SHEAR STRENGTHENING OF REINFORCED CONCRETE BEAMS
USING CFRP

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ABSTRACT

A strengthening system of Carbon Fiber Reinforced Polymer L-Shaped Plates (CarboShear-L) is one of the methods which can be used to increase the shear capacity of existing reinforced concrete bridge girders. This research reports on a series of tests performed on two small scale T-beams. The objective of the tests was to evaluate the performance of the CarboShear-L shear strengthening system when applied to concrete beams with existing shear cracks subjected to cyclic loading. The first T-beam was tested under increasing cyclic shear loads as a control specimen. The second T-beam was loaded following the same routine as the control specimen until significant shear cracks formed in the test shear span. CarboShear-L stirrups were then installed over the shear cracks, without any repair of the cracks themselves. The retrofit was effective at controlling opening of the existing shear cracks and reducing the strain in the internal steel stirrups during subsequent cyclic loading. Under monotonic loading to failure, both T-beams failed in shear, with a 41% shear capacity increase provided by the CarboShear-L stirrups.

An important concern regarding FRP shear strengthening systems is the potential delamination caused by cyclic loading and the movement of existing cracks over extended periods. The anchorage of the CarboShear-L stirrups at both top and bottom of the beam web was adequate to prevent stirrup failure even after delamination of the vertical leg of the stirrup from the concrete surface. In addition, three double shear bonding tests were performed to study the performance of the bond between FRP and the concrete surface, particularly adjacent to cracks in the concrete.
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CHAPTER 1. INTRODUCTION

1.1 Background

The Salt Lake Boulevard Bridge over Halawa Stream in Honolulu, Hawaii is a 3-span, prestressed concrete bridge. During a recent visual inspection, hairline shear cracks were found in 16 girders. Subsequent analysis showed that many of the girders had inadequate shear capacity based on current AASHTO standards (Riggs et al., 2002). The City and County of Honolulu plans to strengthen the girders in shear using an FRP composite system. One of the systems under consideration is the use of Carbon Fiber Reinforced Polymer L-Shaped Plates (Sika CarboShear-L stirrups). One objective of this research study was to evaluate the performance of CarboShear-L stirrup strengthening system when applied to concrete beams with existing shear cracks.

An important concern regarding FRP shear strengthening systems is the potential delamination caused by dynamic loads and the movement of existing cracks over extended periods. High bond transfer stresses will develop immediately adjacent to existing cracks. Repeated loading may lead to bond deterioration and subsequent debonding of the FRP stirrups. Another objective of this research was to study the bond between FRP and the concrete surface, particularly adjacent to cracks in the concrete.

In order to monitor potential delamination of the FRP shear strengthening to be used in the field application on the Salt Lake Boulevard Bridge, an instrumentation and monitoring system is being developed as a separate study. As part of this monitoring system, fiber optic strain gages are proposed to detect initiation of FRP delamination. In
the current study, electrical resistance strain gages were attached to the surface of the FRP stirrups to evaluate their effectiveness for detecting initiation of delamination.

Two small scale T-beams were constructed and tested at the University of Hawaii at Manoa Structural Testing Laboratory to evaluate the performance of Sika CarboShear-L stirrups. One specimen was tested as the control beam to determine the section shear capacity. The second T-beam was loaded cyclically to produce shear cracks in the beam web. The beam was then retrofitted using Sika CarboShear-L stirrups. The retrofitted T-beam was tested under increasing cyclic loading until shear failure. Both T-beams failed in shear, with a 41% shear capacity increase provided by the CarboShear-L stirrups.

In order to study the bond behavior between FRP and concrete three double-face shear bonding tests were performed. These specimens varied in bonding length, and surface preparation.

1.2 Previous Research

During the past decade, fiber reinforced polymer (FRP) has become one of the most popular materials for strengthening and retrofit for existing RC structures. This is due to the advantages of FRP composites, including corrosion resistance, high ratio of strength to weight and easy site handling.

The evaluation of performance of CarboShear-L stirrups is the extension of prior research performed by A. Agapay (Agapay and Robertson, 2003) and J.B. Chen (Chen and Robertson, 2004). These prior studies provide an extensive overview of previous research on FRP retrofit of RC beams and use of the CarboShear-L system.

No matter which method is chosen for installation of FRP, the basic approach is to bond the FRP to the surface of the RC member using resin or epoxy in-situ to make the
member and FRP work as a composite structure. The anchorage between concrete and FRP plays a very important role in the performance of the retrofitted structure. For beams strengthened for flexure, five types of failure modes were observed (Teng, 2002) and termed as (1) flexural failure by FRP rupture, (2) flexural failure by crushing of compressive concrete, (3) shear failure, (4) plate-end debonding failure and (5) intermediate crack-induced interfacial debonding failure. Four failure modes have been observed when RC beams were strengthened for shear (Teng, 2002); shear failure with FRP rupture; shear failure without FRP rupture; shear failure due to FRP debonding; and local failures. Among all these failure modes, FRP debonding often controls the failure mechanism. This section describes some of the research efforts that have been carried out on the mechanics of bonding between FRP and concrete both experimentally and theoretically.

Generally speaking, three types of bonding test methods have been used by researchers to study the bonding behavior between concrete and FRP, including direct tensile tests, shear tests and bending tests as shown in Figure 1.1 (Nabaka, 2001). The direct tensile test (Figure 1.1a) is commonly used in field applications for quality control of the epoxy bond and concrete surface preparation. The double-face shear test (Figure 1.1b) consists of two concrete prisms connected by FRP bonded on two opposite surfaces of the concrete prisms. Tension force is applied by pulling the reinforcing bars embedded in the concrete prisms. The single-face shear test (Figure 1.1c) reduces the material required for each test (compared with the double-face shear test) but requires that the concrete block be restrained against rotation. The bending test consists of two separate concrete blocks connected by FRP on the tension face of the beam. Alternatively a single
concrete block can be used with a saw cut at the middle of the bottom surface. Transverse FRP wrapping is usually applied to one side of the specimen to force failure to occur on the other side. For the inserted shear type specimen, the FRP is sandwiched between two concrete blocks. Two steel plates are glued on the sides of the concrete block and the tension force is applied to the steel plates.

![Bonding Test Specimens](image)

**Figure 1.1: Bonding Test Specimens (Nabaka et al., 2001)**

The effect of test methods and concrete strength on the bonding behavior of CFRP sheet was studied by Horiguchi and Saeki (1997). Three types of bonding tests were conducted with different concrete strengths. High modulus carbon fiber reinforced polymer (CFRP) sheets were used. Concrete surface was dried and cleaned carefully. The
target compressive strengths of concrete were 10, 30 and 50 Mpa. The size of shear type concrete specimens was 100 x 100 x 200 mm. For shear type tests, the width of CFRP was 75 mm and the bonding lengths were 40 mm, 100 mm and 200 mm. For bending type test, the size of concrete block was 150 x 150 x 200 mm. The width of the CFRP sheet was 75 mm and the bonding length was 100 mm. Concrete block size of 150 x 150 x 200 mm and bonding area of 40 x 40 mm were used for the direct tensile tests. They found the ultimate load increases when the bonding length increases while the average stress decreases. For the three kinds of bonding tests, the tensile tests produced the largest average bonding stress while the lowest values were obtained from the shear type tests. Three types of failure mode were observed, shearing of the concrete, debonding, and FRP rupture. The concrete compressive strength played a big role in the bond strength between FRP and concrete. The higher the compressive strength of the concrete, the higher the average bond stress. When concrete strength was low, say 10.5 Mpa, failure occurred in the concrete. Delamination happened when the concrete strength was high or shear type tests were used. Concrete fracture also occurred for high strength concrete when direct tensile tests were conducted. In the case of low compressive strength concrete the average bond stress from all three types of bonding tests were virtually identical. An empirical equation to estimate the average bonding stress was developed based on these test results.

Meada et al. (1997) conducted a total of 37 double-face shear tests. The size of concrete prism was 100 x 100 mm. Width of CFRP was 50 mm. The surface of the concrete was prepared using a sander and primer. Strain gages were placed on the CFRP at a spacing of 10 mm. Three failure modes were observed: FRP delamination, FRP
rupture and concrete fracture. Ultimate loads did not change significantly when the bonding lengths were greater than 100 mm. The maximum load increases as stiffness of the FRP increases. The strain distribution follows a quadratic curve at the early stages of loading, but becomes a bi-linear curve at the ultimate stage (Figure 1.2). When the load is relatively small, bond transfer occurs over a small area adjacent to the joint in the concrete blocks. The effective length, Le, of FRP sustains the entire load until the bond stress exceeds the bond capacity. Then FRP peeling occurs, which causes delamination of the initial bond transfer length and a shift in the effective bonding length (Figure 1.2). Equations to calculate the effective bonding length and ultimate bonding load were developed based on the bi-linear strain relationship. Effective bonding length decreases as the stiffness of CFRP increases. A non-linear finite element analysis was also performed to simulate the strain distribution in CFRP.

![Figure 1.2: Schematic Strain Distribution (Meada et al., 1997)](image)

Ueda et al. (1999) carried out an experimental study on bond strength of continuous fiber sheet. The aspects considered in this study include bonding length, width of FRP, stiffness of FRP, loading condition (with/without eccentricity), and method of anchorage (with/without mechanical anchor). Two types of carbon FRP were used, namely high
strength and high modulus CFRP. Five series of shear type tests, designated as A, B, C, D, and E, were conducted. The surface of the concrete blocks was prepared using a disk sander. A total of 28 specimens were tested. In type A and C tests, tensile load was applied through steel reinforcing bars embedded in the concrete block to which the CFRPs were attached. Type B was single-face shear type test. For type D and E, hydraulic jack was placed between concrete blocks to apply tension force to CFRP. Type A is ordinary double-face shear type tests. There were two jacks placed between concrete blocks in type D. These two jacks were used to apply non-uniform stress in FRP. In type E the two ends of FRP were anchored by steel plates bolted to the concrete base. They found: (1) bonding strength doesn’t increase with the increase of bond length for bonding length longer than 100mm; (2) the maximum local and average bonding stresses at delamination increase and CFRP strain gradient decreases when CFRP stiffness increases; (3) the bonding strength of CFRP increases with the decrease of width of CFRP; (4) non-uniform loading decreases the bonding strength and for non-uniform loaded FRP the delamination starts when the maximum strain in FRP reaches the maximum strain when FRP subjects to uniform loading; (5) anchor steel plate with tensioned bolt enhances the bonding strength.

Brosens and Van Gemert (1999) carried out 24 shear type bonding tests. The specimen type was inserted shear type as shown in Figure 1.1(e). The size of the concrete prism used was 150 x 150 x 300 mm. Two widths, 80 mm and 120 mm, and two bonded length, 150mm and 200 mm, of the CFRP laminates were adopted for test. Each configuration consisted of two specimens. The surface of the concrete prisms was sandblasted to remove the weak concrete top layer. The CFRP sheets used were Forca
Tow Sheets FTS-C1-30. And the wet lay-up method was used to apply CFRP on the concrete surface. A theoretical model to calculate the effective bonding length and the maximum bonding strength based on nonlinear fracture mechanics techniques was provided.

The influence of bonded length, concrete strength, and number of plies of CFRP on the bonding behavior between FRP and concrete was studied by Miller and Nanni (1999). Bending type specimen was chosen to avoid cracking in the bonded length. The carbon fiber sheets used in the test were MBrace CF-130 which is a unidirectional fiber tow sheet. Three kinds of resins, primer, putty and saturant, were used in the application of CFRP to concrete. The specified concrete strengths were 3000 and 6000 psi. The widths of CFRP used were 50.8 mm and 101.6 mm. The concrete surface was sandblasted. The number of plies was one and two. Three different lengths of CFRP were used. There were two repetitions for each case. Some extra specimens were tested too. One of the extra specimens had been roughened by adding notches on concrete surface using a hammer and chisel. They reported that the bonding length had no effect on the bonding strength since the bonded lengths were more than the effective bonding length. The concrete strength did not have an effect on the bond strength. The surface preparation had more impact on the bonding strength. The roughing of concrete surface improved the average bonding strength. The number of CFRP plies increases the average bonding strength, but the increase is not proportional to the number of plies. The width of CFRP sheet has no effect on the average bonding strength.

Nakaba et al (2001) performed both experimental tests and theoretical analysis of the bonding behavior between FRP and concrete. A double-face shear type test was adopted.
The primary test variables were the types of fiber (FRP stiffness), concrete strength, and putty thickness. The size of concrete prism was 100 x 100 x 600 mm. After reinforcing with FRP laminates the concrete prism was cracked at the center using a hammer on the notch. The width of the laminates was 50 mm and the bonding length was 300 mm. The fiber used included standard carbon fiber, high-stiffness carbon fiber and aramid. There were three specimens in each combination of concrete/mortar-fiber. A total of 36 specimens were tested. LVDTs were used to measure the total displacement and the crack width at the center. Strain gages were installed on one side at an interval of 15 mm and one strain gage was glued on the other side at the center of specimen. Most of the specimens failed in bonding and several failed by FRP rupture. The maximum load increases as the stiffness of FRP increases. Putty thickness has no effect on the maximum load. The increase of concrete strength makes an increase in the local bond stress while the stiffness of FRP has no effect on it. A local bond stress - slip model was proposed based on Popovics’s equation. The maximum loads and effective bonding lengths were calculated using proposed model. They are in good agreement with their test results.

Harmon et al. (2003) thought that the properties of the resin layer between surface-mounted FRP and a concrete substrate are critical factors in determine bonding strength. They performed two experimental investigations, one for determining the effect of bond layer properties on the bonding performance using bending type bonding tests and one for verify the design equations they proposed using beam tests. The concrete block used was 610 mm long x 305 mm high and 305 mm wide. The width of FRP strip was 50.8 mm. Nine bonding tests were carried out. The test parameters were the thickness of bonding layer, the shear modulus of the bonding layer resin system, the thickness and
stiffness of the FRP layer, the concrete strength, and the bonding length. They found the thickness and the shear modulus of the bond layer are critical to the bond performance which is controversial to the conclusion from Nakaba et al. The bond stress is proportional to $\sqrt{f_c}$. Five beams reinforced with conventional reinforcement plus CFRP as flexural reinforcements were also tested. The five beams differed from each other by different fiber systems and different resin systems. An analytical model was proposed and used to calculate the tested beams.

Though some research has been done on bonding, there is more work need to do. There are conflict conclusions from different researchers. The strain distribution in FRP for Sika pultruded carbon fiber reinforced polymer laminate is needed for the main project - “Instrumentation and Monitoring the Performance of the FRP Shear Strengthening of the Salt Lake Boulevard Bridge.”

1.3 Objective

The objectives for this research are summarized as follows:

1. To evaluate the performance of Sika CarboShear-L stirrup applied on shear cracked concrete beam. Two nominally identical T-beams were tested. One was the control beam to determine the section shear capacity. The other was loaded to form shear cracks, then retrofitted using Sika CarboShear-L stirrups and finally loaded to failure. For the second beam ten load cycles were performed both for the initial cracking test and the retrofitted beam test. All the results from the tests are presented, compared and discussed.
2. To study the bonding behavior between FRP and concrete. Three shear type bonding specimens were tested. Test results are presented and discussed.
CHAPTER 2. T-BEAM TEST DESCRIPTION

2.1 Introduction

Two nominally identical reinforced concrete T-beams were tested at University of Hawaii at Manoa Structural Testing Laboratory. One, designated as T-1, served as the control beam to determine the shear capacity of the T-beam without retrofit. The second, which was designated as T-2, was subjected to cyclic loading to a shear force of 48 kips so as to induce shear cracks in the test shear span. The cracked T-2 was then retrofitted in shear using CarboShear-L CFRP external stirrups. The retrofitted beam was under the same cyclic loading as the control beam and finally loaded to failure.

This chapter describes the design of specimens, the material properties of all materials used for this research, the specimen preparation and the test setup, instrumentation and test procedure.

2.2 T-Beam Description

The two T-beams tested in this program were designed specifically to evaluate the shear retrofit performance of CFRP CarboShear-L stirrups when applied over existing shear cracks. The beams were nominally identical, with the same reinforcement layout and poured from the same concrete batch. The reinforced concrete beams were designed to be tested under three-point loading with shear failure in one shear span only. Half of each beam was designed with no web thickness reduction and additional internal shear reinforcement to preclude a shear failure in this span (Figure 2.1 BB). The test shear span represents a typical bridge girder cross-section with bottom bulb to accommodate placement of tension reinforcement (prestressed, and/or non-prestressed steel), a
relatively thin web section with internal shear reinforcement, and a top slab representing the bridge deck slab (Figure 2.1 AA). The tension reinforcement was designed to preclude a flexural failure.

2.3 Design of Specimen

The nominal shear capacity of the test shear span without FRP retrofit is 41 kips with 15 kips contributed by the concrete section and 26 kips by the internal steel stirrups.

Figure 2.1: T-beams T-1 and T-2 Details
When CarboShear-L stirrups are installed spaced at 10 inch on center as external shear reinforcement, the shear capacity of the T-beam increased by 16 kips if the wrapping scheme is considered as 3-sided “U-wrap” or 25 kips if it is treated as completely wrapped according to the ACI 440 design recommendation. The total nominal shear capacity of the retrofit shear span is then 57 to 67 kips, representing a total beam load of 114 to 134 kips. The nominal shear capacity of the reaction shear span is 191 kips, which ensures that shear failure will occur in the test shear span. The nominal flexural capacity of the beam is 681 ft-kip which represents a beam load of 382 kips. This far exceeds the test shear span capacity thereby ensuring a shear failure.

### 2.4 Material Properties

#### 2.4.1 Concrete

The normal weight concrete used in both T-beams was provided by a Ready Mix concrete supplier from a single batch. Three 12x6 inch cylinders were tested according to ASTM C39-99 to determine the concrete compressive strength (Table 2.1). These tests were performed two weeks after the final test of the second T-beam.

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>P(k)</th>
<th>Area</th>
<th>fc (psi)</th>
<th>fc (Avg.) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>253</td>
<td>28.11</td>
<td>9001</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>270</td>
<td>27.95</td>
<td>9642</td>
<td>9110</td>
</tr>
<tr>
<td>3</td>
<td>243</td>
<td>27.92</td>
<td>8686</td>
<td></td>
</tr>
</tbody>
</table>

#### 2.4.2 FRP

Sika CarboShear-L stirrups were used to strengthen the cracked T-beam T-2. CarboShear-L stirrups are fabricated using a pultruded carbon fiber reinforced polymer
(CFRP) laminate designed for strengthening concrete structures. The stirrups are bonded to the concrete surface as external reinforcement using Sikadur 30 epoxy resin as the adhesive. The properties of Sika CarboShear-L and Epoxy Sikadur 30 as provided by the manufacturer are listed in Table 2.2 and Table 2.3.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value from manufacturer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength (psi)</td>
<td>$3.80 \times 10^5$</td>
</tr>
<tr>
<td>Modulus of elasticity (psi)</td>
<td>$22.48 \times 10^6$</td>
</tr>
<tr>
<td>Elongation at break</td>
<td>1.69%</td>
</tr>
<tr>
<td>Width (inch)</td>
<td>1.575</td>
</tr>
<tr>
<td>Thickness (inch)</td>
<td>0.047</td>
</tr>
<tr>
<td>Ultimate Tensile Force (kip)</td>
<td>28</td>
</tr>
</tbody>
</table>

**Table 2.3. Material Properties for Epoxy Sikadur 30**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value from manufacturer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Strength (psi)</td>
<td>2175</td>
</tr>
<tr>
<td>Static E-Modulus (psi)</td>
<td>$1.856 \times 10^6$</td>
</tr>
<tr>
<td>Adhesive Strength (psi)</td>
<td>580 (Concrete Failure)</td>
</tr>
</tbody>
</table>

2.4.3 **FRCC Infill**

FRCC (Fiber Reinforced Cementitious Composite) was used to form infill blocks for installing the CFRP external stirrups across the recessed web (Figure 2.26). Mixture design and strength are shown in Table 2.4 and Table 2.5. Four 8x4 inch cylinders were tested to determine the FRCC compressive strength.

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (lb/ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>23.77</td>
</tr>
<tr>
<td>Fly Ash</td>
<td>47.52</td>
</tr>
<tr>
<td>Sand</td>
<td>33.26</td>
</tr>
<tr>
<td>Water</td>
<td>17.59</td>
</tr>
<tr>
<td>Super Plastic</td>
<td>0.47</td>
</tr>
<tr>
<td>Fiber (PVA)</td>
<td>1.62</td>
</tr>
</tbody>
</table>
Table 2.5. Strength for FRCC

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>P (kips)</th>
<th>Area(in²)</th>
<th>fc (psi)</th>
<th>fc (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>78.00</td>
<td>12.45</td>
<td>6263</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>98.75</td>
<td>12.43</td>
<td>7943</td>
<td>7249</td>
</tr>
<tr>
<td>3</td>
<td>97.50</td>
<td>12.44</td>
<td>7840</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>88.63</td>
<td>12.43</td>
<td>7129</td>
<td></td>
</tr>
</tbody>
</table>

2.5 Specimen Construction

The two T-beams were constructed at the Structural Testing Laboratory at the University of Hawaii at Manoa. Concrete was supplied by a local Ready Mix concrete company. Figure 2.2 and Figure 2.3 show the internal steel stirrups in the test span and the reaction span, respectively. To ensure shear failure, the beams were designed with a high flexure capacity, resulting in significant tension reinforcement. The beam span did not provide adequate develop length for the reinforcement, so anchorage was provided by means of steel plates welded to the end of the bottom reinforcement (Figure 2.4). Figure 2.5 shows the reinforcing steel cage in the formwork prior to concrete placement. Figure 2.6 shows a beam after concrete placement. Red dye was added to the concrete for another unrelated application poured from the same concrete batch. It has no effect on the properties of the concrete.
Figure 2.2: Stirrups in Test Span

Figure 2.3: Stirrups in Reaction Span

Figure 2.4: End Anchorage for Flexural Reinforcement
Figure 2.5: Steel Cage in Form

Figure 2.6: Beam after Pour
2.6 FRP Installation

T-beam T-1 was tested without any CFRP stirrups. T-beam T-2 was subjected to cyclic loading to induce shear cracking. CarboShear-L stirrups were then installed on this cracked beam as described in this section.

2.6.1 Fabrication and Installation of FRCC Infill Blocks

2.6.1.1 Fabrication of FRCC Infill Blocks

To provide continuous bond between the CFRP and the concrete beam web, infill blocks were used to fill the recessed web area. In order to minimize the weight added to the beam, the width of infill blocks was just sufficient to provide a bonding surface for the CarboShear-L stirrups (Figure 2.26).

Based on the results of previous tests performed by Chen and Robertson (2004), FRCC infill blocks perform better than concrete blocks. This is attributed to the superior tensile performance of FRCC when compared with regular concrete. The FRCC is designed to induce numerous hairline cracks when subjected to tension. This results in improved bond behavior between FRCC and internal steel reinforcement, or external FRP reinforcement as used in the current application. The FRCC infill blocks were cast in wood forms. The rectangular cross section of each block was 2.5 inch by 3 inch. After moist curing for one week, the blocks were cut to required shape (Figure 2.7) to match the slope on the bottom bulb of the T-beam.
2.6.1.2 Surface Preparation of T-Beam and FRCC Infill Block

To provide a suitable contact surface for the epoxy bond between the infill blocks and the concrete beam, the contact surfaces were roughened. The FRCC infill blocks were roughened using a sand blaster. The condition of the roughened surface is shown in Figure 2.8.
A pneumatic needle gun was used to roughen the contact surface of the concrete T-beam (Figure 2.9). Only areas where the T-beam would be in contact with the infill blocks and CarboShear-L stirrups were roughened (Figure 2.10).

![Figure 2.9: Roughened Concrete T-beam Surface](image)

The bottom corners of the T-beam were rounded using a concrete grinder to match the bend in external CarboShear-L stirrups (Figure 2.10 and Figure 2.11).

![Figure 2.10: Roughened Surface of T-beam](image)
Immediately prior to installation of the filler blocks and FRP stirrups, the roughened surfaces on the FRCC filler blocks and T-beam were cleaned using a high pressure air hose to remove loose dust.

**Figure 2.11: Rounded Corner of T-Beam Bottom Bulb**

2.6.1.3 Installation of FRCC Filler Blocks

Sikadur 30 two-part epoxy was used to bond the FRCC filler blocks to the concrete T-beam. A thin layer of Sikadur 30 epoxy was applied to the contact surfaces of the filler blocks and T-beam. The filler blocks were then pressed manually into place, forcing excess epoxy out the sides of the joint. Figure 2.12 shows the FRCC filler blocks after installation. The contact between FRCC filler blocks and T-beam is shown in Figure 2.13. Care was taken to ensure that the surface of the FRCC infill block aligned with the T-beam bulb to provide a smooth surface for the CarboShear-L stirrup.
2.6.2 Anchorage of CarboShear-L Stirrups

One objective of this research was to evaluate the effectiveness of end anchorage of FRP. The top of the CarboShear-L stirrups were anchored into the compression concrete slab. Slots were cut through the beam slab using a one inch diameter concrete coring
machine (Figure 2.14). Three holes were drilled to produce a slot with approximate size of 1 inch by 2.6 inch (Figure 2.15). The end of the CarboShear-L stirrup was embedded in Sikadur 30 epoxy in the slot to provide anchorage.

The minimum anchorage length recommended by the stirrup manufacturer is 4 inch (100mm). For the 4 ½ inch thick slab in this study, the slots were cut through the entire slab thickness. Figure 2.16 shows the layout of these slots on the beam slab.
2.6.3 Preparation of CarboShear-L Stirrups

The FRP external stirrups used for this study were Sika CarboShear-L 4/50/100 stirrups. This designation means the stirrup is 40 mm wide (1.57 inch) and the lengths of the two legs are 500mm (19.7 inch) and 1000mm (39.4 inch) respectively. The two legs were cut to the appropriate length to suit the T-beam application.

The day before installation of the stirrups, the top end of each stirrup was coated with Sikadur 30 epoxy over the anchorage zone to improve the anchorage as recommended by the manufacturer (Figure 2.17). Both surfaces of the Sika CarboShear-L stirrup are covered by a transparent peel-ply to provide protection after manufacture. The peel-ply was removed from the 4 inch anchorage length. MEK solvent was used to clean the exposed surface. Sikadur 30 epoxy was applied to both sides of the stirrup with a serrated towel to form ridges transverse to the stirrup leg (Figure 2.18).
2.6.4 Installation of CarboShear-L Stirrups

CarboShear-L stirrups were installed on the T-beam following the procedures described below.

The FRP stirrups on the back side of the T-beam were installed first. Each anchorage slot was filled with Sikadur 30 epoxy. A thin layer of Sikadur 30 epoxy was applied to the roughened surfaces of the FRCC filler block and the bottom of the T-beam. The protective peel-ply was removed from the CarboShear-L stirrup and a thin layer of epoxy was applied to the inner face of the stirrup. The vertical leg of the stirrup was inserted.
into the epoxy-filled slot in the top slab of the T-beam to fully embed the pretreated anchorage length in the slot. A weighted wood board was placed over the top of the slot to retain the epoxy in the slot. The stirrup was then pressed firmly onto the FRCC infill block and T-beam. All CarboShear-L stirrups on the back side of the beam were installed following the same procedure.

The installation of CarboShear-L stirrups on the front side of the T-beam followed the same procedure. The exposed surface of the horizontal legs of the FRP stirrups on the back side of the beam was primed with Sikadur 30 epoxy to provide a bond between the overlapping horizontal stirrup legs under the beam soffit.

The finished installation of CarboShear-L stirrups is shown in Figure 2.19.

Figure 2.19: CarboShear-L Stirrups on T-Beam T-2

2.7 Test Setup and Instrumentation

The T-beams were tested under a single concentrated load applied at mid-span. Beams were simple supported at the ends. The shear force in each shear span is therefore half of the total applied load. The results will be presented in terms of shear force instead of total applied load. The load was applied by a hydraulic actuator supported by a four-
post frame. The loading was controlled by a MTS TestStar II Controller. All readings were recorded by a National Instruments Data Acquisition system controlled by Labview.

2.7.1 T-Beam T-1

One linear variable displacement transducer, designated as LVDT1, was installed at mid-span to record the maximum displacement of the beam. Six electrical resistance strain gages were installed on two of the internal steel stirrups during beam construction as shown in Figure 2.20. The applied load, displacement at midspan, and strain readings from the six strain gages were recorded at ½ second intervals during the entire test. During the test, shear cracks formed in the test shear span. Two crack gages were installed over two of the major shear cracks as shown in Figure 2.20 and Figure 2.21. These crack gages monitored the change in crack width during subsequent loading.
Figure 2.20: Test Setup and Instrumentation for T-1

Figure 2.21: Crack Gage
2.7.2  T-Beam T-2

The test on T-2 was performed in two phases. The first phase involved loading T-2 with the same loading cycles used for T-1 until shear cracks formed similar to the beams in the Salt Lake Boulevard Bridge. This phase was designated as T-2I, with a maximum shear force of 48 kips. The cracked T-2 was then retrofitted in shear using Sika CarboShear-L Stirrups as described earlier. In the second phase, the retrofitted T-2 was loaded cyclically to a shear force of 85 kips and then monotonically loaded to failure. The second phase test of the retrofitted T-2 is referred to as T-2R in the following sections.

2.7.2.1  Test T-2I

The test setup and instrumentation for T-2I is similar to T-1, except that the two crack gages were mounted in slightly different locations (Figure 2.22).
Figure 2.22: Test Setup and Instrumentation for T-2I

Figure 2.23: T-2I before Test
2.7.2.2  Test T-2R

After T-2 was retrofitted with CarboShear-L stirrups, 42 electrical resistance strain gauges were installed on the surface of the FRP stirrups as shown in Figure 2.24 and Figure 2.26. Six strain gages were installed on the vertical leg of each FRP stirrup to monitor axial strain in the FRP. One gage was located over the joint between the FRCC infill block and the beam bottom bulb (e.g., SGF5 on FRP stirrup F3 in Figure 2.26). A second gage was located in the middle of the bottom bulb, 2 ½” below the first gage, and four gages were located at 3” on center above the first gage (Figure 2.26). In addition, two strain gages were attached to the horizontal leg of the front stirrup under the beam soffit (e.g., SGF41 and 42 on stirrup F3, Figure 2.26). LVDTs were used to measure anchorage slip at the top of the FRP stirrups and relative movement between the T-beam and the FRCC filler blocks. One LVDT was bonded to the top of each FRP stirrup to monitor movement between the stirrup and the top slab (Figure 2.25). LVDTs were also bonded to the sides of the center FRCC infill block to monitor joint opening between the FRCC block and the top slab and bottom bulb (Figure 2.24 and Figure 2.26).

The two crack gages were removed during FRP stirrup application, but were reinstalled in the same locations for test T-2R.
Figure 2.24: T-2R before Test

Figure 2.25: LVDT on FRP Stirrup
Figure 2.26: Test Setup and Instrumentation for T-2R
2.8 Test Procedure

The goal of this study was to evaluate the behavior of concrete bridge beams retrofitted in shear with CFRP stirrups. Cyclic loading was selected to simulate cyclic loads due to traffic on the bridge. This section describes the loading levels and number of cycles at each load level for all the tests performed on T-1 and T-2. These load cycle designations were used to label the beam cracks during each test. They are also used in the subsequent presentation of the test results.

2.8.1 T-Beam T-1

T-beam T-1 had a design shear capacity of 41 kips according to ACI (Appendix A).

T-1 was loaded monotonically until shear cracks formed in the test span at a shear force of 21 kips. The beam was unloaded and crack gages were installed on two of these cracks. The beam was then reloaded to the same 21 kips load and unloaded four more times (Table 2.6). These cycles are designated as 21-1 through 21-5. Loading cycles were then performed at load increases of 5 to 10 kips (Table 2.6). Five loading and unloading cycles were performed at each load level. After five cycles at a shear load of 85 kips, the beam was loaded monotonically to failure.

<table>
<thead>
<tr>
<th>Peak Shear Force (kips)</th>
<th>No. of cycles</th>
<th>Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>21</td>
<td>5</td>
<td>21-1, 21-2, 21-3, 21-4, 21-5</td>
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<tr>
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<td>26-1, 26-2, 26-3, 26-4, 26-5</td>
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<td>5</td>
<td>85-1, 85-2, 85-3, 85-4, 85-5</td>
</tr>
<tr>
<td>97</td>
<td>1</td>
<td>Monotonic loading to failure</td>
</tr>
</tbody>
</table>
2.8.2  T-Beam T-2

2.8.2.1  Test T-2I

T-2 was loaded monotonically until shear cracks formed in the test span at a shear force of 18 kips. The beam was unloaded and two crack gages were installed on two of these shear cracks. The beam was then reloaded to a shear force of 18 kips, repeated for four more cycles (cycles 18-1 through 18-5 in Table 2.7). T-2 was then subjected to the same cyclic loading as T-1 for the next four load levels (Table 2.7). After the 48 kips loading cycles, the shear crack pattern on the beam web was similar to that observed on the Salt Lake Boulevard Bridge girders. The beam was then removed from the load frame for installation of the CarboShear-L stirrups.

<table>
<thead>
<tr>
<th>Peak Shear Force (kips)</th>
<th>No. of cycles</th>
<th>Designation</th>
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<tr>
<td>48</td>
<td>5</td>
<td>48-1, 48-2, 48-3, 48-4, 48-5</td>
</tr>
</tbody>
</table>

2.8.2.2  Test T-2R

The last two load levels, applied during test T-2I were repeated in test T-2R to compare the performance of the T-beam with and without the FRP external stirrups. To distinguish these load cycles from the T-2I cycles, they were designated 40-6 to 40-10 and 48-6 to 48-10 (Table 2.8). Test T-2R then continued to follow the same loading cycles used for T-1. After five cycles at a shear load of 85 kips, the beam was loaded monotonically to failure.
<table>
<thead>
<tr>
<th>Shear Force (kips)</th>
<th>No. of cycles</th>
<th>Designation</th>
</tr>
</thead>
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<tr>
<td>137</td>
<td>1</td>
<td>Monotonic load to failure</td>
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</tbody>
</table>
CHAPTER 3. T-BEAM TEST RESULTS

3.1 Introduction

All results for T-beams tests T-1, T-2I, and T-2R are presented in this chapter. The results are presented in terms of crack development, load-deflection, crack gage readings, strain in the internal steel stirrups and external CFRP stirrups, and FRP anchorage slip.

3.2 Test T-1

T-1 served as the control beam to determine the shear capacity of the T-beams without external FRP stirrups. T-beam T-2 is nominally identical to T-1.

3.2.1 Crack Development

The first shear cracks appeared in the web of T-1 when the shear force reached 21 kips. Several minor flexural cracks also appeared through the bottom bulb close to mid-span. T-1 was then unloaded. The angles of these shear cracks relative to a horizontal reference were between 36° and 45° on the front face and between 37° and 49° on the back face of the web. Two crack gages were installed on two of these shear cracks, one on the front face and the other on the back face as shown in Figure 3.1. During the following four cycles at a shear force of 21 kips, the existing cracks extended slightly and additional minor cracks appeared.

The shear cracks continued to extend and new flexural cracks formed during the five cycles of loading at 26 kips, 32 kips and 40 kips (Figure 3.2 and Figure 3.3).
Figure 3.1: Initial Web Shear Cracks at 21 kips

Figure 3.2: Cracks on T-Beam 1 at Shear Force of 26 kips

Figure 3.3: Cracks on T-Beam 1 at Shear Force of 40 kips
At an applied shear force of 48 kips, a new shear crack formed closer to mid-span and new cracks formed through the bottom bulb.

At a shear force of 75 kips, another major shear crack appeared close to the end of the beam.

The last cyclic loading was at a shear load of 85 kips. After these 5 cycles, T-1 was loaded monotonically until it failed at a shear force of 97 kips. The failure resulted from propagation of the shear cracks through the top slab as shown in Figure 3.4.

Figure 3.4: Failure of T-1
3.2.2 Load-Deflection Response

The maximum deflection of T-1 was measured at mid-span using an LVDT as described in chapter 2. Figure 3.5 shows the shear-deflection response for all loading cycles and for monotonic loading to failure. During the 55 kips cycles, it was noted that the hydraulic actuator was not aligned with the beam center line. The actuator has been realigned prior to the 65 kips load cycles. The backbone curve is also plotted in Figure 3.5. T-1 failed in shear at a maximum shear of 97 kips and a mid-span deflection of 0.3 inch. The comparison of deflection of T-1, T-2L, and T-2R are discussed in chapter 4.

Initial shear cracking was observed at 21 kips, compared with the ACI 318 prediction of 15 kips. The failure shear of 97 kips was more than double the ACI 318 predicted value of 41 kips.

![Figure 3.5: T-1 Shear-Deflection Relationship](image_url)
3.2.3 Crack Widths

Two crack gages were installed on two major shear cracks as shown in Figure 3.2. The shear vs. crack width curves for these two cracks are shown in Figure 3.6 and Figure 3.7.

During cyclic loading at shear loads up to 65 kips, there is only a slight increase in the crack width as each new load level is applied. There is almost no increase in crack width during the cycling at a particular load level. During cyclic loading at shear loads of 75 kips and 85 kips, the cracks opened significantly. This was particularly noticeable for crack gage CG1 (Figure 3.6) which was located over a more active shear crack than CG2 (Figure 3.7). The majority of the crack opening occurred during the initial cycle to the new load level, however, the residual crack width continued to increase under each subsequent cycle to the same load level. This indicates that continued cycling at these high shear load level would likely lead to continued opening of the shear crack and eventual shear failure of the beam.
Figure 3.6: Shear vs. Crack Width from CG1 (T-1)

Figure 3.7: Shear vs. Crack Width from CG2 (T-1)
3.2.4 Strain in Internal Steel Stirrups

Three strain gages were installed on each of two steel stirrups as described in chapter 2. Strain readings from SG1 were not recorded reliably by the data acquisition system and had to be discarded. Strain readings from the other 5 strain gauges are plotted in Figure 3.8 through Figure 3.12.

SG2 is located at a major shear crack. The initial formation of the shear crack caused a jump in the strain value in SG2 (Figure 3.8) at loading cycle 21-1. During cycling at a 40 kips, the strain reading exceeded 2000 µε. With each cycle at 55 kips, the strain in steel stirrup at this location continued to increase, with a matching increase in the residual strain under no load. From shear force 65 kips, strain gage failed.

SG3 is at the top of the web and is a distance away from the initial shear cracks. The strain remained below 1000 µε until the shear force reached 75 kips (Figure 3.9). A shear crack formed close to the beam end causing a sudden jump in SG3 strain value at the peak of loading cycle 75-1. Subsequent cycling resulted in peak strain readings exceeding 2000 µε.

SG4 is very close to an initial shear crack. After formation of this crack, the strain readings in SG4 are relatively high. At a shear force of 65 kips, strain in SG4 exceeded 2000 µε. During the first cycle to 75 kips, the steel stirrup yielded. The strain gage failed during subsequent cycling.

SG5 is located close to one of the initial shear cracks. When the shear force reached 65 kips, the strain in SG5 exceeded a strain of 2000 µε. During cycling to 75 kips, the strain increases with each cycle showing that the stirrup had yielded.
SG6 is close to a major shear crack, which results in relatively high strain readings. At a shear force of 65 kip, the strain exceeded $2000 \mu\varepsilon$. The strain readings during cycling to 85 kips show that the stirrup had yielded, despite some erratic readings. The maximum strain reached $9500 \mu\varepsilon$ before beam failed.

Figure 3.8: Shear vs. Strain at SG2 for T-1
Figure 3.11: Shear vs. Strain at SG5 for T-1

Figure 3.12: Shear vs. Strain at SG6 for T-1
3.3 Test T-2I

T-2 was tested in the same manner as T-1 till the 48 kip shear load cycles. The initial shear cracks formed at 18 kips, so the peak shear force for the first five load cycles was 18 kips instead of 21 kips for T-1. Significant shear cracks formed on the T-beam web. After cracked T-2 was retrofitted in shear using Sika CarboShear-L external stirrups, T-2 was loaded following the same cycles as T-1 to failure. All the experimental results from T-2I and T-2R are presented here.

Test T-2I was performed to create shear cracks in the web of the beam similar to those in the Salt Lake Boulevard Bridge. Detailed test setup, instrumentation information and test procedure are provided in chapter 2.

3.3.1 Crack Development

First visible shear cracks appeared when the shear force reached 18 kips (Figure 3.13). The angles of these shear cracks were between 36° to 47° on the front side and 38° to 45° on the back side of the web.

Two crack gages were installed on these cracks and four more cycles with a peak shear load of 18 kips were applied. During the four loading cycles, the shear crack close to the T-beam end extended downward to the beam support. A major flexural crack appeared at mid span. And some minor flexural cracks formed in the bottom bulb.

During the five cycles at peak shear force of 26 kips, the shear crack close to the T-beam end extended upward to the bottom of the beam top slab. Some minor flexural cracks appeared.

At a shear load of 32 kips, there was some minor extension of the existing cracks and more vertical flexural cracks appeared in the bulb (Figure 3.14).
There was some minor extension of existing cracks during the 5 cycles at loading level of 40 kips. When the shear load reached 48 kips, three more major shear cracks appeared in both front and back of the web. After this loading level, the test was stopped. The resulting shear crack pattern is shown in Figure 3.15.

Figure 3.13: First Web Shear Cracks at 18 kips Shear Force (T-2I)

Figure 3.14: Cracks on T-beam 2 at Shear Force of 32 kips (T-2I)
3.3.2 Load-Deflection Response

The deflection was measured at the top of the beam at midspan using LVDT1. The plot of shear force vs. deflection at mid span is shown in Figure 3.16. The comparison between T-1, T-2I, and T-2R will be presented in chapter 4.
3.3.3 Crack Widths

Two crack gages were installed on two major shear cracks right after the initial major shear cracks formed, one on the front side, and one on the back side (Figure 3.13). The relationships of shear force vs. crack width for these two cracks are plotted in Figure 3.17 and Figure 3.18.

During the first loading cycle to a shear force of 26 kips, crack width at CG1 increased with the increasing load. During subsequent cycles to 26 kips, the crack width at CG1 began to decrease. This is due to the formation of a new shear crack nearby, marked on the beam as 26-1 in Figure 3.14. The concrete strain redistributed between the new and existing cracks, which resulted in decreased crack width at CG1. At load cycles 48-3, 48-4, and 48-5, several new shear cracks formed close to CG1, which caused further strain redistribution between cracks and decreased crack width at peak loading compared to the previous cycle.

At load cycle 26-1, CG2 recorded a big increase in crack width (Figure 3.18). The crack width continued to increase with subsequent load cycling and increasing load.

During load cycle 48-5, the crack width at peak loading was 0.0115 inch for CG1 and 0.0233 inch for CG2. The residual crack width was 0.0058 inch for CG1 and 0.0072 inch for CG2.

The test was terminated to allow for installation of the FRP shear retrofit.
Figure 3.17: Shear vs. Crack Width for Crack Gage 1 (T-2I)

Figure 3.18: Shear vs. Crack Width for Crack Gage 2 (T-2I)
3.3.4 Strain in Internal Steel Stirrups

SG1 is located close to a major shear crack (Figure 3.19). When the initial shear crack formed, SG1 saw a sudden increase in strain at loading level 18-1. Then strain at SG1 increased gradually with the increase of loading. At a shear load of 48 kips, the strain in the steel stirrup at this location exceeded 2000 με. The strain increased with each subsequent cycle until failure of the strain gage.

There was a sudden increase in strain at SG2 at loading level 18-1 since SG2 is right at one of the initial major shear cracks close to the beam end (Figure 3.20). At loading level 26-1, this crack extended upward to the top slab, which resulted in another sudden increase in strain reading at SG2. During cycling at a shear force of 48 kips, the steel stirrup strain at SG2 exceeded 2000 με.

Since there was no crack passing through or near SG3, the strain reading at this gage was small for all loading up to 48 kips (Figure 3.21).

Strain readings at SG4 were small until the shear load reached 40 kips for the second cycle. At loading level 40-2 a new shear crack formed close to the location of SG4, which caused a big increase in strain reading in SG4 (Figure 3.22).

SG5 was close to an initial shear crack (Figure 3.23). During the first cycle at a loading level of 48 kips, a new shear crack formed close to SG5 on the back side of the beam. During the second cycle at a shear load of 48 kips, this shear crack appeared at the front side as well. As a result of this crack, strain readings in SG5 recorded a relative big increase at loading levels 48-1 and 48-2. During the second cycle of shear force at 48 kip, the steel stirrup strain exceeded 2000 με.
SG6 was very close to an initial major shear crack, which contributed to the big increase in SG6 reading at loading level 18-1 (Figure 3.24). When the shear force reached 48 kips, the steel stirrup strain exceeded 2000 µε.

Because a number of the internal steel stirrups were at or above the yield strain, the test was stopped so that the FRP shear retrofit could be applied. The last two load levels were then repeated with the FRP stirrups in place so as to compare the retrofit performance with the pre-retrofit beam (cycles 40-6 to 40-10 and 48-6 to 48-10). This comparison is shown in chapter 4.

![Figure 3.19: Shear vs. Strain at SG1 for T-2I](image-url)
Figure 3.20: Shear vs. Strain at SG2 for T-2I

Figure 3.21: Shear vs. Strain at SG3 for T-2I
Figure 3.22: Shear vs. Strain at SG4 for T-2I

Figure 3.23: Shear vs. Strain at SG5 for T-2I
3.4 Test T2-R

After developing the desired shear crack pattern during test T-2I, T-beam T-2 was retrofitted in shear using Sika CarboShear-L CFRP stirrups. The procedures to retrofit T-2 are presented in chapter 2. Test setup and instrumentation are also described in detail in chapter 2.

Cyclic loading to shear force levels of 40 kip and 48 kip were repeated for comparison of T-2R with FRP and T-2I without FRP. The remaining loading cycles used for T-1 were then applied to the retrofitted T-2R. After 5 cycles at the 85 kip load level, T-2R was loaded monotonically to failure in the same manner as T-1. The maximum shear carried by the test span was 137 kips at a midspan deflection of 0.29 inches (Figure 3.24: Shear vs. Strain at SG6 for T-2I)
3.25). Failure occurred when the L-anchor at the base of the CarboShear-L stirrups debonded from the soffit of the beam.

Sika CarboShear-L stirrups increased the failure shear capacity of the section by 40 kips (41%), compared with the 16 to 25 kip (16% to 26%) increase predicted by the ACI 440 Report on externally applied FRP strengthening. The shear failure was sudden, but there was sufficient visual warning through crack formation and widening before final failure.

The maximum strain measured in the Sika CarboShear-L stirrups was 6211 µε, compared with 2644 to 4000 µε predicted by the ACI 440 Report.

![Figure 3.25: Shear vs. Deflection at Midspan for T-2R](image)

**3.4.1 Crack Development**

During load cycle 40-6, one of the existing shear cracks extended. Cracks also appeared on filler blocks F2, F3, B1 and B2. During the following four cycles at a peak
shear force of 40 kips, more cracks formed on the FRCC filler blocks F1, F2, F3, B1 and B2.

During the loading to 48 kip cycles, more cracks formed in filler blocks F1 and F3, and the existing shear cracks extended on both the front and back sides of the beam (Figure 3.26 and Figure 3.27).

During load level 55-1, existing shear cracks extended on both the front and back sides of the beam. New cracks formed in infill blocks F2, B1, B2 and B3. Additional cracks formed during load cycles 55-2 and 55-3.

During the 5 cycles to a shear load of 65 kips, more cracks appeared in filler blocks F1, F2, B1, B2 and B3 and one existing shear crack extended.

At a load level of 75 kips, a new shear crack formed and more cracks appeared in infill blocks F1, F2, F3, B1, and B3.

At a load level of 85 kips, more cracks formed in the infill blocks during each cycle, and a new shear crack formed (Figure 3.28 and Figure 3.29).

The cracks in the filler blocks tend to coincide with cracks in the web. However the cracks in the filler blocks are smaller and more distributed than those in the beam web.

After load cycle 85-5, T-2R was loaded monotonically until failure. T-2R failed at a maximum shear force of 137 kips when the CarboShear-L Stirrups delaminated at the L anchorage at the bottom of the beam (Figure 3.30). At failure, infill blocks F2 and B2 delaminated from the web as shown in Figure 3.31. Figure 3.32 shows close-up views of the L anchorage at the bottom of stirrups F2 and B2 after delamination.
Figure 3.26: Cracks on Beam at load 48 kips for T-2R (Front)

Figure 3.27: Cracks on Beam at load 48 kips for T-2R (Back)
Figure 3.28: Cracks on Beam at load 85 kips for T-2R (Front)
Figure 3.29: Cracks on Beam at load 85 kips for T-2R (Back)

Figure 3.30: Failure of T-2R
3.4.2 Crack Width

The two crack gages were installed at the same locations as for test T-2I. The relationships of shear force vs. crack width for these two locations are plotted in Figure 3.33 and Figure 3.34. The crack widths for T-2R are compared with those measured during T-2I in chapter 4.
Figure 3.33: Shear vs. Crack Width for Crack 1 (T-2R)

Figure 3.34: Shear vs. Crack Width for Crack 2 (T-2R)
3.4.3 Strain in Internal Steel Stirrups

SG1 was damaged during T-2I test. Shear force vs. strain relationships for SG2, SG3, SG4, SG5 and SG6 under T-2R cyclic loading are plotted in Figure 3.35 to Figure 3.39.

During test T-2I, the steel stirrup at SG2 location had already reached a strain of 2000 µε under a loading level of 40 kips. With the CarboShear-L CFRP external stirrups in place, the strain in the steel stirrup reduced to 1500 µε for the same 40 kip loading level. The steel stirrup reached 2000 µε again when the shear force reached 55 kips.

SG3 was not close to any of the initial shear cracks. The strain in the steel stirrup at this location was very small during the T-2I test. The value in SG3 was still small during T-2R test. Steel stirrup at this location reached 2000 µε only during the final loading at a shear force of 116 kips.

The strain in the steel stirrup at SG4 location also reduced due to the addition of CarboShear-L stirrups. The slope of the shear vs. strain curve increased. Steel stirrup reached 2000 µε at a shear load of 85 kips.

The steel stirrup at SG5 reached 2000 µε at a shear force of 48 kips during T-2I test. After the CFRP external stirrups were installed, the strain in the steel stirrup at SG5 location was reduced. The steel stirrup reached 2000 µε again at a load level of 65 kips.

Steel stirrup at SG6 reached 2000 µε during T-2I test at a shear force of 48 kips. The CFRP external stirrups reduced the stress in the internal steel stirrups. The strain at SG6 reached 2000 µε at a shear force of 75 kips.
Figure 3.35: Shear vs. Strain at SG2 for T-2R

Figure 3.36: Shear vs. Strain at SG3 for T-2R
Figure 3.37: Shear vs. Strain at SG4 for T-2R

Figure 3.38: Shear vs. Strain at SG5 for T-2R
3.4.4 FRP Anchorage Slip

Six LVDTs were installed on the surface of the FRP stirrups to monitor any top anchorage slippage of the CarboShear-L CFRP external stirrups. Another four LVDTs were mounted on the sides of the two middle filler blocks to monitor any relative movement between the filler blocks and the T-beam. The LVDT locations and designations are shown in Chapter 2.

During the T-2R test, readings from all LVDTs stayed at their original value except for tiny variations due to signal noise. Based on the results from the LVDTs, it was concluded that no anchorage slippage occurred at the top of the CFRP external stirrups, and no relative movement occurred between the filler blocks and the T-beam until failure of the beam.
3.4.5 Strain in CarboShear-L Stirrups

A total of 42 strain gages were bonded to the CarboShear-L stirrup surface to measure strain in the CFRP stirrups and to monitor any delamination. The locations of these strain gages are shown in chapter 2.

The plots of shear vs. strain in each strain gage are shown in Appendix B. The readings from strain gages on the same FRP stirrup are also plotted in Figure 3.53 to Figure 3.58, for the purpose of investigating the possible delamination of each CFRP stirrup. The final monotonic loading to failure is plotted independently in Figure 3.67 to Figure 3.72. For the plots showing the final loading only, the strain values have been zeroed with respect to the values at the beginning of the final loading procedure.

SGF11 was damaged after three cycles at load level 40 kips (Figure 3.48). This strain gauge was replaced between loading cycles 65-2 and 65-3. Therefore, strain readings for SGF11 from load level 40-8 to 65-2 are not available. The results from all other cycles are shown in Figure 3.48.

All other strain gages performed well throughout the test.

3.4.5.1 FRP Stirrup F1

At a load level of 40 kips, cracks in the beam web extended into the infill block and formed multiple cracks in the infill. These cracks in the filler block resulted in increasing readings in SGF16 and SGF17. SGF18 was located on the bulb of the T-beam. SGF18 measured a larger strain than other strain gages on F1 because a major shear crack passed through the T-beam bulb at this location. At load level 65-5, there was a sudden increase in strain at SGF17 which brought the reading from SGF17 very close to that from SGF-
18 (Figure 3.59). Since there was no new crack appearing near SGF18 and SGF17, the sudden increase of strain in SGF17 indicates delamination between SGF17 and SGF18.

At load level 85-1, a new shear crack formed on the T-beam web, which also extended into the filler block (Figure 3.40). The large strain increase in SGF14 and SGF15 at load level 85-5 might be due to this new shear crack or the delamination of the stirrup from the filler block (Figure 3.60).

The sudden increase in the reading from SGF13 at load level 85-1 is due to the new shear crack. Because there is still a large difference in the strain measurements at SGF13 and SGF14, the stirrup had not delaminated between these strain gage locations.
3.4.5.2 FRP Stirrup F2

At a load level of 40 kips, shear cracks on the beam web extended into the FRCC filler blocks, resulting in multiple cracks in the filler block. These cracks cause sudden strain increases in SGF7, SGF8, SGF9 and SGF10 (Figure 3.55). After a loading level of 75 kips, the strain reading in SGF12 began to increase rapidly until it reached the same strain as SGF11 at the peak load of load level 85-5. Subsequently, SGF12 and SGF11 show similar readings (Figure 3.61). Since there is no crack going through or near
SGF12, the change of reading in SGF12 indicated that there was delamination between SGF11 and SGF12. This delamination initiated from SGF11 and propagated towards SGF-12 gradually from load cycle 75-1 to 85-5. At each peak loading of these cycles, delamination propagated a little further which resulted in the relatively big increase in strain reading in SGF12 at each peak loading.

Before loading cycles at 85 kips, the strain reading in SGF11 was smaller than for the strain gauges above. During load cycles 85-1 and 85-2, strain readings from SGF11 increased suddenly at peak loading. The readings from SGF11 and SGF10 were then very similar. This is also an indication of delamination propagating toward SGF10.

![Figure 3.41: Cracks on FRCC Filler Block F2](image)

3.4.5.3 FRP Stirrup F3

During loading level 40 kips to 55 kips, existing shear cracks in the beam web extended into the FRCC filler block. Multiple cracks formed at the top potion of the filler block. These cracks resulted in relatively large strain readings in SGF1, and SGF2. At
this stage, readings from the other strain gauges on this FRP stirrup were small (Figure 3.57).

At a load level of 75 kips, an existing shear crack in the beam web extended, resulting in more cracks in the FRCC filler block. These cracks are close to SGF3. This resulted in a rapid increase in strain reading in SGF3.

During load cycles at peak shear of 85 kips, new shear cracks formed in the T-beam web and extended into the FRCC filler block near SGF4. It can be seen from Figure 3.47 that the strain in SGF4 increased rapidly when the shear force reached 85 kips for the first and second times.

During the final loading at a shear force of 134 kips, a jump in strain was observed for SGF5 and SGF6, which brought their readings very close to that from SGF4. This is an indication of delamination between SGF4 and SGF6 just prior to beam failure.

Figure 3.42: Cracks on FRCC Filler Block F3
3.4.5.4 FRP Stirrup B1

During loading cycles to 40 kips, cracks in the beam web extended into the FRCC filler block. These cracks are between SGF-22 and SGF-23 and caused sudden strain increases in these two strain gages during the first cycle at load level 40 kips.

A new shear crack formed in the T-beam web at a loading level 85 kips. Extension of this crack into the filler block formed multiple cracks between SGF19 and SGF21 (Figure 3.43). This coincides with the rapid increase in strain readings from these three strain gauges at the peak of loading level 85-1. After these rapid strain increases, readings from SGF20 and SGF21 were the same (Figure 3.63), which means there was no bond transfer between SGF20 and SGF21 and delamination had occurred over this section of the stirrup. After the strain increase, the strain in SGF19 was still much smaller than SGF20. This indicates that there was still bond transfer between FRP and FRCC filler block between these two strain gages.

![Left Side](image1) ![Right Side](image2)

**Figure 3.43: Cracks on FRCC Filler Block B1**
3.4.5.5 FRP Stirrup B2

At a loading level of 40 kips, cracks in the beam web extended into the filler block. These cracks are between SGF25 and SGF27 and caused the sudden strain increase in these three strain gages at loading level 40-6 (Figure 3.49, Figure 3.50, and Figure 3.51). Even though no crack is marked on the filler block between SGF25 and SGF26 at loading level 40-6, the crack marked as 48-6 may have formed at loading level 40-6, but was too small to be visible (Figure 3.44).

Strain in SGF28 was relatively small until the peak of load cycle 55-1. At the peak of loading 55-1, the strain in SGF28 increased about 200 µε without increase of shear force (Figure 3.52). This is due to the extension of shear cracks into the filler block. The multiple cracks marked as 65-1 might have happened at loading level 55-1 but were too small to be visible until load level 65-1 (Figure 3.44).

During final testing to failure, when the shear was less than 99 kips, the strain in SGF30 was smaller than SGF29. At a shear force of 99 kips, the reading from SGF30 suddenly increased to the same reading as the neighboring SGF29 (Figure 3.64). This indicates the stirrup had delaminated between these two gage locations.

For shear force greater than 99 kips, the slope of the shear vs. strain curve for SGF29 began to decrease and after 4 relatively big strain increases at shear forces 99 kips, 113 kips, 115 kips and 117 kips, the strain reading in SGF29 agreed with the neighboring strain gage SGF28, indicating delamination had occurred between SGF29 and SGF28 (Figure 3.64).
3.4.5.6 FRP Stirrup B3

At a load level of 85 kips, the strain in SGF33 and SGF34 increased rapidly. At the peak of load level 85-3, the strain in SGF33 increased to the same strain as the neighboring SGF32 indicating that the FRP stirrup had delaminated between these two gage locations as confirmed in Figure 3.45.

During final loading at a shear force of 121 kips, there is a sudden strain increase in SGF35. After two more rapid increases, the reading in SGF35 is the same as that in neighboring SGF34, which indicated that delamination occurred between these two gage locations.
3.4.5.7 Observations

Strain gages attached to the surface of the FRP stirrups were effective at detecting increases in strain due to cracking in the FRCC infill blocks even when these cracks were
still not visible. The strain gages were also effective at detecting FRP delamination before signs of delamination were noted by visual inspection. This was particularly evident when delamination occurred during cycling at a particular load level. Although not yet visible, this delamination is detected by the strain gage readings.
Figure 3.47: Shear vs. Strain for SGF4

Figure 3.48: Shear vs. Strain for SGF11
Figure 3.49: Shear vs. Strain for SGF25

Figure 3.50: Shear vs. Strain for SGF26
Figure 3.51: Shear vs. Strain for SGF27

Figure 3.52: Shear vs. Strain for SGF28
Figure 3.53: Strain on CarboShear-L Stirrup F1

Figure 3.54: Strain on CarboShear-L Stirrup B1
Figure 3.55: Strain on CarboShear-L Stirrup F2

Figure 3.56: Strain on CarboShear-L Stirrup B2
Figure 3.57: Strain on CarboShear-L Stirrup F3

Figure 3.58: Strain on CarboShear-L Stirrup B3
Figure 3.59: Strain on F1 (Partial 1)

Figure 3.60: Strain on F1 (Partial 2)
Figure 3.61: Strain on F2 (Partial)

Figure 3.62: Strain on F3 (Partial)
Figure 3.63: Strain on B1 (Partial)

Figure 3.64: Strain on B2 (Partial)
Figure 3.65: Strain on B3 (Partial 1)

Figure 3.66: Strain on B3 (Partial 2)
Figure 3.67: Strain on F1 (Final Loading)

Figure 3.68: Strain on B1 (Final Loading)
Figure 3.69: Strain on F2 (Final Loading)

Figure 3.70: Strain on B2 (Final Loading)
Figure 3.71: Strain on F3 (Final Loading)

Figure 3.72: Strain on B3 (Final Loading)
CHAPTER 4. COMPARISON AND DISCUSSION FOR T-BEAMS

4.1 Introduction

Test results for T-1, T-2I and T-2R will be compared and discussed in this chapter. T-1 and T-2 are nominally identical reinforced concrete T-beams. Performance of T-1 and T-2 without retrofit (T-2I) will be compared first. The performance of FRP shear retrofit will be evaluated through the comparison of test results from T-2I and T-2R.

4.2 T-1 and T-2I Comparison

4.2.1 Midspan Deflection

The test setup and layout were identical for T-1 and T-2I. The plots for shear vs. midspan deflection for the two tests are shown in Figure 4.1. For T-1 only the portion under load level 48 kips is plotted since the maximum shear load for T-2 was 48 kips.

The shape of the curves for T-1 and T-2 are similar, though T-2 is slightly stiffer than T-1.
4.2.2 Strain in Internal Steel Stirrups

Since strain readings from SG1 for T-1 were not available, only the results from SG2, SG3, SG4, SG5, and SG6 are compared here.

Figure 4.2 shows the comparison for strain gage SG2. At a shear force of 48 kips, SG2 for T-1 and T-2I both recorded about 1500 µε. The shapes of the shear vs. strain curves are very similar.

Figure 4.3 shows that the strains in SG3 were small for both beams. The changes in strain with respect to shear force for SG3 are very close for T-1 and T-2I.

For SG4, the strain in T-2I is smaller than that in T-1 (Figure 4.4). At a shear force of 48 kips, strain at SG4 in T-2I was about 1000 µε while in T-1 it was about 1600 µε. One
of the initial shear cracks in T-1 was very close to SG4, while in T-2, a shear crack only formed near SG4 during the load cycles to 48 kips.

The strain at SG5 in T-1 is smaller than in T-2I, but the shapes of these two curves are similar (Figure 4.5). There is also a smaller residual strain at SG5 in T-1 than T-2. The high residual strain in T-2 indicates that stirrup may have yielded.

The strains at SG6 in both T-1 and T-2I are similar as shown in Figure 4.6.

The strain distributions in the internal steel stirrups in T-1 and T-2 are very similar. The values are all large at SG2, SG5 and SG6 and small at SG3. The shear vs. strain curves for SG2 and SG3 are almost the same for T-2 and T-1. The difference in strain reading from SG4 is attributed to differences in the crack pattern development in the two beams.

Based on the observations above it can be concluded that T-1 and T-2 are nominally identical and the shear capacity of T-1 can be used for comparison with the shear capacity of T-2R with retrofit.
Figure 4.2: Shear vs. Strain at SG2 for T-1 and T-2I

Figure 4.3: Shear vs. Strain at SG3 for T-1 and T-2I
Figure 4.4: Shear vs. Strain at SG4 for T-1 and T-2I

Figure 4.5: Shear vs. Strain at SG5 for T-1 and T-2I
4.3 Comparison between T-1, T-2I, and T-2R

Beam T-2 was loaded cyclically to 48 kips to induce shear cracks (T-2I) and then retrofitted in shear using CarboShear-L stirrups. Two loading levels, 40 kip and 48 kip, were repeated for the retrofitted test (T-2R). These loading cycles can then be compared for T-2I without FRP stirrups and T-2R with FRP stirrups.

T-2R was then subjected to increased loading cycles as used for T-1, followed by monotonic loading to failure. The maximum shear force in T-2R was 137 kips which is 41% higher than the shear capacity of T-1.

4.3.1 Deflection at Midspan

The plots for shear vs. midspan deflection for both T-2I and T-2R are shown in Figure 4.7 for loading cycles at 40 and 48 kip shear force. At the same loading level, T-
2R has less deflection than T-2I. It can be concluded that Sika CarboShear-L stirrups increased the stiffness of the beam. It also can be observed from Figure 4.7 that FRP external stirrups reduced both the deflection at peak loading and the residual deflection when the beam was unloaded.

The backbone envelopes for T-1, T-2I and T-2R are shown in Figure 4.8 along with the cyclic loading for T-2I and T-2R. It can be seen that T-2R is much stiffer than T-2I and T-1. At the maximum shear force of 97 kips, the midspan displacement for T-1 was 0.31 inch, but only 0.18 inch for T-2R. For T-2R, the maximum shear force was 137 kips at a midspan deflection of 0.29 inch.

Figure 4.7: Shear vs. Deflection for T-2I and T-2R
4.3.2 Crack Opening

Shear vs. crack width for T-2I and T-2R are plotted in Figure 4.9 for crack gage CG1 and Figure 4.10 for CG2. It can be observed that the CarboShear-L stirrups reduced the shear crack opening significantly.

For CG2, during the final loading cycle to 48 kips, the crack opening for T-2R was 0.009 inch compared with 0.016 inch for T-2I without FRP stirrups, a 44% reduction.

Crack opening at CG1 was a maximum of 0.011 inch during load cycle 48-1. During subsequent cycles to 48 kips, new shear crack formation reduced the crack opening at CG1. The final cycle at 48 kips (48-5) showed a crack opening of 0.006 inch. After FRP stirrup installation, the crack width opening at CG1 was stable at 0.005 inch.
Figure 4.9: Shear vs. Crack Width at CG1 for T-2I and T-2R

Figure 4.10: Shear vs. Crack Width at CG2 for T-2I and T-2R
4.3.3 *Strain in Internal Steel Stirrups*

The shear vs. strain curves for internal steel stirrups in T-2 before and after retrofit with FRP are plotted from Figure 4.11 to Figure 4.15. Since SG1 was damaged during the T-2I test, only results from the other five gages are compared here. T-2R results are only shown for load cycles to 40 kips and 48 kips for comparison with the same cycles for T-2I.

From Figure 4.11 to Figure 4.15, it can be seen that the FRP stirrups reduce the strain in the steel stirrups, especially where this strain is relatively high, such as locations SG2, SG5, and SG6. During test T-2I, the strain in SG2, SG5 and SG6 had already reached 2000µε, implying that the steel was close to the yielding strain. During test T-2R, at the same loading level of 48 kips shear, the strain in SG2, SG5 and SG6 all reduced below 2000 µε. At a shear force of 48 kips, the reduction in strain was 31% for SG2, 25% for SG5 and 32% for SG6.

By sharing the shear on the section with the internal steel stirrups, the FRP external stirrups reduce the tension stress in the internal steel stirrups, thus keeping the steel stirrup from yielding and reducing further growth of the crack widths under repeated cycling.
Figure 4.11: Shear vs. Strain at SG2 for T-2I and T-2R

Figure 4.12: Shear vs. Strain at SG3 for T-2I and T-2R
Figure 4.13: Shear vs. Strain at SG4 for T-2I and T-2R

Figure 4.14: Shear vs. Strain at SG5 for T-2I and T-2R
Figure 4.15: Shear vs. Strain at SG6 for T-2I and T-2R
CHAPTER 5. DEBONDING TEST

5.1 Introduction

To understand better the bonding between FRP and the concrete beam surface, three bonding tests were carried out in the University of Hawaii at Manoa Structural Testing Laboratory. The first was a trial test which was used to determine whether the specimen would perform as desired. The results from the trial test showed that the specimen would perform as required. The failure was a debonding failure, instead of failure in the concrete or in the test fixture. Two more specimens were constructed based on the trial specimen with certain modifications to improve the test results.

The specimen design, material property, test setup and instrumentation, and test results will be presented in the chapter.

5.2 Specimen Design

As discussed in Chapter 1, there are three types of bonding tests, bending test, shear test, and direct tensile test. Double face shear type tests were used for this research because they replicate the possible delamination between FRP stirrups and a concrete web when concrete beams are retrofitted in shear using FRP.

The test specimens were developed to investigate FRP delamination adjacent to an existing crack in a concrete specimen. Formation of additional cracks along the bonding length was avoided by inducing compression in the concrete. The test fixture is shown in Figure 5.1.

The fixture consisted of a steel rod with a square steel plate welded at one end.
Each specimen consists of two concrete prism blocks connected by surface mounted CFRP (Figure 5.2). The gap between the two concrete blocks simulates the crack in a concrete beam. One concrete block is the test side and the other is the anchor side. To ensure failure would occur in the test side, the anchor side was wrapped with transverse CFRP as shown in Figure 5.2.

By applying tension to the steel rods, the concrete blocks are loaded with uniform distributed compressive stress. The force in each FRP plate would be $P/2$, where $P$ is the total load applied to the specimen.
Figure 5.3: Load Transfer between FRP, Epoxy, and Concrete

The free body diagram of load transfer between FRP, epoxy, and concrete is shown in Figure 5.3, where

\[ \tau_{ef-i} : \text{Average shear stress between FRP and epoxy in ith segment} \]

\[ \tau_{ec-i} : \text{Average shear stress between concrete and epoxy in ith segment} \]

\[ P_i, P_{i+1} : \text{Forces in FRP at distance } i \text{ and } i+1 \]

\[ t_f : \text{Thickness of FRP plate} \]

\[ t_e : \text{Thickness of epoxy} \]

If the contact widths between FRP and epoxy and between epoxy and concrete are the same, then \( \tau_{ef} \) equals \( \tau_{ec} \). Actually the two widths are slightly different because when the FRP delaminates from the concrete, a small area of concrete besides the bonding area is pulled out, which makes \( \tau_{ec-i} \) less than \( \tau_{ef-i} \). This difference is ignored here since the extra area is relatively small compared to the bonding area.

The load transferred to concrete in \( \Delta x_i \) distance is

\[ \Delta P_i = P_{i+1} - P_i = E_f \cdot \varepsilon_{f_{-i+1}} \cdot w_f \cdot t_f - E_f \cdot \varepsilon_{f_{-i}} \cdot w_f \cdot t_f = E_f \cdot w_f \cdot t_f \cdot \Delta \varepsilon_{f_{-i}} \]

The average shear stress in the concrete over \( \Delta x_i \) distance is
\[ \tau_{ef-i} = \frac{\Delta \varepsilon_{f-i} \cdot E_f \cdot t_f}{\Delta x_i} \]  

(5-1)

Where

\[ \Delta \varepsilon_{f-i} = \varepsilon_{f,i+1} - \varepsilon_{f,i} \]: Difference in strain between two adjacent strain gages

\[ \varepsilon_f \]: Strain in FRP

\[ E_f \]: Modulus of elasticity of FRP

\[ w_f \]: Width of FRP plate

The three specimens tested for this research are named DB1, DB2, and DB3. DB1 was a preliminary specimen which was tested to ensure the specimen design performed properly. It was found that DB1 performed as required. Then DB2 and DB3 were constructed with some modifications for better results. Two kinds of FRP plates were used for this research, Sika Carbodur and Sika CarboShear-L. The properties of these two FRP plates are described later in this chapter.

The following aspects were different among the three specimens: concrete strength, FRP plate, surface preparation, bonded length. Although the concrete mixture design was the same for all three specimens, the curing times were different, which resulted in different concrete strengths. The concrete strength for each specimen is listed later in this chapter. Two kinds of surface preparation methods were used, needle gun and sandblaster. DB1 was roughed using a needle gun while for DB2 and DB3 a sandblaster was used since this is often a more economical method for surface preparation in the field.

The lengths of the two concrete prism blocks were the same for DB1 (Figure 5.4). After DB1 was tested, it was found that the bonding length should be longer for better
results. As a result, the length of the test side for DB2 and DB3 was increased to 23 inch (Figure 5.5 and Figure 5.6). Due to the limitation of the MTS universal test frame, the anchor side was reduced to 9 inch. Therefore CFRP wrap was provided over the non-test block to prevent delamination. Table 5.1 provides a description of each specimen.

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>1st Bond Length (inch)</th>
<th>2nd Bond Length (inch)</th>
<th>3rd Bond Length (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DB1</td>
<td>12</td>
<td>DB2</td>
<td>23</td>
</tr>
<tr>
<td>DB2</td>
<td>23</td>
<td>DB3</td>
<td>23</td>
</tr>
<tr>
<td>DB3</td>
<td>23</td>
<td>DB4</td>
<td>23</td>
</tr>
</tbody>
</table>

Table 5.1. Description of Specimens
5.3 Material Properties

5.3.1 Concrete

Normal weight concrete was used for this investigation. Concrete was mixed in the Structural Testing Laboratory at the University of Hawaii at Manoa. The mixture design is shown in Table 5.2.

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight per 0.75 ft³</th>
<th>Weight per cuyd (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Aggregate (Basalt)</td>
<td>43.78</td>
<td>1576</td>
</tr>
<tr>
<td>Dune Sand</td>
<td>11.99</td>
<td>432</td>
</tr>
<tr>
<td>Basalt Sand</td>
<td>22.96</td>
<td>827</td>
</tr>
<tr>
<td>Cement</td>
<td>20.37</td>
<td>733</td>
</tr>
<tr>
<td>Water</td>
<td>8.12</td>
<td>292</td>
</tr>
</tbody>
</table>

The compression strength of the concrete was determined according to ASTM C39-99. The test results are shown in Table 5.3.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cylinder</th>
<th>P(k)</th>
<th>Area (in²)</th>
<th>Strength (psi)</th>
<th>Avg. (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>1</td>
<td>256</td>
<td>25.77</td>
<td>9934</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>249</td>
<td>27.00</td>
<td>9221</td>
<td>10123</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>287</td>
<td>25.60</td>
<td>11213</td>
<td></td>
</tr>
<tr>
<td>2nd</td>
<td>1</td>
<td>225.5</td>
<td>28.18</td>
<td>8003</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>240</td>
<td>28.18</td>
<td>8518</td>
<td>8261</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>239</td>
<td>28.35</td>
<td>8431</td>
<td></td>
</tr>
<tr>
<td>3rd</td>
<td>1</td>
<td>228</td>
<td>28.15</td>
<td>8100</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>235</td>
<td>28.17</td>
<td>8342</td>
<td>8291</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>239</td>
<td>28.35</td>
<td>8431</td>
<td></td>
</tr>
</tbody>
</table>

5.3.2 FRP System

Two kinds of FRP plate were used in this research, Sika Carbodur S512 and Sika CarboShear-L. The properties of these two kinds of FRP, as provided by the manufacturer, are listed in Table 5.4.
Table 5.4. Properties of FRP

<table>
<thead>
<tr>
<th>Property</th>
<th>Sika CarboShear-L</th>
<th>Sika Carbodur S512</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength (psi)</td>
<td>$3.80 \times 10^7$</td>
<td>$4.06 \times 10^7$</td>
</tr>
<tr>
<td>Modulus of elasticity (psi)</td>
<td>$22.48 \times 10^6$</td>
<td>$23.9 \times 10^6$</td>
</tr>
<tr>
<td>Elongation at failure</td>
<td>1.69%</td>
<td>1.69%</td>
</tr>
<tr>
<td>Width (inch)</td>
<td>1.575</td>
<td>1.97</td>
</tr>
<tr>
<td>Thickness (inch)</td>
<td>0.047</td>
<td>0.047</td>
</tr>
<tr>
<td>Ultimate Tensile Force (kip)</td>
<td>28</td>
<td>37.8</td>
</tr>
</tbody>
</table>

FRP is bonded onto the surface using Sikadur 30 epoxy resin as the adhesive. The properties of Sikadur 30 epoxy, as provided by the manufacturer, are listed in Table 5.5.

Table 5.5. Properties of Sikadur 30 Epoxy

<table>
<thead>
<tr>
<th>Property</th>
<th>Value from manufacturer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Strength (psi)</td>
<td>2175</td>
</tr>
<tr>
<td>Static E-Modulus (psi)</td>
<td>$1.856 \times 10^6$</td>
</tr>
<tr>
<td>Adhesive Strength (psi)</td>
<td>580 (Concrete Failure)</td>
</tr>
</tbody>
</table>

5.4 Specimen Preparation

The specimen preparation includes concrete surface preparation and FRP installation.

5.4.1 Surface Preparation

After the concrete blocks were cured, the surfaces where FRP would be applied were roughened to provide improved bond between FRP and concrete.

For DB1 a needle gun was used to roughen the surface. Since the surfaces of the two concrete blocks were not flat enough for the FRP alignment, the needle gun was also used to remove excess concrete. As a result, most cement paste on the surface was removed and coarse aggregate was exposed.
The surfaces of DB2 and DB3 were sandblasted since sandblasting is an economical procedure for field application. Sandblasting produced a textured concrete surface, but did not remove the outer cement paste layer or expose the coarse aggregate (Figure 5.7).

![Sandblasted Concrete Surface](image)

**Figure 5.7: Sandblasted Concrete Surface**

Before the FRP was applied, compressed air was used to blow away dust and particles on the roughened surface.

At the anchor side, the sharp corners were rounded using a concrete grinder. This was done to reduce the corner radius for the transverse CFRP wrap which was installed to prevent delamination on this side of the specimen. The rounded concrete block is shown in Figure 5.8.
5.4.2 FRP Installation

Before the FRP was applied, the sides of the two steel plates were covered with transparent tape to prevent bond between the FRP and steel plate.

The peel-ply on the surface of the Sika CarboShear-L was removed. The contact surfaces of the FRP were cleaned using MEK cleaner as suggested by the manufacturer.

When the surfaces of the FRP dried, a thin layer of Sikadur 30 epoxy was applied to the concrete surface and the FRP surface with a trowel. The FRP plate was placed onto the concrete surface. Epoxy was cured for 24 hours without disturbance.

The anchor side was wrapped with wet lay-up CFRP. Sika Wrap 103C uni-direction carbon fiber sheets saturated with Sikadur-300 two-part epoxy were used to wrap the anchor blocks.
5.5 **Instrumentation and Test Setup**

The test was performed on a MTS universal test frame in displacement control mode. The controller is a MTS TestStar II. All data were recorded by a National Instrument Data Acquisition system. The ends of the steel rods were gripped by the hydraulic wedge grips on the test machine. Axial load was applied to the specimen through the steel rods. The alignment of the steel rods in the two concrete blocks is critical to the test.

5.5.1 **Specimen DB1**

Since DB1 was a trial specimen, only one crack gage and nine strain gages were installed (Figure 5.9). Readings from three strain gages, SG2, SG6, and SG7 were not available due to a problem with the strain gages or reading channels.

![Figure 5.9: Instrumentation for DB1](image)

5.5.2 **Specimen DB2**

Two crack gages and 27 strain gages were installed on DB2 as shown in Figure 5.10.

![Figure 5.10: Instrumentation for DB2](image)
5.5.3 Specimen DB3

Two crack gages and 28 strain gages were installed on DB3 as shown in Figure 5.11.

![Instrumentation for DB3](image)

Figure 5.11: Instrumentation for DB3

5.6 Test Results

5.6.1 Trial Specimen DB1

5.6.1.1 Specimen Response

The specimen was loaded in displacement control. The load increased to 14 kips, after which it dropped to 13.2 kips and then began to increase again until failure (Figure 5.12). This drop was due to partial delamination of the FRP. Since the test was performed in displacement control, the load dropped proportional to the strain release during delamination. The maximum load applied was 14.77 kips. Assuming no eccentricity, and that the total load was carried equally by the FRP on each side of the specimen, the maximum load carried by one FRP plate was 7.4 kips.

The propagation of delamination was not obvious during the test, but was detected by the strain gages as described below.

The specimen failed due to delamination between the Carbodur plate and epoxy (Figure 5.13).
Figure 5.12: Total Load on Specimen DB1

Figure 5.13: Failure of DB1
5.6.1.2 Strain in FRP

Figure 5.14 shows the strain reading in each strain gage during the test. The location of the strain gages is shown in Figure 5.9. A total of nine strain gages were installed, but due to damaged strain gages or bad data acquisition channels, readings from sg2, sg6, and sg7 were lost. Strain readings from all other strain gages are presented here.

Before the 2252nd reading, the closer the strain gage was to the center (crack), the higher the strain reading. At the 2252nd reading point, there were sudden strain increases in sg3, sg4 and sg5. After this increase, strains in these three strain gages were about the same as to those in sg0 and sg1. This indicates that delamination happened between these gage locations. This is also the point at which the load dropped.

Figure 5.15 shows the relationship of load vs. strain. The sudden increase in strains in sg3, sg4, and sg5 is observed to occur at the same time as the drop in total load. The remaining bonded region of FRP was able to resist additional load before debonding completely at 14.77 kips.
Figure 5.14: Strain Readings during Test of DB1

Figure 5.15: Load vs. Strain
5.6.1.3 Strain Profile

The profiles of strain along the length of the FRP at selected loading stages are plotted in Figure 5.16. Sg0 is at the unbonded zone and sg1 is right at the intersection of bonded region and unbonded region. The strains in these two strain gages represent the strain at the crack, and should be the same, which was confirmed by the strain profile plot. The slopes of these strain curves are proportional to the shear stresses according to Equation (5-1). When the slope reached the maximum, the shear stress between the FRP, epoxy and concrete reached its maximum. The maximum slope is 1022 $\mu\varepsilon$/inch. The maximum shear stress is therefore:

$$\tau_{\varepsilon} = \frac{\Delta \varepsilon_{f,i} \cdot E_f \cdot t_f}{\Delta \chi_i} = \frac{1022}{1000000} \cdot \frac{\text{in}}{\text{ksi}} \cdot \frac{23900\text{ksi} \cdot 0.047\text{in}}{\text{in}} = 1148\text{psi}$$

After delamination, the strain is uniform for the first four inches from the crack (curve marked as 14.14k in Figure 5.16).
5.6.1.4 Crack Width

One crack gage was installed to measure the crack width. The load vs. crack width for DB1 is plotted in Figure 5.17. Immediately before the delamination, the crack width was 0.027 inch. After the delamination, the crack width increased to 0.033 inch.
5.6.2 Specimen DB2

5.6.2.1 Specimen Response

FRP delamination was observed during the test. The specimen failed by delamination between the epoxy and concrete. The failure surface is in the substrate of concrete (Figure 5.18 and Figure 5.19). The surface paste of concrete block remained bonded to the FRP, exposing the coarse aggregate in the concrete.

As for specimen DB1, the load was applied by displacement control mode. That is, the two concrete blocks were pulled apart at a uniform rate. Consequently, the load on the specimen did not increase monotonically, but dropped twice (Figure 5.20). The first drop happened at reading sequence 2904 and the second at 3079. Each drop in load was associated with partial delamination of the FRP. The maximum load carried by DB2 was
7.2 kips, with 3.6 kips for each FRP plate, assuming that the FRP plates shared the load equally.

Figure 5.18: FRP Surface after Failure

Figure 5.19: Concrete Surface after Failure
5.6.2.2 Strain in FRP

Strain along the length of the FRP (CarboShear-L plate) was recorded by closely spaced strain gages as shown in Figure 5.10. The strain readings in each strain gage during the test are shown in Figure 5.21 to Figure 5.25. In Figure 5.21 readings from all strain gages are included. For clarity the reading sequence is plotted from 2400 since load was only applied after this reading. Also for clarity, the strain results are plotted from Figure 5.22 to Figure 5.25 with selected strain gage readings in each plot.

Before reading increment 2900, the strain in sg27, sg26, sg25, sg24, sg23, sg22, sg21, sg20, sg19, and sg18 were all different. Strain gages closer to the gap (crack) typically recorded higher strain than neighboring strain gages closer to the free end. The
strain difference means a difference in force in the FRP at the strain gage locations, and therefore a load transfer from the FRP to the concrete between the two gage locations.

After reading increment 2900, strain in strain gages from sg18 to sg25 increased rapidly to match the strain readings in sg26 and sg27. The strain readings in these 9 strain gages are about the same after the increase, which means there is no force transfer from FRP to concrete between the neighboring strain gages, and delamination had occurred from sg18 to sg27. This is the first delamination.

Before the first delamination, strain in sg17, sg16, and sg15 were very small. After delamination the strain in these three strain gages began to increase quickly. There were still big strain differences in neighboring strain gages from sg18 to sg11, which means the FRP plate was still bonded to the concrete block and forces were transferred by the bond between FRP, epoxy, and concrete. Strain in sg17 gradually increased to about the same as that in sg18. At reading increment about 3074, readings in sg12, sg13, and sg14 increased rapidly to the same strain as sg17. The second delamination had occurred.

After the second delamination, strain readings in sg11, sg10, sg9, and sg8 began to increase quickly. Load was sustained by the remaining bonded section. Load transfer could be observed from the reading sequence vs. strain curve by the different strain readings in strain gages from sg11 to sg6. At reading increment about 3210, the strain suddenly increased in sg10 to sg1. At that time, the specimen failed.

The relationship of load on specimen and strain reading in each strain gage is shown from Figure 5.26 to Figure 5.30. It can be observed that when there were sudden strain increases, the load dropped and then began to increase again.
Based on the three observed delaminations, the effective bonding length is about 6 inch. When the third delamination occurred, the remaining bonded part was too short to carry the load and the FRP separated completely from the concrete block.

The maximum strain in the FRP was about 2200 $\mu\varepsilon$.

Figure 5.21: Strain in FRP during Test of DB2
Figure 5.22: Strain in FRP (sg1~sg7)

Figure 5.23: Strain in FRP (sg7~sg14)
Figure 5.24: Strain in FRP (sg14~sg20)

Figure 5.25: Strain in FRP (sg20~sg27)
Figure 5.26: Load vs. Strain (all)

Figure 5.27: Load vs. Strain (sg1~sg7)
Figure 5.28: Load vs. Strain (sg7~sg14)

Figure 5.29: Load vs. Strain (sg14~sg20)
5.6.2.3 Strain Profile along the Length of the FRP

The distribution of strain along the length of the FRP at selected test stages is plotted in Figure 5.31. The first number in the legend is the reading increment, while the load carried by the specimen at that moment is given in parentheses.

Initially, only a small portion of the FRP close to the crack experienced strain. The curve has a quadratic shape at the early stages of loading (strain curve 2542(2k) and 2632(4.8k) in Figure 5.31). When the load increased, the strain distribution began to change to a linear shape (strain curve 2877(6.4k) in Figure 5.31). Once the slope became linear, it maintained approximately the same slope till the end of the test.

The dashed lines are the strain profiles recorded during delamination. Comparing the strain profile before and after these dashed lines indicates what portion of the FRP delaminated. For example, the strain in strain gages from 1 inch to 4 inch from the crack
increased quickly to the same strain as the FRP at the crack. The same phenomenon occurred for strain gages located from 8 inch to 10 inch and from 14 inch to 20 inch. This can be seen more clearly in Figure 5.32 where the strain profiles are plotted such that the time intervals between curves are approximately equal.

The maximum shear stress between FRP and concrete for each strain gage interval was calculated using equation (5-1). The average value is 754 psi with COV (coefficient of variation) of 0.23. The maximum shear stresses from 6 inch to 8 inch were excluded from the calculation of the average maximum shear stress between FRP and concrete because the strain reading from sg20 (at 7 inch) appears unreliable. The value from 21 inch to 22 inch was also excluded since the strain in sg1 (at 22 inch) was not fully developed.

![Figure 5.31: Strain Profile along the Length of FRP](image)

Figure 5.31: Strain Profile along the Length of FRP
5.6.2.4 Crack Width

The crack width was measured using two crack gages installed on opposite sides of the specimen as shown earlier in this chapter. The curves of load vs. crack width are presented in Figure 5.33. It can be seen that readings from cg1 and cg2 are about the same, implying that the specimen was loaded concentrically. The crack width was about 0.01 inch when the first FRP delamination occurred.
5.6.3 Specimen DB3

5.6.3.1 Specimen Response

The specimen was loaded in displacement control mode. FRP delamination propagation was observed during the test. When the first delamination occurred, load was still carried by the remaining bonded part. A second delamination occurred, followed by the third delamination and failure of the specimen. The FRP debonded from the concrete. The failure surface was in the substrate of concrete as for DB2 (Figure 5.18 and Figure 5.19). The cement on the surface of the concrete was removed by the FRP and epoxy. Coarse aggregate in the concrete was exposed.

Figure 5.34 shows the total load applied to the specimen during the test. The first load drop occurred at reading sequence about 1583. The reading sequence for the second
load drop was about 1730. The maximum load carried by DB3 was 9.83 kips. Assuming the FRP at two sides share the load equally, the maximum load on one FRP plate was 4.92 kips.

![Graph showing load on Specimen DB3](image)

**Figure 5.34: Load on Specimen DB3**

### 5.6.3.2 Strain in FRP

A total of 28 strain gages were installed to monitor the strain change in FRP (Carbodur plate) along its length as shown in Figure 5.11. The strain readings in each strain gage during the test are shown in Figure 5.35. The same information is shown in Figure 5.36 to Figure 5.39 with selected strain gages included in each figure. The plots of load vs. strain are shown from Figure 5.40 to Figure 5.44.

From Figure 5.35, it can be observed that before reading sequence about 1585, there are differences between the strain in sg28, sg27, sg26, sg25, sg24, sg23, sg22, and sg21. Strain gages closer to the gap (crack) usually feel higher strain than strain gages closer to
the free end. The strain difference indicates the different forces in the FRP at the gage locations and the load difference was carried by shear transfer to the concrete between the gage locations.

At about reading sequence 1585, strain in strain gages from sg16 to sg27 increased rapidly. After the increase the strain readings in these 13 strain gages are all about the same, which indicates delamination occurred from sg16 to sg28. This is the first delamination.

Before the first delamination, strain in sg15, sg14, sg13, sg12 and sg11 were very small. After delamination, the strain in these five strain gages began to increase quickly. There were big strain differences in neighboring strain gages from sg16 to sg11 between reading sequence 1585 and 1790. This indicates that from sg16, the FRP plate was still bonded to the concrete block and shear was transferred by the bonding between FRP and concrete. Strain in sg16 gradually increased to about the same as that in sg18. At reading sequence about 1790, the strain in sg8, sg9, sg10, sg11, and sg12 increased quickly to the same level as sg16. The second delamination had occurred.

After the second delamination, strain readings in sg6, sg5, and sg4 began to increase quickly. The load was carried by the remaining bonded part. Load transfer could be observed from the reading sequence vs. strain curve by the strain difference between neighboring strain gages from sg11 to sg6 at the same moment. At reading sequence about 2000, the sudden strain increase happened from sg5 to sg1 indicating delamination, leading to failure of the specimen.
The relationship between load on specimen and strain reading in each strain gage is shown from Figure 5.40 to Figure 5.44. It can be observed that when the sudden strain increases occurred at the first time, the load dropped.

Based on the three observed delaminations, the effective bonding length is about 7 inch. When the third delamination occurred, the remaining bonded section was too short to carry the load, and the remaining FRP delaminated.

When delamination occurred, the maximum strain in the FRP was about 1750 µε.
Figure 5.36: Strain in FRP during Test Time for DB3 (sg1~sg7)

Figure 5.37: Strain in FRP during Test Time for DB3 (sg8~sg14)
Figure 5.38: Strain in FRP during Test Time for DB3 (sg15~sg21)

Figure 5.39: Strain in FRP during Test Time for DB3 (sg22~sg28)
Figure 5.40: Load vs. Strain for DB3

Figure 5.41: Load vs. Strain for DB3 (sg1~sg7)
Figure 5.42: Load vs. Strain for DB3 (sg8~sg14)

Figure 5.43: Load vs. Strain for DB3 (sg15~sg21)
5.6.3.3 Strain Profile along the FRP

The distributions of strain along the length of the FRP at selected test stages are plotted in Figure 5.45. The first number in the legend is the reading sequence number and the number in parentheses is the load carried by the specimen at that moment.

The curves have a quadratic shape at the early stages of loading (strain curve 550(3.8k) and 600(4.1k) in Figure 5.46). When the load increased, the curve began to change to a linear shape (strain curve 1563(9.8k) in Figure 5.45). Once the slope became linear, it maintained bi-linear shape till failure.

The dashed lines are the strain profiles during the very short period of delamination. Comparing strain profiles before and after these dashed lines, it can be observed that the slope of the curve became flat within a very short period. Strain in strain gages from 2.0
inch to 7 inch from center increased quickly to the maximum strain in the FRP. The same phenomenon occurred for strain gages located from 11 inch to 16 inch and from 18 inch to 22 inch. This can be seen more clearly in Figure 5.46 which is plotted such that the time interval is the same between all the curves except the last one.

The maximum shear stress between FRP and concrete for each strain gage interval was calculated using equation (5.1). The average value is 710 psi with COV of 0.26. The value from 20 inch to 22 inch was excluded from the calculation of the average maximum shear stress since strains in sg1 and sg2 were not fully developed.

Figure 5.45: Strain Profile along the Length of FRP for DB3
5.6.3.4 Crack Width

Two crack gages (cg1 and cg2) were installed on opposite sides of the specimen to record the crack width opening. The results from cg1 and cg2 are plotted in Figure 5.47. It can be seen that reading from cg1 is significantly larger than that from cg2. At the peak load, crack width measured by cg1 is about twice that measured by cg2. This difference is due to mis-alignment of the specimen. Using average value from cg1 and cg2, the crack width was 0.014 inch when the first delamination occurred.
5.7 Discussion and Conclusion

Three FRP debonding double shear type specimens were tested. These three specimens varied in surface preparation, FRP type, concrete strength, and bonding length. Based on the test results presented above, the following conclusions can be made:

1. Surface preparation is critical to bonding strength.

The same type of FRP was used in DB1 and DB3. A needle gun was used to roughen the concrete surface for DB1 while sandblasting for DB3. The bonding surface of DB1 was much rougher than that of DB3. The total load was 14.77 kips for DB1 and 9.83 kips for DB3. DB1 failed by delamination between the FRP and epoxy. The failure for DB3 was in the concrete surface. It can be concluded that the rougher surface improves bond strength and changes the failure type.
The shear strength between FRP and concrete was also affected by the surface preparation. The shear strength is higher when the surface is rougher. The average maximum shear strength for DB1 was 1148 psi, but only 710 psi for DB3.

The higher concrete strength for DB1 compared with DB2 and DB3 would also contribute to this increased shear capacity.

2. When delamination occurred, the slope of the strain profile becomes flat and strain in the strain gages increased suddenly to the same value as the strain gages on the unbonded zone.
CHAPTER 6. SUMMARY AND CONCLUSIONS

6.1 Summary

This research study consists of two parts; evaluation of the performance of Sika CarboShear-L stirrups for shear retrofit of reinforced concrete T-beams, and the study of the bonding behavior between FRP and concrete. They are both parts of the project entitled “Instrumentation and Monitoring the Performance of the FRP Shear Strengthening of the Salt Lake Boulevard Bridge.”

Two nominally identical T-beams, T-1 and T-2, were constructed. T-1 was loaded cyclically at nine different loading levels, with five cycles at each load level before it was loaded monotonically to failure. T-2 was loaded until shear cracks formed in the beam web. The cracking load was cycled five times. Cyclic loading of T-2 then continued for four more load levels as applied to T-1. Significant shear cracks appeared in the web. The cracked T-2 was then retrofitted using Sika CarboShear-L stirrups with FRCC filler blocks installed for load transfer between the FRP external stirrups and beam web. The last two load levels (10 load cycles) performed on T-2I were repeated for the retrofitted T-2R. T-2R with FRP shear retrofit was then subjected to the same cyclic loading as T-1, before monotonic loading to failure. A total of 42 strain gages were installed on the surface of the CarboShear-L stirrups to monitor the strain in the FRP and detect initiation of delamination.

Three pure debonding tests were carried out to study the bonding behavior between FRP and concrete. Sika Carbodur FRP plate and CarboShear-L FRP plate were used for these tests. Different concrete surface preparations were used. Specimen DB1, a trial
specimen, was tested to ensure the specimens would perform properly. Only limited strain gages were installed on DB1. It was found that DB1 performed as expected. DB2 and DB3 were based on DB1 with longer bonding length and many more strain gages to monitor the strain in the FRP.

6.2 Conclusions

6.2.1 Concrete T-beam Shear Retrofit Test

Based on the results from tests carried out on T-beams T-1 and T-2, the following conclusions can be drawn:

1. The shear capacity of T-1 was 97 kips while the shear capacity of T-2 was 137 kips with CarboShear-L stirrup retrofit, a 41% increase. Sika CarboShear-L stirrups increased the shear capacity of the T-beam section by 40 kips, compared with ACI 440 predictions of 16 kips if the wrapping scheme is considered as 3-sided U-wrap or 25 kips if it is considered as completely wrapped.

2. Sika CarboShear-L stirrups increased the stiffness of the concrete T-beam. The deflection at midspan at the peak loading for T-1 was 0.3 inch. At the same load level, T-2R had a deflection of 0.19 inch. T-2R with Sika CarboShear-L stirrup retrofit had a 0.29 inches deflection at peak load. For the same beam before and after retrofit, it was observed that T-2R had less deflection than T-2I at the same load level.

3. Sika CarboShear-L stirrups help control crack width and reduce the strain in the internal steel stirrups.
4. The anchorage for Sika CarboShear-L stirrups used in this test performed well. No anchorage slip was detected at the top anchorage where the FRP stirrups were bonded into a slot cut in the top slab. Strain gages on the L-anchor under the beam soffit showed very little strain until failure of the beam.

5. FRCC filler blocks helped to distribute the shear cracks and to reduce the crack widths.

6. Delamination between external FRP stirrups and concrete was detected through strain readings from the strain gages before visible signs of delamination. If strain in one strain gage increased suddenly to match the higher strain in a neighboring gage, and there was no new crack in the web, then delamination had occurred between these two gage locations. During load cycling, if the strain in a strain gage was observed to increase gradually until it match the higher reading in a neighboring strain gage, and there was no new crack near by, it indicated that the FRP was delaminating from the concrete.

6.2.2 Bonding Tests

Surface preparation is important for bonding strength and failure type. The same FRP plates were used for specimens DB1 and DB3. The bonding length for DB1 was shorter than that for DB3. DB1 was roughened using a needle gun. Almost all the cement paste on the concrete surface was removed and the coarse aggregate was exposed. The bonding surfaces of DB3 were sandblasted. The maximum load carried by DB1 was 14.77 kips while only 9.83 kips was carried by DB3. DB1 failed due to debonding
between the FRP plate and the epoxy. For DB3 the failure surface was in the surface of the concrete.

Initiation of delamination was observed by the strain change in strain gages applied to the FRP. If strain in the gages increased rapidly to match the strain in the neighboring strain gage on an unbonded section of FRP, it could be concluded that delamination had occurred between these gage locations.

At the early stages of loading, the shape of the strain profile curve is quadratic. With increasing load, the curve changes to a linear shape. Soon after the curve reached a linear shape, delamination occurred and the slope of the curve becomes bi-linear.

Immediately before delamination, the crack width was between 0.01 and 0.014 inches when the delamination occurred in the concrete, and 0.027 inch when the FRP debonded from the epoxy.

The maximum strain in the FRP was $2200\mu e$ for Sika CarboShear-L plate on specimen DB2. The maximum strain in Sika Carbodur plate was $3300\mu e$ for DB1 and $1750\mu e$ for DB3.

### 6.3 Recommendation for Future Research

The beams tested for this research are small scale beams. The performance might be somewhat different from the full scale bridge girders because of the size effect. Full scale shear retrofit tests of prestressed concrete bridge girders are recommended.

Research on criteria for delamination under cyclic load is recommended. It has been found from the concrete T-beam retrofit test that under cyclic load, FRP delamination occurs in a more gradual manner than under monotonic loading. Detection of
delamination based on strain readings can be different from the criteria for monotonic loading.
APPENDIX A. T-BEAM DESIGN CALCULATION

Figure A.1: T-beams T-1 and T-2 Details
Flextural and Shear Capacity of Test Beam
(According to ACI318-02)

Given Information

\[ L_{\text{span}} := 85.5 \text{ inch} \]
\[ t_f := 4.5 \text{ inch} \quad b_{wt} := 4 \text{ inch} \quad b_f := 22.5 \text{ inch} \quad h := 22.5 \text{ inch} \]
\[ s_f := 10 \text{ inch} \quad b_{wr} := 10 \text{ inch} \quad s_t := 10 \text{ inch} \quad s_r := 3 \text{ inch} \]
\[ f_c := 9110 \text{ psi} \quad f_y := 60000 \text{ psi} \quad f_{yy} := 60000 \text{ psi} \quad f_{yc} := 60000 \text{ psi} \]
\[ T_{\text{CFRP}} := 28000 \text{ kip} \quad E_s := 29000000 \text{ psi} \quad a_v := 0.11 \text{ in}^2 \quad E_f := 22480000 \text{ psi} \]
\[ n := 1 \quad \varepsilon_{fu} := \frac{380710.66}{E_f} \quad \varepsilon_{fu} = 0.017 \]
\[ t_{\text{fiber}} := \frac{1.2}{25.4} \quad t_{\text{fiber}} = 0.047 \text{ in} \quad w_f := \frac{40}{25.4} \quad w_f = 1.575 \text{ in} \]
\[ \psi_{f_u} = 0.95 \quad \psi_{f_{\text{fully}}} = 0.85 \]

Effective Flange Width

\[ b_{\text{eff}} := \min \left( \frac{L_{\text{span}}}{4}, b_{wt} + 16t_f, b_f \right) \]
\[ b_{\text{eff}} = 21.375 \text{ in} \]

\[ d, d' \text{ Calculation} \]
\[ a_{s1} := 0.60 \text{ in}^2 \quad a_{s2} := 1.27 \text{ in}^2 \quad a_{s3} := 1.00 \text{ in}^2 \]
\[ A_{s1} := 2a_{s1} \quad A_{s2} := 2a_{s2} \quad A_{s3} := 4a_{s1} \quad A_{s4} := 1a_{s3} \]
\[ A_{s1} = 1.2 \text{ in}^2 \quad A_{s2} = 2.54 \text{ in}^2 \quad A_{s3} = 2.4 \text{ in}^2 \quad A_{s4} = 1 \text{ in}^2 \]
\[ y_1 := \frac{3}{4} + \frac{7}{16} \quad y_2 := \frac{3}{4} + \frac{10}{16} \quad y_3 := y_1 + 2 \quad y_4 := y_3 + 2 \]
\[ y_1 = 1.188 \text{ in} \quad y_2 = 1.375 \text{ in} \quad y_3 = 3.188 \text{ in} \quad y_4 = 5.188 \text{ in} \]
\[ A_s := A_{s1} + A_{s2} + A_{s3} + A_{s4} \]
\[ y := \frac{A_{s1} \cdot y_1 + A_{s2} \cdot y_2 + A_{s3} \cdot y_3 + A_{s4} \cdot y_4}{A_s} \]
\[ y = 2.487 \text{ in} \]
\[ d := h - y \quad d = 20.013 \text{ in} \]
\[ d_\text{\textunderscore} := \left( \frac{3}{4} \right) + \frac{7}{16} \quad d_\text{\textunderscore} = 1.188 \text{ in} \]
\[ d_t := h - y_1 \quad d_t = 21.313 \text{ in} \]
\[ d_{\text{min}} := h - y_4 \quad d_{\text{min}} = 17.313 \text{ in} \]
\[ d_f := d - 0.5 \quad d_f = 19.513 \text{ in} \]
Nominal Flexural Strength

Assume compressive steel and tension steel yield and the compression block is within flange

\[ M_{n_{\text{comp}}} := \frac{A_S \cdot f_{yc} \cdot (d - d_\text{c})}{12000} \]

\[ M_{n_{\text{comp}}} = 339 \text{ k}\cdot\text{ft} \]

\[ a := \frac{A_S - A_S}{0.85 \cdot f_y} \]

\[ a = 1.283 \text{ in} \quad \beta_1 \cdot c < t_f \quad \text{OK} \]

\[ \beta_1 := 0.65 \]

\[ c := \frac{a}{\beta_1} \]

\[ c = 1.974 \text{ in} \]

\[ \varepsilon_{\text{c}} := \frac{0.003}{c} \cdot (c - d_\text{c}) \]

\[ \varepsilon_{\text{c}} = 0.0012 \quad \beta_1 \cdot c < 0.002 \quad \text{Compressive steel doesn't yield.} \]

Solve equation \( A_s \cdot \sigma_s + 0.85 \cdot f_c' \cdot b_{\text{eff}}' \cdot a = A_s \cdot f_y \) for \( a \)

Where \( \sigma_s = E_s \cdot 0.003 \cdot (c - d_\text{c}) / c \)

\[ a = \beta_1 \cdot c \]

We have

\[ a := 1.606 \text{ in} \]

\[ c := \frac{a}{\beta_1} \]

\[ c = 2.471 \text{ in} \]

\[ \varepsilon_{\text{c}} := 0.003 \cdot \left( 1 - \frac{\beta_1 \cdot d_\text{c}}{a} \right) \]

\[ \varepsilon_{\text{c}} = 0.0016 \]

\[ \varepsilon_{\text{min}} := \left( \frac{0.003}{c} \right) \cdot d_{\text{min}} \]

\[ \varepsilon_{\text{min}} = 0.021 \quad \beta_1 \cdot c > 0.002 \quad \text{Tension steel yield} \]

\[ \phi_{\text{M}} := 0.9 \]

\[ M_{n_{\text{comp}}} := \frac{E_s \cdot \varepsilon_{\text{c}} \cdot A_S \cdot (d - d_\text{c})}{12000} \]

\[ M_{n_{\text{comp}}} = 255 \text{ k}\cdot\text{ft} \]

\[ M_{n_{\text{c}}} := \frac{0.85 \cdot f_c' \cdot b_{\text{eff}}' \cdot a \cdot \left( d - \frac{a}{2} \right)}{12000} \]

\[ M_{n_{\text{c}}} = 426 \text{ k}\cdot\text{ft} \]

\[ M_n := M_{n_{\text{comp}}} + M_{n_{\text{c}}} \]

\[ M_n = 681 \text{ k}\cdot\text{ft} \]

\[ \phi M_n := \phi_{\text{M}} \cdot M_n \]

\[ \phi M_n = 613 \text{ k}\cdot\text{ft} \]

\[ P_{\text{mn}} := \frac{4 \cdot M_n}{L_{\text{span}}} \]

\[ P_{\text{mn}} = 382 \text{ kips} \]
Nominal Shear Strength

Test Shear Span

\[ V_c := 2 \cdot \sqrt{f_c \cdot b_{wt} \cdot \frac{d}{1000}} \quad V_c = 15 \text{ kip} \]
\[ V_{\text{max}} := 10 \cdot \sqrt{f_c \cdot b_{wt} \cdot \frac{d}{1000}} \quad V_{\text{max}} = 76 \text{ kip} \]
\[ V_s := a_v \cdot 2 \cdot f_{yv} \cdot \frac{d}{s_t \cdot 1000} \quad V_s = 26 \text{ kip} \]

3-sided U-wraps:

\[ \frac{2500}{\left(n \cdot t_{\text{fiber}} \cdot E_f\right)^{0.58}} \quad \text{Le} = 0.799 \]

\[ k_1 := \left( \frac{f_c}{4000} \right)^3 \quad k_1 = 1.731 \]

\[ \kappa_v := \min \left( 0.75, \frac{k_1 \cdot k_2 \cdot \text{Le}}{468 \cdot e_{fu}} \right) \]

\[ \varepsilon_{fe_u} := \min \left( \kappa_v \cdot e_{fu} \cdot 0.004 \right) \]

\[ A_{fv} := 2 \cdot n \cdot t_{\text{fiber}} \cdot w_f \]
\[ f_{fe_u} := \varepsilon_{fe_u} \cdot E_f \]
\[ V_{f_u} := A_{fv} \cdot f_{fe_u} \cdot \frac{d_f}{s_f \cdot 1000} \]
\[ \psi_{f_u} V_{f_u} = 15.733 \]

Completely wrapped:

\[ \varepsilon_{fe_{\text{fully}}} = \min \left( 0.75 \cdot e_{fu} \cdot 0.004 \right) \]

\[ f_{fe_{\text{fully}}} := \varepsilon_{fe_{\text{fully}}} \cdot E_f \]

\[ V_{f_{\text{fully}}} := A_{fv} \cdot f_{fe_{\text{fully}}} \cdot \frac{d_f}{s_f \cdot 1000} \]
\[ \psi_{f_{\text{fully}}} V_{f_{\text{fully}}} = 24.804 \]
Nominal shear capacity:

3-sided u-wrap:
\[ V_{n\_u} := V_c + V_s + \psi f_u V_f_u \]
\[ V_{n\_u} = 57 \text{ kip} \quad \text{if} \quad V_{n\_u} < V_{\text{max}} \]

Completely wrapped:
\[ V_{n\_fully} := V_c + V_s + \psi f_{fully} V_f_{fully} \]
\[ V_{n\_fully} = 67 \text{ kip} \quad \text{if} \quad V_{n\_fully} < V_{\text{max}} \]

Check Stirrup Spacing
\[ s_{t\_max} := \min\left( \frac{d}{2}, 24 \right) \]
\[ s_{t\_max} = 10.007 \text{ in} \quad \text{if} \quad s_t > s_{t\_max} \quad \text{OK} \]

Check Minimum Shear Reinforcement
\[ A_{vt\_min} := \max\left( 0.75 \sqrt{\frac{f_c \cdot b_{wr}}{f_{yv}}}, s_t \frac{s_t}{f_{yv}} \right) \]
\[ A_{vt\_min} = 0.048 \text{ in}^2 \]
\[ A_{vt} := 2 \cdot a_v \quad A_{vt} = 0.22 \text{ in}^2 \quad \text{if} \quad A_{vt} > A_{vt\_min} \quad \text{OK} \]

Nominal beam load:
\[ V_n := V_{n\_fully} \]
\[ V_{n\_fully} = 67 \text{ kip} \]
\[ \phi_V := 0.75 \]
\[ \phi V := \phi_V V_{n\_fully} \]
\[ \phi V = 50 \text{ kip} \]
\[ P_{vtnt} := V_n^2 \]
\[ P_{vtnt} = 133 \text{ kips} \quad \text{if} \quad P_{vtnt} < P_{\text{mn}} \quad \text{Fail in shear} \]

Reaction Shear Span
\[ V_{cr} := 2 \cdot \sqrt{\frac{f_c \cdot b_{wr}}{1000}} \cdot \frac{d}{1000} \]
\[ V_{cr} = 38 \text{ kip} \]
\[ V_{max\_r} := 10 \cdot \sqrt{\frac{f_c \cdot b_{wr}}{1000}} \cdot \frac{d}{1000} \]
\[ V_{max\_r} = 191 \text{ kip} \]
\[ V_{sr} := a_v \cdot 4 \cdot \frac{d}{s_r \cdot 1000} \]
\[ V_{sr} = 176 \text{ kip} \quad \text{if} \quad V_{sr} > 4 \sqrt{\frac{f_c \cdot b_{wr}}{1000}} \cdot \frac{d}{1000} = 76 \text{ kip} \]

Check Stirrup Spacing
\[ s_{r\_max} := \min\left( \frac{d}{4}, 12 \right) \]
\[ s_{r\_max} = 5.003 \text{ in} \quad \text{if} \quad s_r > s_{r\_max} \quad \text{OK} \]

Check Minimum Shear Reinforcement
\[ A_{vr\_min} := \max\left( 0.75 \sqrt{\frac{f_c \cdot b_{wr}}{f_{yv}}}, s_r \frac{s_r}{f_{yv}} \right) \]
\[ A_{vr\_min} = 0.036 \text{ in}^2 \]
\[ A_{vr} := 4 \cdot a_v \quad A_{vr} = 0.44 \text{ in}^2 \quad \text{if} \quad A_{vr} > A_{vt\_min} \quad \text{OK} \]
Shear Strength

\[ V_{nr} := V_{cr} + V_{sr} \]
\[ V_{nr} := V_{\text{max}_r} \]
\[ \phi_v := 0.75 \]
\[ \phi V_r := \phi_v \cdot V_{nr} \]
\[ P_{vrn} := V_{nr}^2 \]

\[ V_{nr} = 214 \text{ kip} \quad \square > V_{\text{max}_r} \]
\[ V_{nr} = 191 \text{ kip} \]
\[ \phi V_r = 143 \text{ kip} \]
\[ P_{vrn} = 382 \text{ kips} \quad \square > P_{vtm} \quad \text{Fail in test shear span} \]
APPENDIX B. SHEAR VS. STRAIN IN FRP STIRRUPS

Figure B.1: Shear vs. Strain for SGF1
Figure B.2: Shear vs. Strain for SGF2

Figure B.3: Shear vs. Strain for SGF3
Figure B.4: Shear vs. Strain for SGF4

Figure B.5: Shear vs. Strain for SGF5
Figure B.6: Shear vs. Strain for SGF6

Figure B.7: Shear vs. Strain for SGF7
Figure B.8: Shear vs. Strain for SGF8

Figure B.9: Shear vs. Strain for SGF9
Figure B.10: Shear vs. Strain for SGF10

Figure B.11: Shear vs. Strain for SGF11
Figure B.12: Shear vs. Strain for SGF12

Figure B.13: Shear vs. Strain for SGF13
Figure B.14: Shear vs. Strain for SGF14

Figure B.15: Shear vs. Strain for SGF15
Figure B.16: Shear vs. Strain for SGF16

Figure B.17: Shear vs. Strain for SGF17
Figure B.18: Shear vs. Strain for SGF18

Figure B.19: Shear vs. Strain for SGF19
Figure B.20: Shear vs. Strain for SGF20

Figure B.21: Shear vs. Strain for SGF21
Figure B.22: Shear vs. Strain for SGF22

Figure B.23: Shear vs. Strain for SGF23
Figure B.24: Shear vs. Strain for SGF24

Figure B.25: Shear vs. Strain for SGF25
Figure B.26: Shear vs. Strain for SGF26

Figure B.27: Shear vs. Strain for SGF27
Figure B.28: Shear vs. Strain for SGF28

Figure B.29: Shear vs. Strain for SGF29
Figure B.30: Shear vs. Strain for SGF30

Figure B.31: Shear vs. Strain for SGF31
Figure B.32: Shear vs. Strain for SGF32

Figure B.33: Shear vs. Strain for SGF33
Figure B.34: Shear vs. Strain for SGF34

Figure B.35: Shear vs. Strain for SGF35
Figure B.36: Shear vs. Strain for SGF36

Figure B.37: Shear vs. Strain for SGF37
Figure B.38: Shear vs. Strain for SGF38

Figure B.39: Shear vs. Strain for SGF39
Figure B.40: Shear vs. Strain for SGF40

Figure B.41: Shear vs. Strain for SGF41
Figure B.42: Shear vs. Strain for SGF42
REFERENCES

ACI Committee 318 (2002), “Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary, (ACI 318R-02),” American Concrete Institute, Farmington Hills, MI

ACI (2002), “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures (ACI 440.2R-02),” Reported by ACI Committee 440


