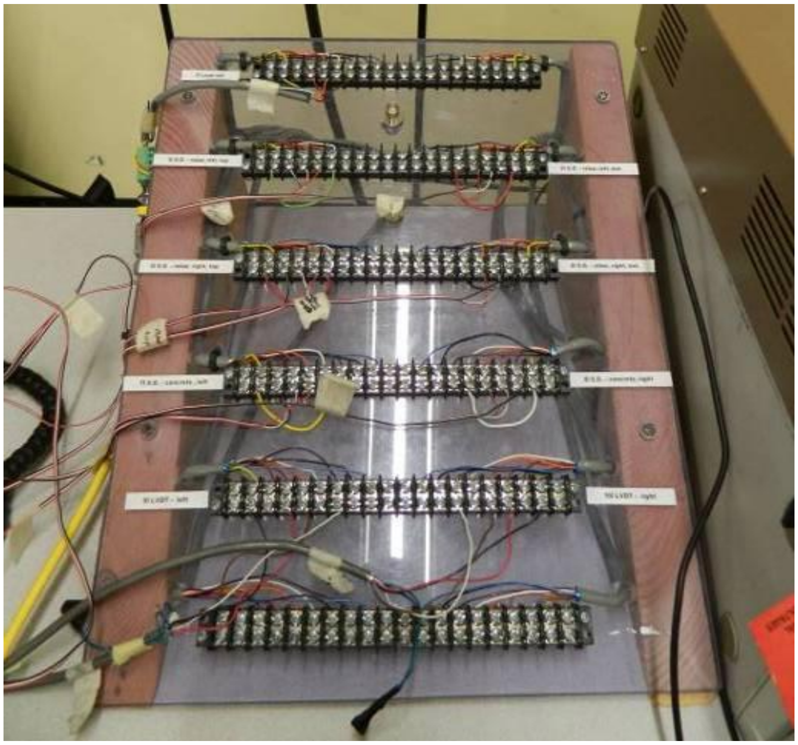
Table L.5 LB Test Channel Assignments

Ch. 1)	SGT - CLC-300K load cell	**Ch. 2)**	
Ch. 3)	SG - rebar, left, top	**Ch. 4)**	SG - rebar, left, bottom
Ch. 5)	SG - rebar, right, top	**Ch. 6)**	SG - rebar, right, bottom
Ch. 7)	SG - concrete, left	**Ch. 8)**	SG - concrete, right
Ch. 9)	LVDT - left	**Ch. 10)**	LVDT - right

Note: SG = strain gage; SGT = strain gage based transducer; LVDT = linear variable differential transducer



Wiring board setup for HB tests.



Wiring board setup for LB tests.

Figure L.17 Wiring Board Set-Up

L.5.1 Strain Gage

For the strain gage setup define a **Uniaxial Strain Gage Sensor** for the three strain gage types. (be sure to assign a descriptor when using more than one type of strain gage). The following was used,

- ¼ inch strain gage
    - **Descriptor** – 0.25 inch
    - **Gage Factor** – 2.120
    - **Resistance** – 120 Ohms
- ½ inch strain gage
    - **Descriptor** – 0.5 inch
    - **Gage Factor** – 2.095
    - **Resistance** – 120 Ohms
- 2 inch strain gage
    - **Descriptor** – 0.25 inch
    - **Gage Factor** – 2.120
    - **Resistance** – 120 Ohms

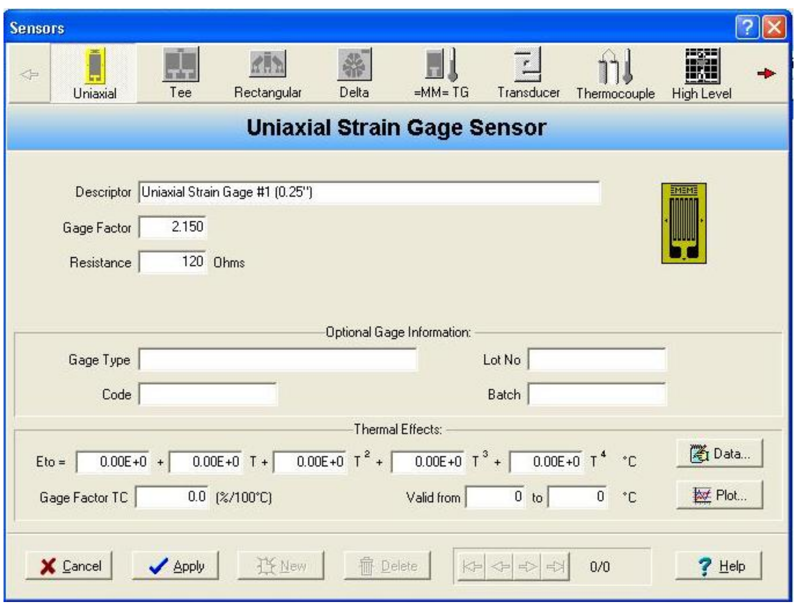


Figure L.18 Uniaxial Strain Gage Sensor Window

L.5.2 CLC-300K

Transducer Techniques®
Order Online !

Tech Support
(800) 344-3965
(951) 719-3965
Fax (951) 719-3900
email: tti@ttloadcells.com

CERTIFICATE OF CALIBRATION

SERIAL NUMBER:	304387	DATE OF CALIBRATION:	03/13/2014
SENSOR MODEL:	CLC-300K	DATE OF RECALIBRATION:	03/13/2015
JOB NUMBER:	99322		
TECHNICIAN:	LFJ		

COMPRESSION LOAD LBS.	MV/V INC	MV/V DEC
0	0.0000	0.0000
150000	1.0848	1.0852
300000	2.1624	

NON-LINEARITY	-0.17	PCT FS
NON-REPEATABILITY	-0.03	PCT FS
HYSTERESIS	0.02	PCT FS

SHUNT CALIBRATION

PCT LOAD	LOAD LBS.	SIGNAL MV/V	SHUNT K OHMS	SHUNT PINS
46.78	140330.70	1.0115	87.325	(-E,-S)
93.55	280661.40	2.0230	43.575	(-E,-S)

DPM-2 SCALE FACTOR 0.4162

CALIBRATION COMPUTED FROM THREE (3) RUNS INCREASING AND DECREASING
TRACEABLE TO NIST TEST # 58183
CALIBRATION PERFORMED AT 10 VDC
MAXIMUM BRIDGE EXCITATION 12 VDC

WIRING

PIN	COLOR	CODE
N/A	RED	+EXCITATION
N/A	RED/BLK	-EXCITATION
N/A	GRN	+SIGNAL
N/A	RED/WHT	-SIGNAL
N/A	RED/YEL	NOT USED
N/A	RED/BLU	NOT USED
N/A	SHD	GROUND

42480 Rio Nedo, Temecula, CA 92590 (951) 719-3965 Fax (951) 719-3900
URL: http://www.transducertechniques.com email: tti@ttloadcells.com

Figure L.19 Calibration Sheet for CLC-300K Load Cell

The CLC load cell has a load capacity of 300,000 lb in compression. The load cell contains a four active arm wheatstone bridge (full bridge) strain gage with a gage resistance of 350 Ohms. The CLC will be setup as a **Strain Gage Based Transducer**. Figure L.20 shows the wiring configuration for a full bridge sensor.

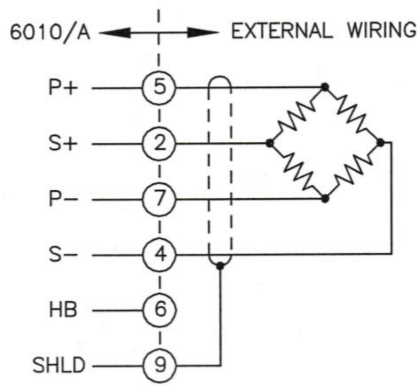


Figure L.20 Full Bridge Wiring Configuration

Remove the strain gage card that will be used to run the load cell from the back of the scanner. On strain gage card, set the bridge completion for 350 Ohms. This is done by changing the JPM1 pin from 120 Ohms to 350 Ohms.

Use the screw terminal adapter to wire the CLC to the scanner. Table L.6 shows the wiring for the CLC-300K to System 6000.

Table L.6 CLC Wiring Chart

Load Cell Pin from Screw Terminal Adapter	Scanner
1	2
6	4
2	5
7	7

Figure L.21 Screw Terminal Adapter

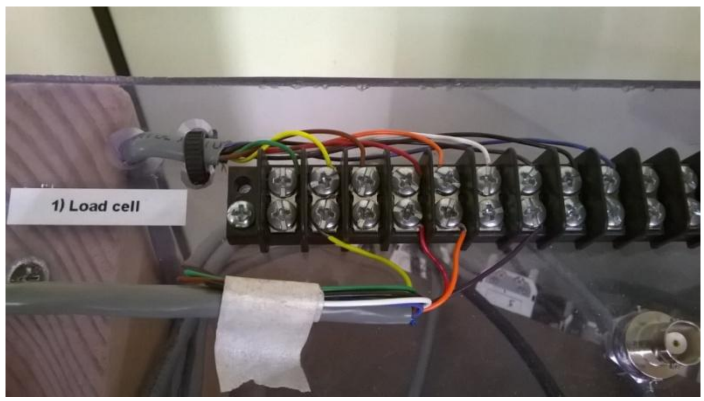


Figure L.22 CLC Wiring to Scanner

In the StrainSmart setup proceed to the “New Sensor” window of the program. Define a **Strain Gage Based Transducer** for the load cell. (assign a descriptor such as CLC-300K). Excitation is 10V, shown in the calibration sheet.

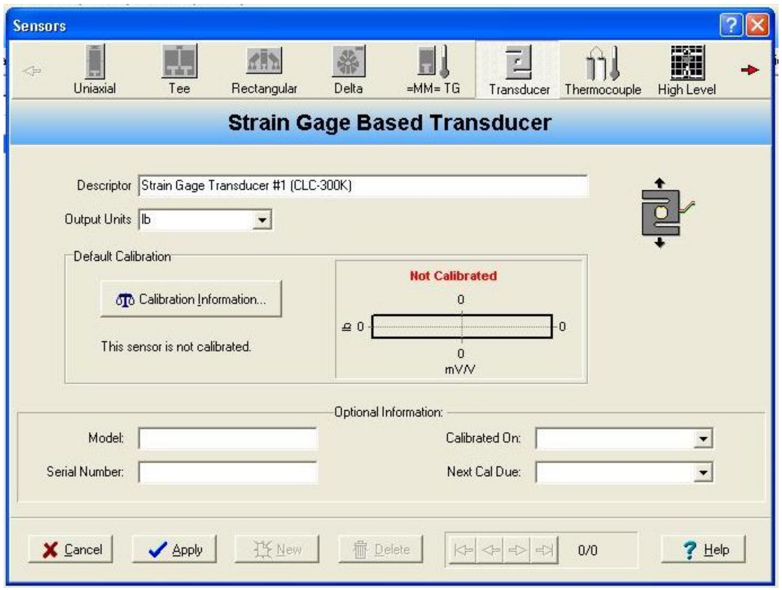


Figure L.23 Strain Gage Based Transducer Window

Calibration: For the zero/calibration step input the following two data sets,

- 0 mV/V – 0 lb
- 2.1624 mV/V – 300,000 lb

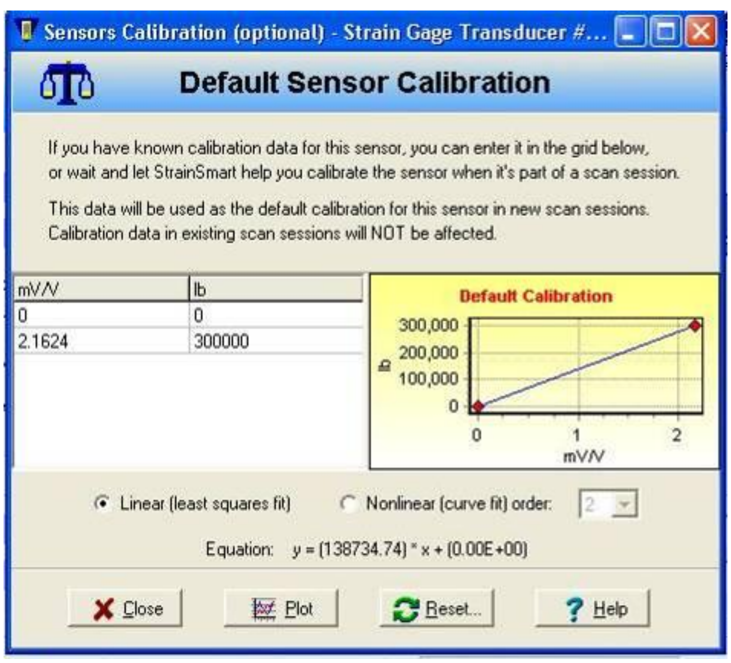
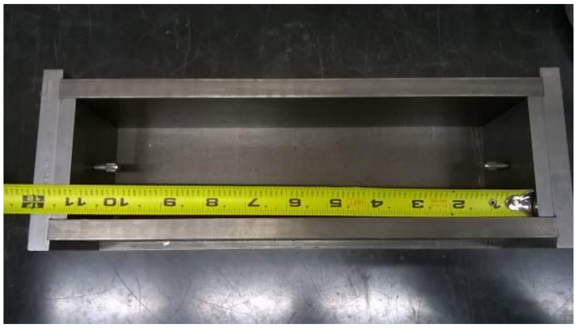


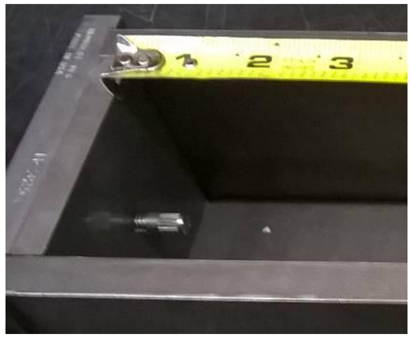
Figure L.24 CLC-300K Calibration Setup Screen

Appendix M Additional Photos

M.1 Shrinkage Molds



Metal mold.



Gage stud screwed into end of mold.

Wood mold. A wood sealer was applied to protect the mold over multiple uses.

Gage stud screwed into end of mold.

M.2 Rebar preparation

Rebars with Lenton Terminators (Headed rebars).

Headed rebar before (bottom) and after (top) grinding.

Grinding gage areas on rebar.

Cleaning rebar surfaces.

Preparing to attach strain gages.

After strain gages were applied they were set out to cure.

Silicon pads and a spring clamp applying pressure over gage area.

Strain gage installed.

Strain gages applied to all headed rebars.

M.3 Interface Beam Preparation

Interface beam mold.

Interface beam molds.

Concrete surface retarder (Formula F) being applied to inside of mold.

Concrete surface retarder applied to center divider.

Precast concrete section poured.

Concrete curing after pouring (allowed to cure for 24 hours).

After washing away unhardened concrete produced from concrete retarder.

Exposed aggregate surface finish on the precast concrete.

SSD substrate moisture condition applied to specimen without bonding agent.

Showing a width of 6 inches.

Applying bonding agent (Tammsweld) with brush.

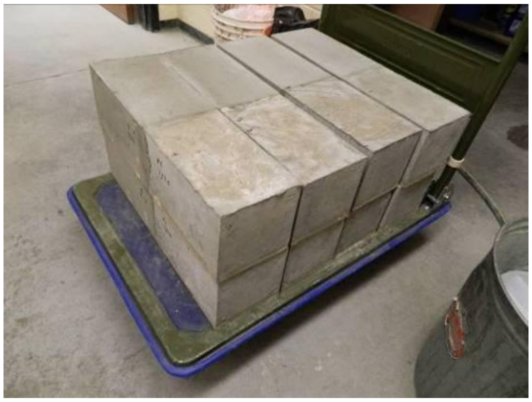
Sample applied with bonding agent (left) and without (right).

Precast portion set back in mold in preparation for closure concrete.

Closure concrete poured.

Closure concrete poured for all specimens

Wrapped in plastic for the first 24 hours.



Interface beam specimens after 28 days of curing.

Interface beam marked and ready to be testing.

M.4 Headed Bar Pullout Preparation

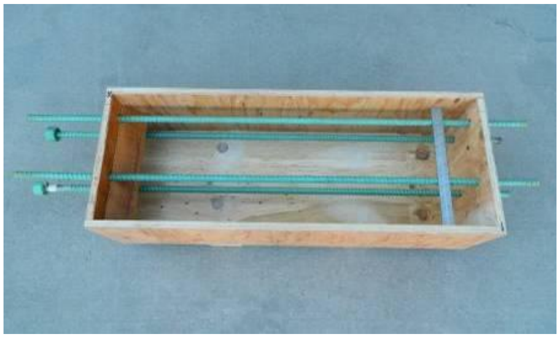
Headed bar pullout mold showing an overall length of 20 inches.

HB pullout mold showing width of 9 inches.

All six HB pullout molds.

Inside dividers painted with Formula F concrete surface retarder.

M.5 Flexural Beam Preparation



Large beam mold showing a width of 12 inches.

Shear key formwork coated with concrete surface retarder.

Showing rebars placed 2 in. and 6 in. from the bottom of the mold.

Showing rebar spaced 6 inches on center.

LB molds prepared for pouring.

LB molds prepared for pouring.

M.6 Pour Day (Precast Concrete for HB and LB)

Pouring LB precast portions.


Rodding the concrete.


Pouring second lift.


Rodding the concrete.

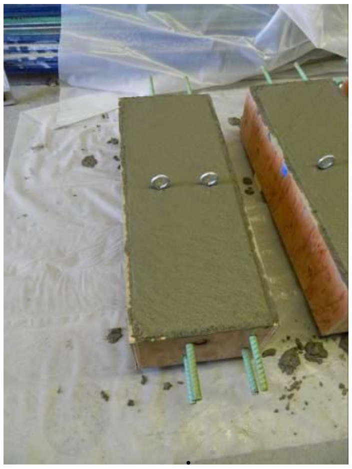

Finishing the surface with a trowel.

Pouring HB precast portions.

Moving finished beams onto plastic tarp.

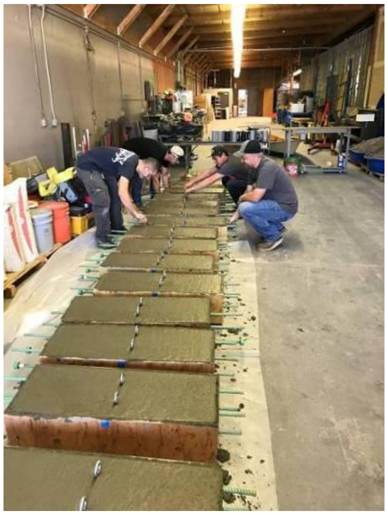
Inserting eye bolts in LB specimens.

Checking the position of the rebars with wood gauge.



After inserting eye bolts and checking rebars.

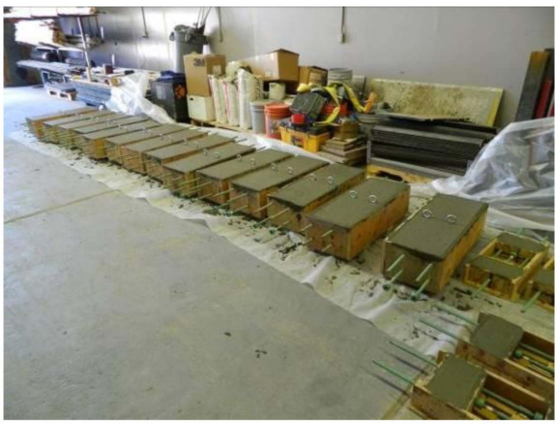
Cleaning rebars and inserting eye bolts.



Checking rebar positions and eye bolt placement.

Over 30 cylinders poured
(4 in. by 8 in.).

6 precast portions of HB
specimens poured.



14 precast portions of LB specimens
(7 sets of beams, 1 used for practice).

Covered with plastic for the first 24
hours.

M.7 HB Mold Removal

HB specimens after 24 hours.

Removing HB precast portions from molds.

After removal from form.

Preparing to spray with water.

Precast interface before washing.

Before and after washing precast interface.

M.8 LB Mold Removal

Removing LB precast beams from mold.

Removing shear key formwork.

Unwashed precast interface.

LB precast before and after washing.

LB precast before washing.

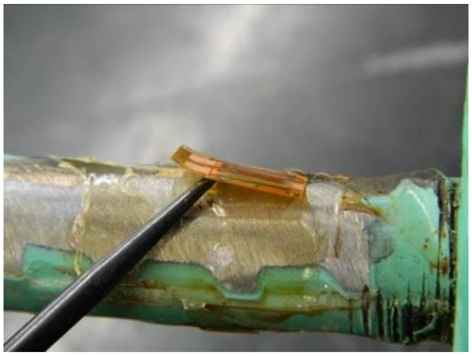
LB precast after washing, showing EA finish.

M.9 Rebar Strain Gages

Strain gages wrapped with tape for protection during curing.

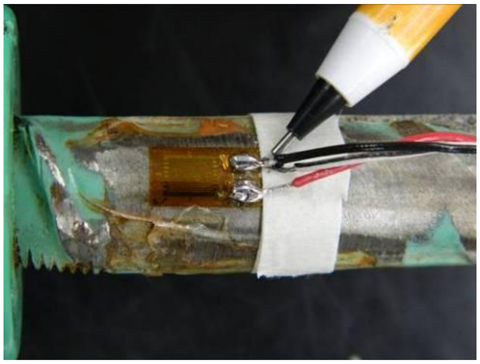
Inspecting strain gages.

Bonded strain gage.



Unbonded strain gage.

Strain gage before attaching lead wires.



Lead wires soldered to strain gage.

Strain gage with lead wires.

M-Coat F gage protection.

M.10 HB Specimen Instrumentation and Preparation

Preparing HB mold for pouring closure concrete.

HB specimen before pouring closure concrete.

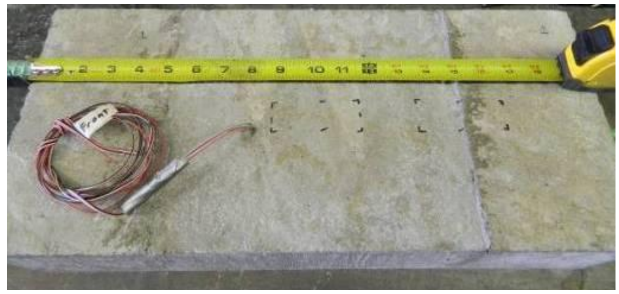
Fully cast HB specimen.

End fixtures made from 1 in. threaded rod and steel plate.

Threaded bar and steel plates welded to bottom rebars.

Threaded bar and steel plates welded to top rebar.

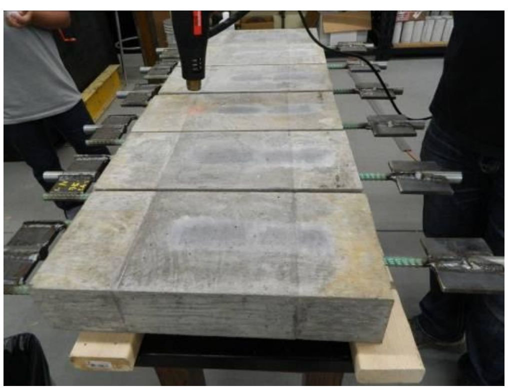
All HB specimens with fixtures.



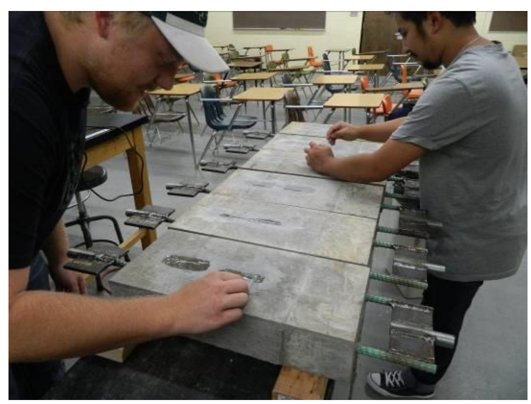
Preparing HB specimen for
concrete strain gage installation.

Gage area marked out for grinding.

Gage area after grinding.



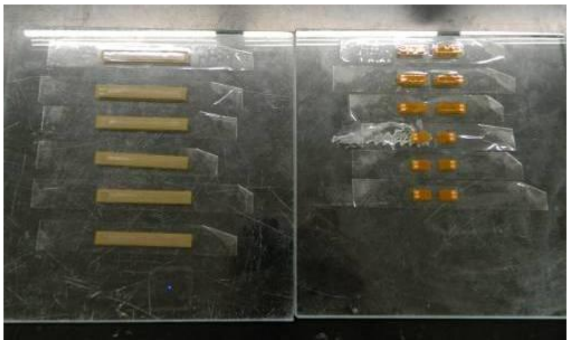
Gage areas cleaned then dried with a
heat gun.



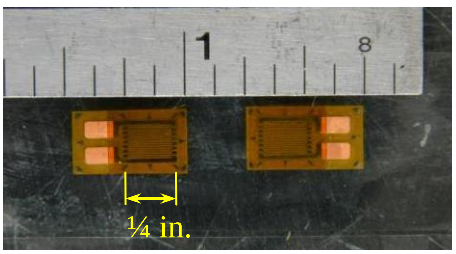
Applying base coat of M-Bond
AE-10 to gage areas.

Base coat after curing 24 hours.

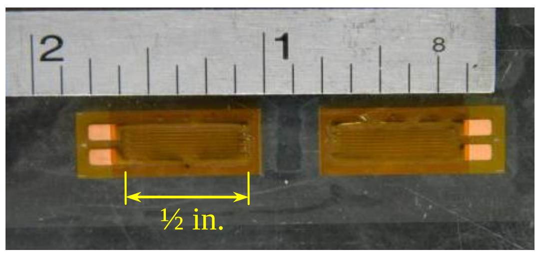
Grinding/sanding base coat down to concrete surface.



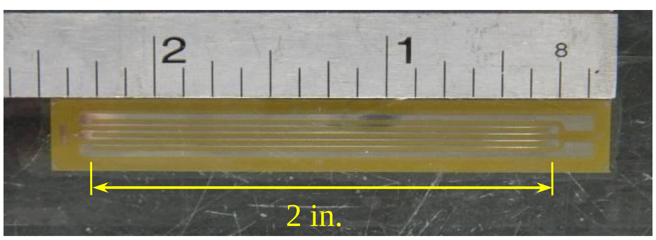
Strain gages ready for placement.



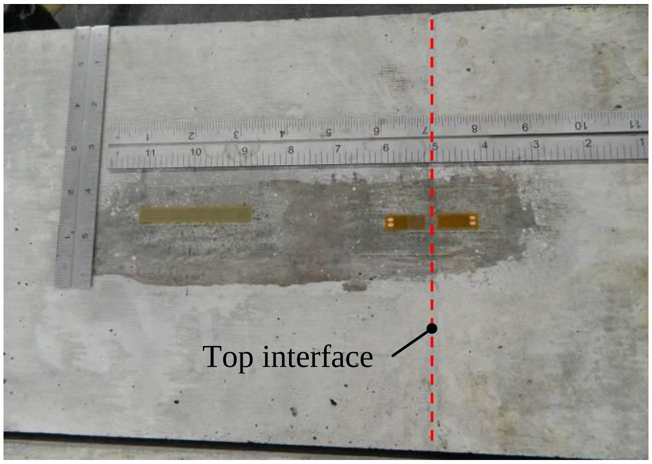
¼ in. strain gages shown spaced ¼ in. apart (placed at interface).



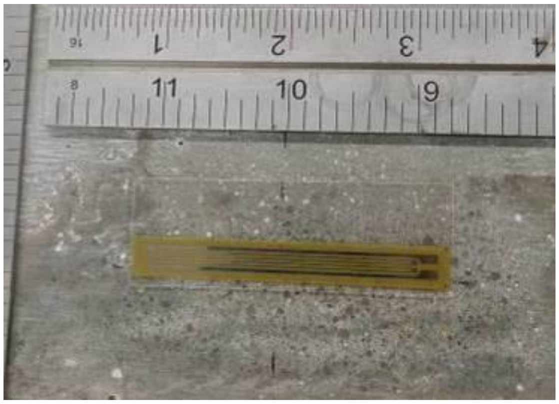
½ in. strain gages shown spaced ¼ in. apart (placed at interface).

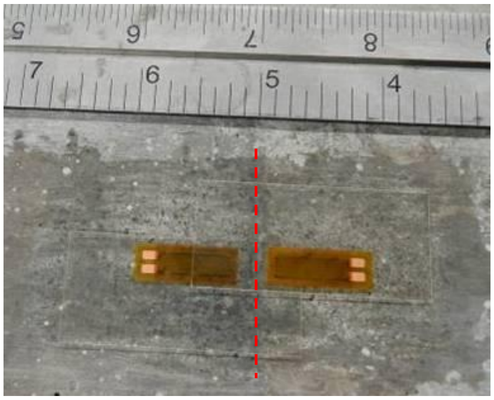


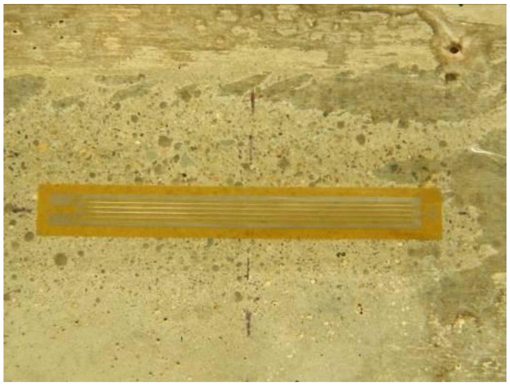
2 in. strain gage (placed in center of closure).

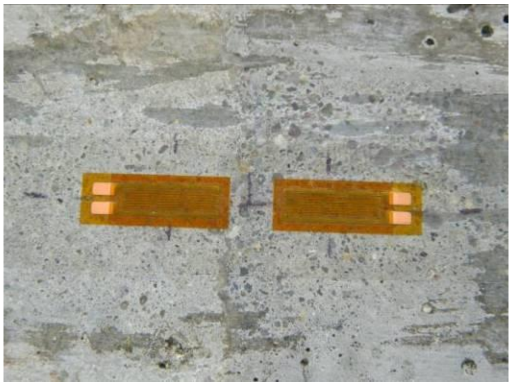


Placement of HB concrete strain gages.


2 in. strain gage placed at center of closure concrete. (Center of gage at 10 in. from top of sample)

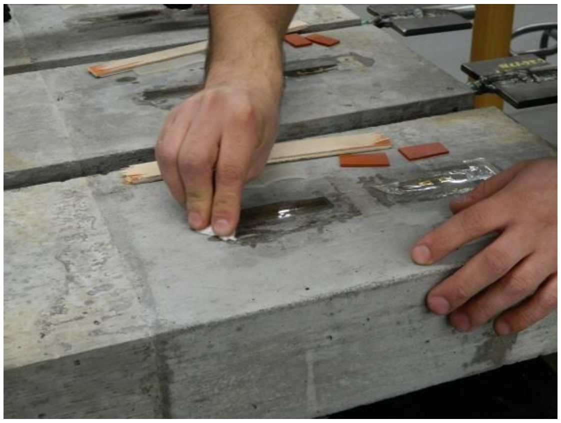

Interface strain gage placed at center of closure concrete. (Interface at 5 in. from top of sample)


2 in. strain gage.

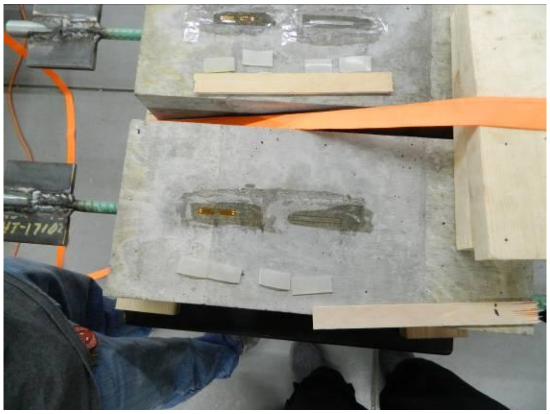

½ in. strain gages.

Applying adhesion to back to back of strain gages.

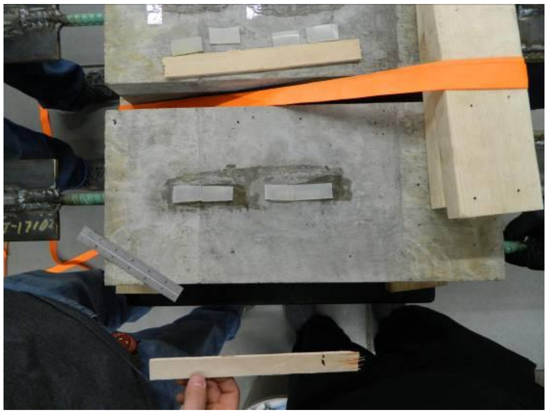
Placing strain gages.



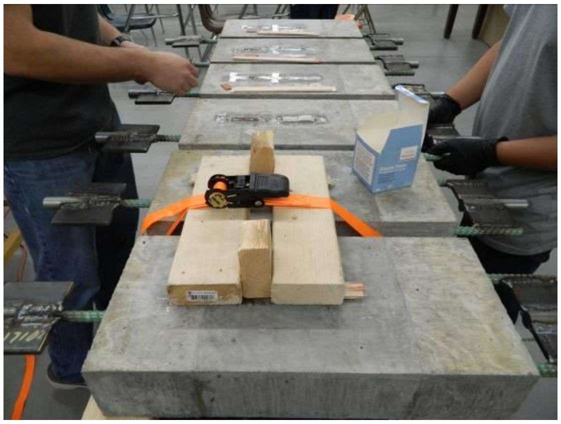
Pressing down gage by wiping with a gauze pad.



After strain gage placement.



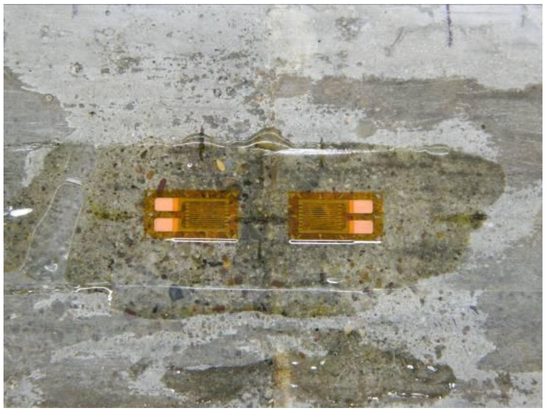
Placing silicon pads and wood strip over gages.



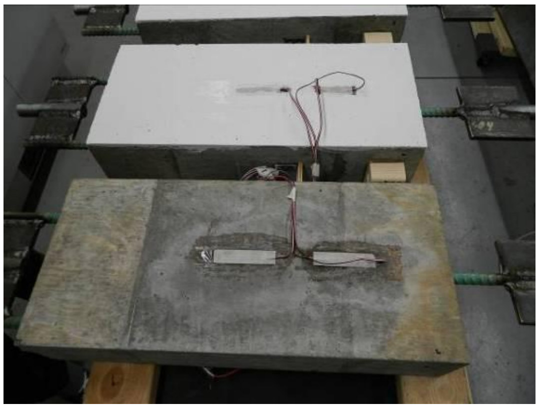
Applying pressure on gages using a tie-down strap.

HB specimen strain gage attached.

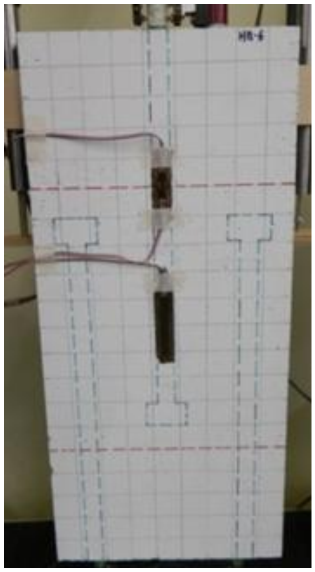
HB 2 in. concrete strain gage.



HB Interface concrete strain gages.



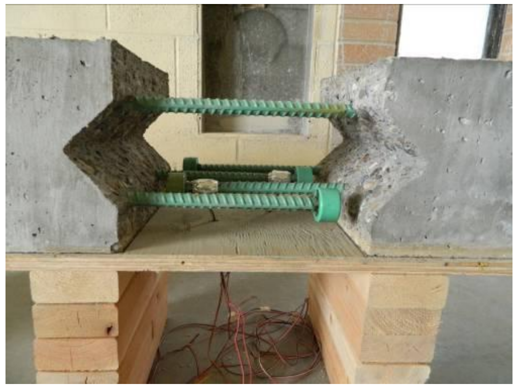
HB specimens being painted.



HB specimen marked with 1 in. x 1 in. grid. Rebars and interface outlined.

M.11 LB Specimen Instrumentation and Preparation

Setup for pouring closure concrete between the two precast beams.

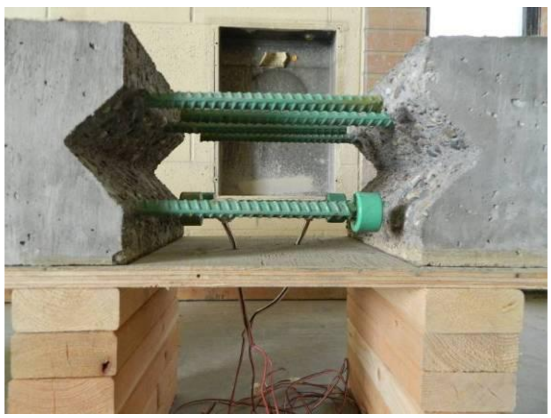


Side view of closure connection.

Rebars spaced 3 in. on center.

View of closure pour connection.

Strain gage wires and protective tubing set through the formwork.



Showing the strain gage wires extending down through the formwork.

10 in. between precast interfaces.

Headed rebars located 1 in. from the interface.

Closure pour concrete formwork.

Beam wrapped in plastic after pouring concrete.

LB specimens curing.

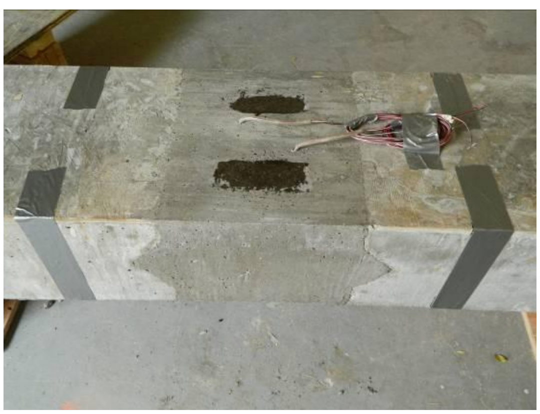
Closure pour concrete cast.

Gage areas marked on bottom of
LB specimen.

Gage areas grinded.

Gage area after grinding with a few
small voids.

Gage area after grinding with many
small voids.

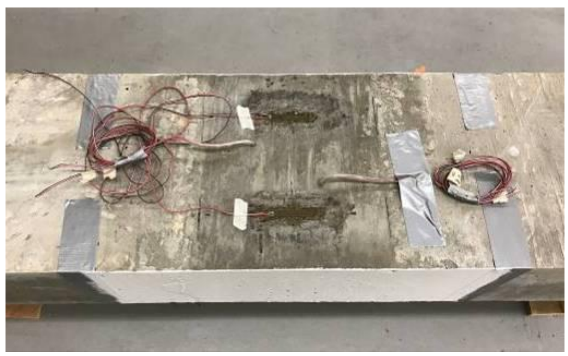


Base coat of M-Bond AE-10 applied
to gage areas.

Before grinding/sanding epoxy base
coat (left) and after (right).

LB concrete strain gages.

LB concrete strain gage placement.

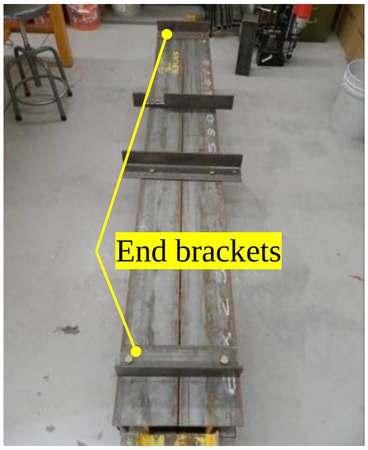


LB concrete strain gages attached.

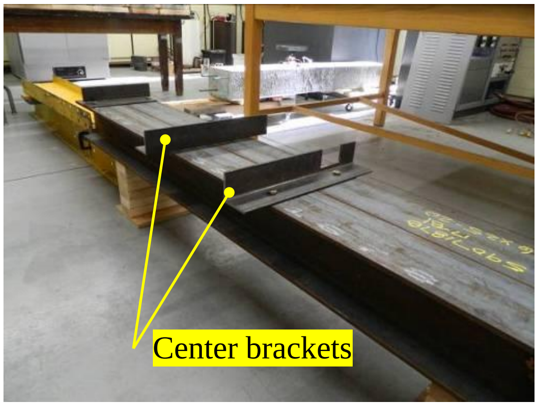
Side of beam painted white
(Only center portion).

Interface between precast and closure
concrete traced with a green marker.

M.12 Beam Test Set-Up

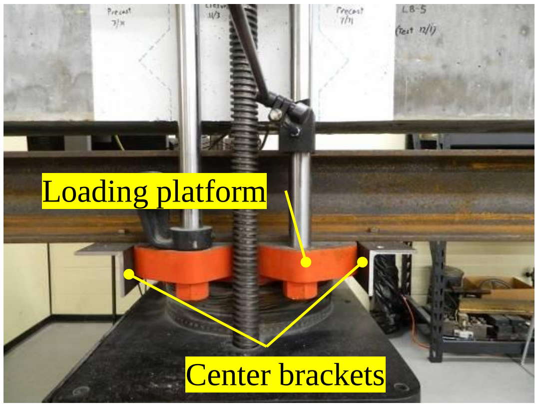


Steel beams connected with sections of steel angles using nuts and bolts. (Shown upside down)



Center brackets. (Shown upside down)


Tinius Olsen Testing Machine.



Center brackets securing steel beams to loading platform.


Back side.


Front side.

Extension beam (yellow) used for inserting and removing concrete beams.

Extension beam connected to support.

Extension beam connected to steel beams.

Extension beam attached.

Extension beam removed.

Extension beam and support bolted.

Extension beam removed and support unbolted from bracket.

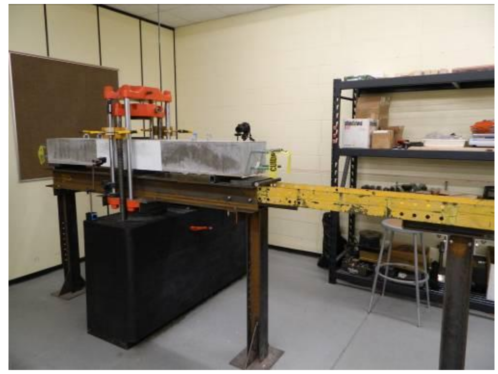

Lifting beam with shop crane.


Lifting beam with shop crane.

Lowering beam onto frame.

Concrete beam on frame.

Concrete beam before setting in Tinius Olsen.



Concrete beam set in place using steel tube rollers.

Test setup.

Test setup.

Back support.

Front support.

Roller support for concrete beam.