Stress-Softening Coefficients for Reinforced Concrete Retrofitted with Fiber Reinforced Polymer Sheets

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Abstract

The peak stress-softening coefficients were calculated for two reinforced concrete panels retrofitted with fiber reinforced polymer sheets (F1P-1 & F1P-3) and compared to theoretical values for reinforced concrete. F1P-1 had a stress softening coefficient of 0.46 at 1.95% longitudinal strain, 22% greater than the predicted softening coefficient for equivalent reinforced concrete. F1P-3 had a stress softening coefficient of 0.64 at 0.05% longitudinal strain, 6.5% greater than the predicted softening coefficient. Factors including sheet configuration, sheet coverage area, and sheet thickness possibly contributed to this stress-softening coefficient increase. More tests needs to be conducted to quantify the effects of these factors and to determine a modified stress-softening equation for the retrofitted reinforced concrete.
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Notation

ζσ = peak stress-softening coefficient
σp = peak panel compressive stress
fk = cylinder compressive strength
ζε = peak strain-softening coefficient
εp = compressive strain at the peak panel compressive stress
ε0 = compressive strain at the cylinder compressive strength
εl = average longitudinal strain in the panel
Introduction

Fiber reinforced polymers (FRP) are composites made of a large number of small, continuous, non-metallic fibers embedded in a resin matrix. For the last two decades, FRP has been used to repair and strengthen a wide range of structures from bridges to pipes to buildings. FRP has been used for applications such as blast mitigation, seismic retrofitting, pipe rehabilitation, corrosion repair, structural upgrades, as well as other applications. As the technology has developed, the behavior of reinforced concrete (RC) retrofitted with FRP can already be predicted in flexure by proposed models. However, the proposed shear models for such retrofitted concrete are inconsistent (Tumialan 2007). This inconsistency stems from a lack of clearly defined constitutive laws regarding FRP retrofitted RC.

FRP can be applied to many of the reinforced concrete structures such as shear walls, box bridges, and off-shore structures. By fully understanding and quantifying the behavior of a shear element of these types of structures, constitutive laws can be developed, and the behavior of the whole structure can be determined by utilizing finite element software. The most accurate way to understand these elements is to test full-scale elements using a Universal Element Tester or UET.

One vital aspect of predicting the shear behavior of reinforced concrete under sequential loading is the stress-softening coefficient. Concrete loaded in tension will crack. These cracks weaken the concrete, reducing the maximum compressive stress the panel can support. The stress-softening coefficient is an accurate way of measuring this effect. Researchers at the University of Houston have previously quantified the stress-softening coefficients for reinforced concrete, steel fiber reinforced concrete (SFRC), and prestressed reinforced concrete (PC). This paper extends the aforementioned research to include the stress-softening coefficient of RC retrofitting with FRP subjected to sequential loading by testing full-scale shear elements retrofitted with FRP in the University of Houston’s unique UET. Results for the softening coefficients are compared to a study done by Belarbi and Hsu, which developed constitutive laws for RC and proposed an equation for determining stress-softening coefficients by testing RC panels.

Objective

Calculate experimental softening coefficients for RC panel elements retrofitted with FRP sheets and compare to theoretical values for RC panels.

Theory

Definition of Softening Coefficients

The peak stress-softening coefficient, \( \zeta_\sigma \), is defined as the ratio of peak compressive stress in the panel, \( \sigma_p \), to the corresponding cylinder compressive strength, \( f_c \):

\[
\zeta_\sigma = \frac{\sigma_p}{f_c} \quad (1)
\]
There also exists a peak strain-softening coefficient, $\zeta_\varepsilon$, is defined as the ratio of the compressive strain at the peak panel compressive stress, $\varepsilon_p$, to the corresponding compressive strain at the cylinder compressive strength, $\varepsilon_0$:

$$\zeta_\varepsilon = \frac{\varepsilon_p}{\varepsilon_0} \quad (2)$$

However, previous studies (Belarbi 1994, Mansour 2004, Wang 2006) have all found the strain-softening coefficient to be approximately one. As such, this paper will focus on the stress-softening coefficient.

**Predicting Softening Coefficients**

In the last twenty years, researchers at the University of Houston have proposed formulas predicting the softening coefficients of reinforced concrete (Belarbi 1994), steel fiber reinforced concrete (Mansour 2004), and prestressed reinforced concrete (Wang 2006). For purposes of comparison in this paper, Belarbi’s work is most relevant.

*Reinforced Concrete (Belarbi 1994)*

$$\zeta_\sigma = \frac{0.9}{\sqrt{1+250\varepsilon_l}} \quad (3)$$

$$\zeta_\varepsilon = 1 \quad (4)$$

Where:

$\varepsilon_l =$ average longitudinal strain in the panel

**FRP**

The FRP used in this study was Tyfo® SCH-11UP Composite bonded with Tyfo® S Epoxy. The bonded FRP is designed to have an ultimate tensile strength in the primary fiber direction of 154,000 psi, which is slightly more than twice the ultimate tensile strength of mild steel. The FRP will fracture at 1.05% strain and exhibit little to no plastic deformation. For comparison mild steel yields at typically yields 0.2% strain and fails at around 20% strain. The FRP has a tensile elastic modulus of 14,800 ksi, about half that of mild steel. Additionally, this particular FRP has negligible tensile strength perpendicular to the primary fiber direction and negligible strength in compression.
Experimental Program

Three FRP reinforced concrete panels (F1P-1, F1P-2*, F1P-3) were tested. Panels F1P-1 and F1P-2 used Rebar Configuration A while panel F1P-3 used Rebar Configuration B (Figure 1). Each panel was wrapped with 8-inch-wide strips of 0.011 inch thick FRP, each in a different configuration (Figure 2). All panels measured 55 x 55 x 7 inches and were reinforced by #4 rebar in both the longitudinal and transverse directions.

* Panel F1P-2 failed prematurely in tension and, as such, did not produce sufficient data to calculate softening coefficients.
The panels were all tested using sequential loading in the University of Houston’s UET. The panels were first loaded in tension longitudinally until the desired tensile strain for the particular panel was achieved. This value varied for all three panels. The panels were then compressed in the transverse direction until failure.

**Concrete Mix**

The concrete mix design used for the panels had a target compressive strength of 6 ksi and a target slump of 4 inches. Weight ratios for the concrete components were $1 : 1.37 : 2.29 : 0.42$ for type III cement, fine aggregate (s.s.d.), coarse aggregate (s.s.d.), and water, respectively.

**Form Preparation**

All test panels were cast in a custom-built metal frame comprised of five steel plates bolted together on steel legs. Shep Farm Fresh form release was painted on the bed and sides of the frame to allow for easy removal after casting. #4 rebar was welded to reused mild steel inserts on a separate steel jig in the given configuration. The inserts with attached rebar were then bolted to the casting frame. Twelve sets of coupling nuts tack welded to all-thread were bolted to the frame’s bed in a square arrangement for later attachment of linearly variable differential deformers (LVDTs). Five seven-inch long PVC tubes, with ends coved by duct tape, were attached to the rebar with steel wire ties along both tension sides of the panel. These tubes allowed for the later attachment of confining steel plates outside of the measurement zone with steel bolts. The fully prepared casting frame can be seen in Figure 3 below.

![Figure 3: Casting Frame Prepared for Casting](image-url)
Casting

Due to the size of mixer used, each panel consisted of 2 batches of concrete. After being thoroughly mixed and measuring approximately 4 inches on a standard slump test, the first batch was poured in the casting frame. The concrete was then tamped along the edges of the frame to ensure the concrete outside of the LVDT measuring area was particularly well packed. The whole batch was then vibrated to reduce voids in the concrete. While the first batch was being vibrated the second batch was mixed and slump test administered. The second batch was then poured on top of the first concrete layer. The edges were once again thoroughly tamped and the whole panel vibrated. Excess concrete was then removed by sliding a board along the edge of the concrete frame. The panels were then touched up with spades until as flat and smooth as possible. The panel was then covered by a large plastic sheet to retain humidity and the surface of the panel was kept moist until the panel was removed from the mold. In addition to the panel, three standard 6”x12” cylinders of each batch were cast and cured in the same manner as the panel.

Panel Test Preparation

FRP Application

After the panel was removed from the mold the top side (rough side) of the panel was ground to remove any cast imperfections. The outline of the given FRP configuration was then drawn with care on both sides of the panel. All open holes on the panel were then covered and the designated sections of the panel FRP were coated with a layer of epoxy. The FRP was then carefully laid over this epoxy layer and an additional layer of epoxy was then painted on the FRP until the FRP appeared fully saturated. FRP was applied to panels F1P-1 and F1P-2 during a two day process, with one side applied a day. In these two cases, the FRP cured on a horizontally placed panel. This application process can be seen below in Figure 4.

Figure 4: FRP Application for One Side of F1P-1
To accomplish the full wrapping configuration required FRP was applied to panel F1P-3 on one day with the panel hanging vertically from the overhead crane. For this wrapping configuration the epoxy was thickened using silica fume to reduce any negative effects of gravity. For panels F1P-2 and F1P-3, after the FRP had been given a day to cure, holes were cut in the FRP sheets to expose the PVC tubes.

**Confining Plates Application**

To help prevent failure outside of the LVDT measurement area, the sides of the panel in the tensile direction were confined by metal plates. These plates were applied by sectioning off the plated area on the rough side of the panel (south side), pouring a thin layer of grout in this area, and then placing metal plates on top of this grout. Plates were then likewise attached to the bottom (north side) of the panel and then connected with bolts running through the PVC. These bolts were then prestressed with an air gun and then with a torque wrench. Additional confining plates were attached to the top and bottom edges of the panel to reduce the effect of shear lag between yokes.

**Panel-UET Connection**

After the FRP and confining plates were added, the inserts’ attachment holes were tapped to be cleaned and then yokes were lifted into place and bolted with an air gun to the inserts. The 20 total yokes were each attached with four bolts and the yokes in the transverse direction (compression direction) had a layer of lead placed between the yoke and the panel to improve stress distributions.

The panel was then ready to be placed into the UET (Figure 5). One of two UET’s worldwide; the machine is capable of testing panels 55 inches square and up to 16 inches thick. The UET contains 37 in-plane hydraulic jacks each with a 100 ton capacity, 3 in-plane rigid links, and, for the purposes of this paper, three out-of-plane rigid links. Each hydraulic jack can be controlled manually or by using an automated, servo-controlled system. The UET can operate in either load control or strain control, the later being a unique feature of this UET. Load is monitored from load cells installed in all jacks and rigid links, while strain is measured from LVDTs installed on both faces of the panel as shown in Figure 6.

![Figure 5: UET Diagram](image-url)
The panel was then lifted by crane into the UET, transferred to trolleys on the machine, and then pulled into place between the jacks with a wench. Steel pins were inserted in between the rigid links and corresponding yokes to secure the panel. The trolley was then detached from the panel and steel pins used to connect the remaining yokes to the 37 hydraulic jacks. Shims were then placed between the jack / rigid link and the yokes in the longitudinal to ensure the jack was connected to the middle of the yoke. Both sides of the panel were then painted white to improve crack visibility and the LVDTs were attached. An example panel fully prepared for testing can be seen in Figure 6 below.
Panel Test Procedure

All three panels were tested under sequential loading. The panels were first loaded in both tension and compression in the elastic region to ensure the UET was functioning properly. The panels were then loaded in tension with an end goal of 0.5% strain† as measured by the horizontal LVDTs. The loading sequence can be seen in Table 1 below.

<table>
<thead>
<tr>
<th>Test Segment</th>
<th>Description</th>
<th>Duration</th>
<th>Tensile End Goal</th>
<th>Compressive End Goal</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Elastic Tensile</td>
<td>6 min.</td>
<td>7 kips</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>Release</td>
<td>6 min.</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>Elastic Compressive</td>
<td>6 min.</td>
<td>0</td>
<td>15 kips</td>
</tr>
<tr>
<td>4</td>
<td>Release</td>
<td>5 min.</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>Tensile (load control)</td>
<td>25 min.</td>
<td>14 kips</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>Tensile (strain control)</td>
<td>60 min.</td>
<td>0.5% strain</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>Compressive (load control)</td>
<td>60 min.</td>
<td>0.5% strain</td>
<td>100 kips</td>
</tr>
<tr>
<td>8</td>
<td>Compressive (strain control)</td>
<td>~40 min.</td>
<td>0.5% strain</td>
<td>Failure</td>
</tr>
</tbody>
</table>

Table 1: Test Loading Sequence

† 0.5% was reached for panel F1P-3. However, panels F1P-1 & F1P-2 exhibited severe cracking outside of the LVDT measurement area before 0.5% measured strain was reached and the compressive phase was implemented at a lesser measured strain.
Experimental Results

**Compressive Stress-Strain Curves**

*Figure 7:* Compressive Stress-Strain Curves for Panels F1P-1, F1P-3, and Their Corresponding Cylinders

<table>
<thead>
<tr>
<th>Panel</th>
<th>$\varepsilon_l$ (in./in.)</th>
<th>$f_c$ (ksi)</th>
<th>$\sigma_p$ (ksi)</th>
<th>$\zeta_{\sigma}$ (ksi/ksi)</th>
<th>$\zeta_{\sigma}$ Predicted RC (ksi/ksi)</th>
<th>Percent Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1P-1</td>
<td>0.0195‡</td>
<td>6.36</td>
<td>2.93</td>
<td>0.46</td>
<td>0.37</td>
<td>22</td>
</tr>
<tr>
<td>F1P-3</td>
<td>0.005</td>
<td>5.79</td>
<td>3.71</td>
<td>0.64</td>
<td>0.60</td>
<td>6.5</td>
</tr>
</tbody>
</table>

*Table 2:* Experimental Data

‡ This strain is the estimated value from an average of hand measurements over the longitudinal direction of the panel. This was substituted for the LVDT strain readings because the major cracking occurred outside of the LVDT measuring area. This will be further justified in the discussion.
Figure 8: Stress-Softening Coefficient v. Compressive Strain

Figure 9: Experimental Data Compared With RC Theory
Discussion

**Difficulties & Adjustments**

Panels F1P-1 and F1P-2 both had significant issues regarding cracking outside of the measurement area. For F1P-1 provisional results were salvaged, while F1P-2 produced not reliable data. F1P-3 had no such cracking issues and was tested according to plan with the minor exception of brief computer difficulties.

**F1P-1**

Panel F1P-1 was constructed with Rebar and Wrapping Configurations A. The wrapping configuration proved to be troublesome, because the strength of the FRP made the LVDT measuring area the strongest portion of the panel. As such, the vast majority of tensile cracks occurred outside of the LVDT measurement area. Major cracks ran underneath the longitudinal compressive plates on the west edge of the panel (Figure 10), showing that the FRP strengthened the concrete more than the confining plates. On this edge, the FRP extended underneath the confining plates between 0.5-1 in. However, on the east edge of the panel, the FRP stopped approximately 0.5 in. before the confining plates. This resulted in a massive crack (Figure 11) running between the FRP and the plates, ultimately causing the failure of the bottom east corner of the panel at 107 kips per jack compression. Also, along the north side of this corner the FRP peeled off from the panel (Figure 12) taking a mass of concrete with it.

As almost all of the tensile strain was contained outside of the LVDT measurement area, the measured peak strain of 0.15% was clearly inaccurate. Hand measurements of the full width of the panel revealed a more logical strain of 1.95%. While this form of measurement in not exceedingly accurate, it can be used provisionally. Should future data confirm this strain value, it may be considered sufficiently accurate. If not, it may be discarded.

In attempt to prevent this pealing and to confine cracks to the measuring area Wrapping Configuration B was designed and implemented for the next panel. The difference between Wrapping Configurations A and B was the extension of the FRP to the edge of the panel, completely under the compressive plates in Configuration B. The hope was that the FRP would strengthen the whole panel equally and that the confining plates would strengthen the area outside the LVDT.
measuring area, causing the vast majority of longitudinal strain to occur in the LVDT measurement area. The minor decision was also made to paint the whole panel white, instead of painting the concrete white and the FRP yellow.

F1P-2

F1P-2 was constructed with Rebar Configuration A and Wrapping Configuration B. When tested, both configurations proved to be troublesome. Similar to F1P-1 a portion of FRP peeled back off of the panel, removing a mass of concrete. However, in this case, Wrapping Configuration B shifted the location of this peeling to the edge of the panel. This proved catastrophic, as the panel’s edge also contained the steel inserts. These smooth inserts do not bond strongly to the concrete and, as a result, while the panels was still being loaded in tension, the FRP in the northwest bottom corner off the panel peeled back stripping a large concrete section right of the metal insert. As this was happening, the rebar failed and the jack pulled the insert out of the panel as seen in Figure 13. The panel was attempted to be salvaged by pushing the insert back into the panel and continuing to load with the middle three jacks. However, a similar peeling issue (Figure 14: FRP Peeling Away from F1P-2Figure 14) caused the bottom two of these three jacks to pull their north inserts from the panel. As such, no substantial data was from this panel.

To correct the peeling issue, FRP Wrapping Configuration C was designed and implemented. In this configuration the FRP sheet is wrapped around the panel such that it partially overlapped the FRP sheet on the opposite side of the panel. The overlap was designed such that the panel outside of the LVDT measuring area contains two layers of FRP; while inside the measuring area only has a single layer of FRP. Additionally, to improve the inserts bond with the concrete and to prevent the rebar from failing prematurely, two shorter lengths of rebar were welded to each insert. These two new configurations proved to be very successful with F1P-3.
**Stress-Softening Coefficient**

As reported, panels F1P-1 and F1P-3 had experimental softening coefficients 22% and 6.5% greater than the predicted softening coefficients for RC. As the FRP is known to strengthen the RC panels, it is logical that the experimental softening coefficients are larger than the predicted RC values. With only two data points it is difficult to quantify exactly how much the FRP strengthened the panel. However, as the data point corresponding to F1P-1 relies on a less accurate form of longitudinal panel strain than F1P-3, it may be considered somewhat less accurate. As such, initial results appear to indicate that the FRP increases the stress softening coefficient by 5-15%. More tests are required to test the accuracy of this assertion and to determine which FRP factors govern the increase in the stress softening coefficient. Such factors could include: FRP thickness, FRP configuration, FRP orientation, as well as other factors.

**Suggested Improvements**

Although the testing of panel F1P-3 went as expected, the south side of the panel experienced noticeable FRP debonding from the concrete. To strengthen the bond between the concrete and the FRP, all future panels should be sand blasted prior to FRP installation. This step is necessary to test the panel at larger longitudinal strains.

**Conclusion**

The peak stress-softening coefficients were calculated for two RC panels retrofitted with FRP and compared to theoretical values for RC. F1P-1 had a stress softening coefficient of 0.46 at an approximated 1.95% longitudinal strain, 22% greater than the predicted softening coefficient for RC. F1P-3 had a stress softening coefficient of 0.64 at 0.05% longitudinal strain, 6.5% greater than the predicted softening coefficient. These initial results suggest that FRP increases the stress-softening coefficient by 5-15%. Factors including FRP thickness, FRP configuration, FRP orientation, as well as other factors possibly contribute to this stress-softening coefficient increase. More research needs to be conducted to quantify the effects of these factors and to determine a modified stress-softening equation for the retrofitted RC.
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