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The 13th International Symposium  
on Fiber-Reinforced Polymer  
Reinforcement for Concrete Structures

Editors:

Raafat El-Hacha, Lijuan (Dawn) Cheng,  
Maria Lopez de Murphy, William J. Gold

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# The 13<sup>th</sup> International Symposium on Fiber-Reinforced Polymer Reinforcement for Concrete Structures

Sponsored by Committee 440

The Concrete Convention and Exposition  
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## Preface

Fiber-reinforced polymer (FRP) composite materials have been widely used in civil engineering new construction and repair of structures due to their superior properties. FRP provides options and benefits not available using traditional materials. The promise of FRP materials lies in their high-strength, lightweight, noncorrosive, nonconducting, and nonmagnetic properties. ACI Committee 440 has published reports, guides, and specifications on the use of FRP materials for many reinforcement applications based on available test data, technical reports, and field applications. The aim of these documents is to help practitioners implement FRP technology while providing testimony that design and construction with FRP materials systems is rapidly moving from emerging to mainstream technology.

The first International Symposium on Fiber-Reinforced Polymer Reinforcement of Reinforced Concrete Structures (FRPRCS) conference was sponsored by the American Concrete Institute (ACI) and held in conjunction with ACI Spring convention in Vancouver, British Columbia, Canada in 1993. Since 1993, FRPRCS has become a prestigious and reputable international conference, and the symposia have been held in Ghent, Belgium (1995); Sapporo, Japan (1997); with ACI in Baltimore, USA (1999); Cambridge, England (2001); Singapore (2003); with ACI in Kansas City, USA (2005); Patras, Greece (2007); Sydney, Australia (2009); with ACI in Tampa, USA (2011); Guimarães, Portugal (2013); and Nanjing, China (2015).

FRPRCS continued the success of the previous conferences and the 13th International Symposium on Fiber-Reinforced Polymer Reinforcement of Concrete Structures (FRPRCS-13) was organized by ACI Committee 440 and held on October 14 and 15, 2017 at the ACI Fall 2018 convention in Anaheim, CA, USA. FRPRCS-13 attracted interest from researchers, practitioners, and manufacturers involved in the use of fiber-reinforced polymers (FRPs) as reinforcement for concrete masonry structures. This includes the use of FRP reinforcement in new construction and FRP for strengthening and rehabilitation of existing structures. The papers/presentations not only emphasized the experimental, analytical, and numerical validations of using FRP composites but also aimed at providing insights needed for improving existing guidelines. New frontiers of FRP research were explored that provide information on emerging materials, systems, and applications for extreme events such as fire and earthquakes. The technical papers also featured discussions on sustainability, novel applications, new technologies, and long-term field data that will result in greater acceptance and use of FRP composites technology by practitioners. Attendees were able to:

- a) Understand many of the critical topics of research related to the use of FRP reinforcement in new construction and FRP systems for strengthening and rehabilitating existing structures.
- b) Develop knowledge on existing international codes and guidelines on these materials and systems.
- c) Exposure to new FRP material technologies as they relate to reinforcing and strengthening concrete and masonry structures.
- d) Learn practical aspects of FRP reinforcement materials and systems in field applications for new construction and rehabilitation of existing concrete and masonry structures.

This Special Publication consists of a total of 54 accepted papers out of 63 submissions from 18 countries. 65 presentations were presented over made two days in sixteen special technical sessions. These sessions provided a worldwide state-of-the-art forum for researchers, civil/structural engineers, contractors, consultants, practitioners, and regulatory authorities to exchange recent advances in both research and practice, and to share information, experience, and knowledge in the implementation of FRP technology. The technical papers presented at the sessions and published in this volume included the most recent analytical and experimental research work as well as selected field applications, design, and construction guidelines. The sessions were well attended and generated substantial technical discussion and exchange of new technology. The sessions held included:

**FRPRCS A1—Bond and Anchorage of FRP Bars, Grids, and Laminates to Concrete -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of the bond and anchorage behavior of FRP bars, plates, and tendons to concrete.

**FRPRCS B1—Strengthening of Concrete Structures Using FRP Systems -> 5 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using FRP composites for the strengthening of reinforced concrete structures.

**FRPRCS A2—Bond and Anchorage of FRP Bars, Grids, and Laminates to Concrete -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of the bond behavior of different FRP-concrete systems.

**FRPRCS B2—Testing of FRP Material Characteristics -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of FRP material characteristics.

**FRPRCS A3—Emerging FRP Systems -> 5 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using emergent FRP systems in concrete infrastructure.

**FRPRCS B3—Strengthening of Concrete Structures Using FRP Systems-> 5 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using FRP composites for the strengthening of reinforced concrete structures.

**FRPRCS A4—Global Codes and Standards -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using FRP-reinforced or -strengthened concrete structures and will be aimed at providing insights needed for improving existing design guidelines.

**FRPRCS B4—Advances in Uses of FRP in Concrete and Masonry -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using FRP in concrete and masonry structures.

**FRPRCS A5—Seismic Resistance of Concrete Structures Using FRP Materials -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using FRP composites in seismic structural applications.

**FRPRCS B5—Strengthening of Concrete Structures Using FRP Systems -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using FRP composites for the strengthening of reinforced concrete structures.

**FRPRCS A6—FRP Reinforcement of Concrete and Masonry Walls -> 3 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using FRP composites for the reinforcement of concrete and masonry walls.

**FRPRCS B6—Field Applications and Case Studies -> 4 Presentations**

This session emphasized field applications and case studies of using FRP composites for internal reinforcement of concrete.

**FRPRCS A7—Effects of Extreme Events on FRP-Reinforced/Strengthened Structures -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of extreme events on FRP-reinforced or -strengthened concrete.

**FRPRCS B7—Effect of Environment on Durability -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of the effects of the environment on the durability of FRP-reinforced or -strengthened concrete structures.

**FRPRCS A8—Design and Performance Under Long-Term Loading and Environmental Exposure -> 3 Presentations**

This session emphasized the experimental, analytical, and numerical validations of FRP-reinforced or strengthened concrete structures under long term loading and environmental exposure.

**FRPRCS B8—Advances in Uses of FRP in Concrete and Masonry -> 4 Presentations**

This session emphasized the experimental, analytical, and numerical validations of using FRP composites for concrete and masonry structures.

In accordance with the standard review procedures established by ACI, all papers were reviewed by at least two experts in the subject area and approved for publication. All submitted papers were given serious consideration before a decision regarding publication was made.

This volume represents the thirteen in the symposium series and could not have been put together without the help, dedication, cooperation, and assistance of many volunteers and ACI staff members. First, we would like to thank the authors for meeting our various deadlines for submission, providing an opportunity for FRPRCS-13 to showcase the most current work possible at the symposium. Second, the International Scientific Steering Committee, consisting of many distinguished international researchers, including chairs of past FRPRCS symposia, many distinguished reviewers and members of the ACI Committee 440 who volunteered their time and carefully evaluated and thoroughly reviewed the technical papers, and whose input and advice have been a contributing factor to the success of this volume.

**Editors of this volume and chairs of the FRPRCS-13 international Symposium**

Raafat El-Hacha  
Lijuan (Dawn) Cheng  
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**ADVANTAGES OF PREPREG FRP SYSTEMS FOR EXTENDING SERVICE LIFE OF CONCRETE MEMBERS IN WET ENVIRONMENTS**

Erblina Vokshi

**SYNOPSIS:** The use of wet lay-up fiber-reinforced polymers (FRPs) in the construction industry continues to grow. Their lightweight, durability, and good material properties makes them very desirable in the concrete repair industry. Research has shown that these products greatly improve the strength of repaired corroded members and reduce the rate of post-repair corrosion. There is a lack of research, however, in understanding how curing of these systems in the presence of moisture or underwater may affect their ability to strengthen and protect reinforced concrete. In this paper, data on material properties of a prepreg system, cured in laboratory conditions and under-water is presented and compared. The results show that there is a very small drop in tensile material properties when the system is cured underwater. Compression of concrete cylinders strengthened with one layer of the system and cured underwater is shown to be in agreement with ACI 440.2R-08's confinement equations. Durability testing of the bond between the system and reinforced concrete using dolly pull-off test shows a 90% bond strength retention after 1,000hr exposure to various aggressive environments.

**Keywords:** under-water repairs, underwater CFRP, pile strengthening, confinement, under-water cure, corrosion, bond durability,

ACI member Eri Vokshi is a structures rehabilitation engineer at NRI. Her roles and responsibilities include ensuring all current products are in compliance with current codes and guidelines, acting as a liaison between NRI's sales and R&D department, managing all third party testing for FRP materials, and performing design calculations in accordance with ACI 440. Eri holds a Master's degree in Material Science from Arizona State University and is a professional engineer in the state of Florida. She has been working at NRI for 6 years. She is also an active member of ACI 440 and a voting member of ACI 440F.

## INTRODUCTION

Application of wet lay-up Fiber reinforced polymer (FRP) systems in the repair industry is well accepted. These applications include structural strengthening of structures such as parking garages, buildings, bridges, etc. which have experience some form of structural deficiency, many due to corrosion. Emergence of new resin chemistries is allowing these technologies to find new types of applications, such as reinforcement of piles submerged underwater, where corrosion of steel rebar is the major cause of damage.

Research by Suh, et al.<sup>1</sup> investigated a prepreg (water activated resin) technology as well as an existing epoxy technology in understanding effectiveness of FRP wraps in corrosion reduction in submerged piles. This study included laboratory testing as well as two field tests. The authors also studied short term and long term bond strengths when these products were applied on two bridge piles. Finally, eccentric testing on submerged and dry columns was performed. The results show that both systems slow down the corrosion rate, irrespective of whether carbon or glass is used. The bond strength and failure mode of these systems in wet conditions was low and the author suggests that improvements in bond strength are necessary to make these wraps more desirable. Also, though eccentric column testing was performed on these systems, the studies do not clearly show how the underwater wraps compare to applications in dry conditions.

In another paper by Mullins, et.al.<sup>2</sup> the materials used, application experience, and bond data for the two field projects mentioned above<sup>1</sup> are summarized. The authors do not provide any data on material properties of these products when fully cured under water. Also, though the author mentions that interaction diagrams can be developed for column strengthening, no correlation is offered to show how experimental data of columns wrapped underwater compare to theoretical calculations.

Other work, which demonstrate how composites protect reinforced concrete from corrosion of steel, was done by Gadve, et al<sup>3</sup> and includes a pull-out testing of a 330mm rebar was placed inside a concrete cylinder with induced corrosion using NaCl and a current. The results show that rebar corrosion drastically slowed down when concrete cylinders were wrapped with both CFRP and GFRP. Another paper by Masoud and Soudki<sup>4</sup> focus on the results of evaluation of the corrosion activity in unrepaired and FRP repaired specimens, using non-destructive and destructive techniques. The data shows that corrosion potential decreased with the progress of corrosion, and the FRP repair caused a higher rate of decrease of the corrosion potential with time than that observed for the unrepaired specimens. Another research by Spainhour and Wootton<sup>5</sup> show that the type of resin used in the FRP can affect the resistance of the reinforced member from corrosion.

Repairs with FRPs were shown to greatly improve the strength and ductility of repaired corroded members and reduced the rate of post-repair corrosion in work completed by Bonacci, et al<sup>6</sup>. Moreover, subjecting the repaired column to extensive, postrepair corrosion resulted in no loss of strength or stiffness and only a slight reduction in the ductility of the repaired member<sup>7</sup>.

Not much research however has been done in understanding how curing process underwater of wet lay-up systems may affect their ability to strengthen and protect reinforced concrete from rebar corrosion. Since underwater pilings require continuous corrosion maintenance at the water to air interface caused by cyclic wet and dry exposure of these areas with tidal height change and large marine organic growth, FRP solutions are a desirable option to asset owners. FRP jackets have been successfully used in many projects to protect concrete and wood pilings in these types of environments. However, the issue with these jackets is that they are size specific and require multi step process.

Research data in this report is presented for a new prepreg, wet lay-up carbon FRP (CFRP) product and epoxy primer, which cure in the presence of moisture and are UV resistant. The CFRP system provides a great advantage to FRP jackets since they are easy to handle and non-size specific. The product is saturated in a moisture-free, controlled

## Advantages of Prepreg FRP Systems for Extending Service Life of Concrete Members in Wet Environments

environment using specialized saturating machines, which ensure continuous levels of fiber to resin ratios. The prepreg rolls are hermetically sealed and opened when ready to be applied/laid-up for field application. The epoxy primer has a high viscosity and tack and is applied on concrete surface by heavily saturating a polyester felt fabric, which acts as a carrier of the primer and makes it easy to be applied underwater.

The intent of this paper is to show the material properties of this system when fully cured under-water, confinement strengthening effectiveness when the product is applied and cured underwater, while meeting existing ACI 440's<sup>8</sup> design equations, and bond degradation when exposed to various harsh environments.

### EXPERIMENTAL PROCEDURE

#### Tensile coupon testing

Tensile testing of the uniaxial carbon prepreg material was completed in accordance ASTM D3039. Panels were made by pressing two layers of fabric unto each other and letting them cure underwater and in laboratory conditions for 7 days before cutting them into 10in X 0.75in (250mm X 20mm) strips for testing. Figure 1 shows the panel fully submerged underwater while curing. A total of 8 samples were tested.

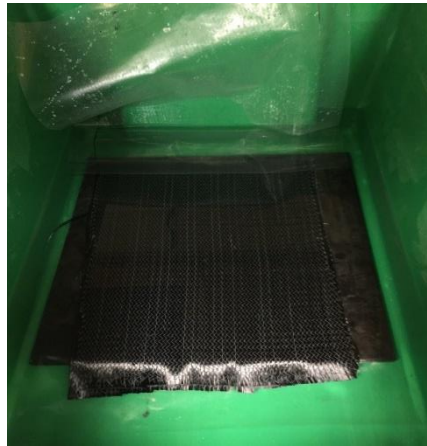


**Figure 1 - Panels curing under-water**

#### Concrete cylinder testing

This portion of the experimental study focused on confining 6in X 12in (150mm X 300mm) cylinders with the prepreg CFRP system loaded monotonically under axial compression to failure. In addition to concrete control samples, three additional configurations of the CFRP system were tested with the concrete cylinders, summarized in Table 1.

**Table 1 – Testing Matrix for Confinement Testing**

|                        | <b>Primer layer</b>                                                               | <b>FRP system</b>        | <b>Curing environment</b> |
|------------------------|-----------------------------------------------------------------------------------|--------------------------|---------------------------|
| <b>Configuration 1</b> | Commercially available epoxy primer – not suitable for underwater applications    | Pre-saturate CFRP system | Laboratory conditions     |
| <b>Configuration 2</b> | Polyester felt saturated with epoxy primer – suitable for underwater applications | Pre-saturate CFRP system | Laboratory conditions     |
| <b>Configuration 3</b> | Polyester felt saturated with epoxy primer – suitable for underwater applications | Pre-saturate CFRP system | Under-water               |

Concrete mix was poured into individual cylinders and was left to cure for 28 days prior to wrapping them with the CFRP system. The surface of concrete specimens was smoothed by grinding away any surface imperfections. Holes >1/4in (6mm) were filled with an epoxy filler. After, the samples to be wrapped with CFRP were prepared by applying the epoxy primer to the concrete surface, followed by wrapping the prepreg CFRP system with the fibers oriented in the hoop direction. On overlap length of 3in was applied at all the termination points of the wrap. While still wet, the

wrapped specimens were let to cure in laboratory conditions (configuration 1 and 2) or submerged fully under-water (configuration 3) and left to cure for seven days. Three identical cylinders were prepared for each configuration. Strain gages applied on the CFRP surface were 30mm grid, 350ohm.

The samples were tested on a CM-2500 Series Compression Testing Machines with a 250,000 lb (1,112 kN) capacity. LabView software was used to record both strain and load data.

#### 1,000hr durability testing of bond between the prepreg CFRP system and concrete substrate cured underwater

In order to assess the durability performance of the prepreg system when applied on the concrete surface and cured under-water, accelerated conditioning protocols were used to evaluate the bond strength performance of these systems. The exposure environments were in accordance with ACI 440.8-13 and ICC-ES AC 125<sup>9</sup>, summarized in Table 2. Dolly pull off test was performed in accordance with ASTM D7234-05 (Standard Test Method for Pull-off Adhesion Strength of Coatings on Concrete Using Portable Pull-Off Adhesion Testers).

**Table 2 – Accelerated exposure environment for durability study**

| Environmental exposure conditions | ASTM test method | Conditioning                                                               | Percent retention after 1000 hrs exposure |
|-----------------------------------|------------------|----------------------------------------------------------------------------|-------------------------------------------|
| Water resistance                  | D2247            | 93 percent relative humidity at 86°F ± 2°F (Maximum Env. Chamber Capacity) | 90 %                                      |
| Salt water resistance             | D1141            | Immersion at 73°F ± 2°F                                                    | 90 %                                      |
| Alkali resistance                 | C581             | Immersion in Ca(CO <sub>3</sub> ) at pH = 9.5 and 73°F ± 3°F               | 90 %                                      |
| Dry heat resistance               | D3045            | 140°F ± 5°F                                                                | 90 %                                      |

The concrete substrate's 28 day compressive strength was equivalent to 8026 psi (55.3 MPa) based on 5 compressive cylinder tests. Concrete samples were 4in X 4in X 2in (100mm X 100mm X 50mm). One layer of the CFRP and primer system was applied to the concrete surface and all samples were submerged underwater to cure. After 5 days of curing, the control samples were removed from water and left to dry for 24 hours prior to being tested whereas 3 durability samples were submerged in each of the indicated environments and left to age for 1,000hrs.

After the 1,000hr exposure, the samples were left to dry for 24 hours and scored through the FRP layer to the concrete surface using a 2in (50.8mm) diamond coring drill as indicated in ASTM D7234. Two stainless steel dolly were then attached to the top of FRP surface using an adhesive epoxy.

## RESULTS

#### Tensile material properties of the prepreg system when cured under and above water

Tensile properties of the CFRP system cured in laboratory condition and below water are shown in Table 3.

**Table 3 – Average tensile properties of the prepreg CFRP system when cured in room temperature and under-water**

|                                         | Laboratory conditioning    | Cured under-water           |
|-----------------------------------------|----------------------------|-----------------------------|
| Tensile Strength per Unit Width (lb/in) | 4,014 lb/in (702.9 N/mm)   | 3,933.21 lb/in (688.8N/mm)  |
| Tensile Modulus per unit width (kip/in) | 354,943 lb/in (62.2 kN/mm) | 318,022 kip/in (55.69kN/mm) |
| % Elongation                            | 1.15                       | 1.26                        |
| Nominal Ply thickness (in/mm)           | 0.034in (0.86 mm)          | 0.035in (0.89mm)            |

Failure mode for all samples was longitudinal splitting and/or lateral as shown in Figure 2. The results show that there is a 2% and 10% reduction in tensile strength and tensile modulus, respectively when samples are cured underwater.

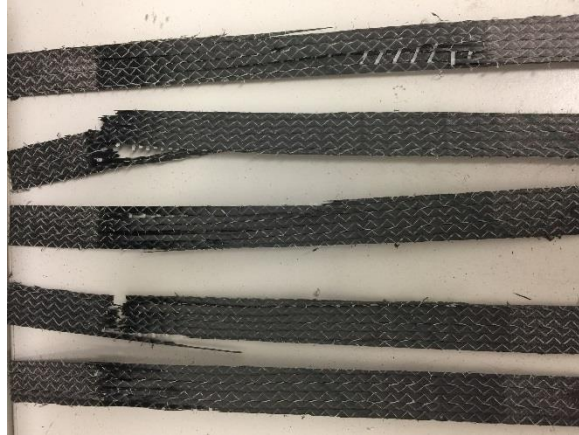


**Figure 2 – Failure mode of underwater tensile samples**

Concrete cylinder compression test results

A summary of the results of compression testing is shown in Figure 3 and Table 4. A typical failure mode of the samples is shown in Figure 4. The results show that all three systems result in an increase of compressive strength of the concrete cylinders. The samples cured above water resulted in an increased strength of ~20%, whereas the samples cured underwater, resulted in an increased strength of 13%.

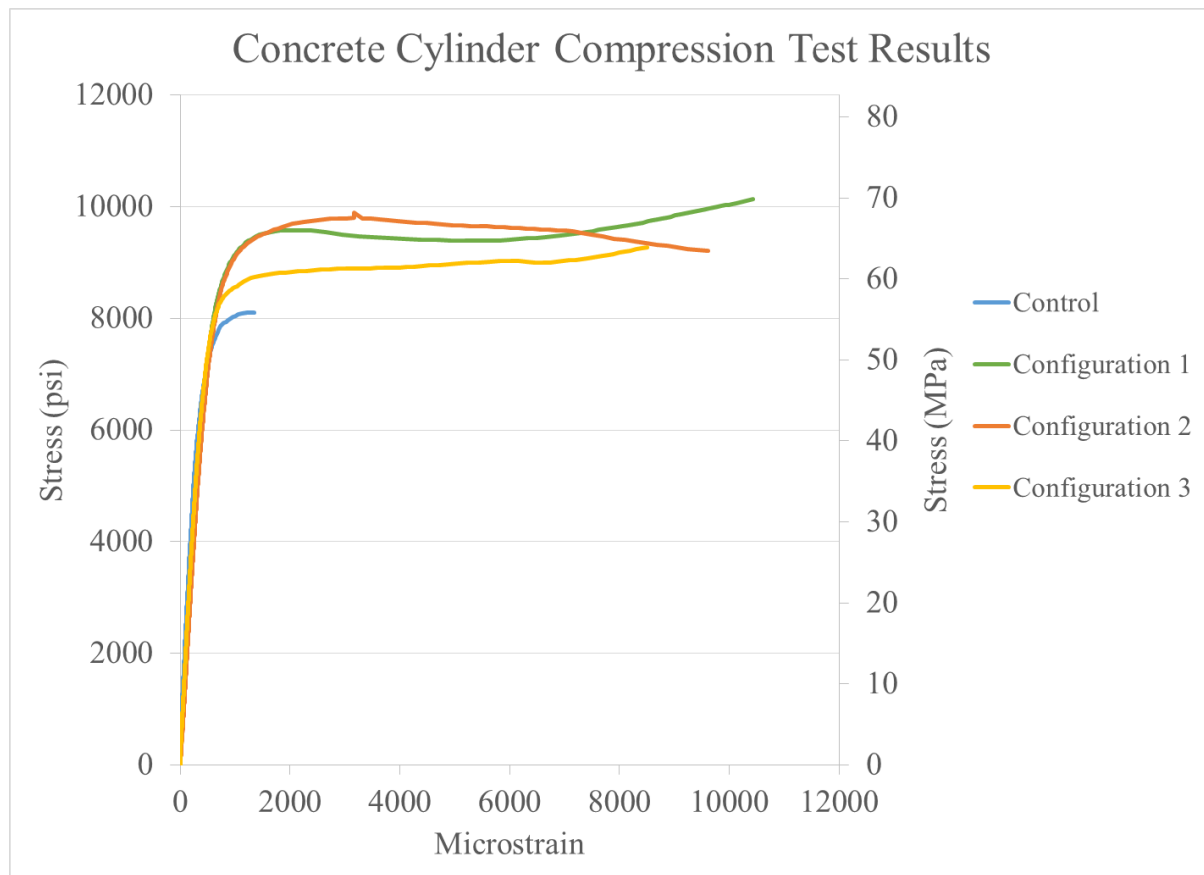


**Figure 3 – Compression results of the carbon prepreg system**



**Figure 4 – Typical failure modes during compression testing**

The experimental values were compared with theoretical calculations. The stress-axial strain model used to estimate theoretical values are based on Lam and Teng confinement theory<sup>10</sup>, which is also used in FRP design guide for reinforced concrete, in ACI 440.2R-08<sup>8</sup>. The equation used to determine the compressive strength of the confined concrete based on this theory is:

$$f'_{cc} = f'_c + \Psi_f 3.3k_a f_l \quad (1)^{10}$$

$$f_l = \frac{2E_f n t_f \varepsilon_{fe}}{D} \quad (2)^{10}$$

Where:

- $f'_{cc}$  is the compressive strength of confined concrete with FRP
- $f_l$  is the maximum confining pressure due to FRP jacket
- $E_f$  is the modulus of the laminate material
- $D$  is the effective depth of cylinder with FRP, approximately the diameter of the tested cylinders
- $t_f$  is the thickness of the FRP laminate
- $\varepsilon_{fe}$  is the effective strain of the FRP laminate
- $n$  number of layer of laminate applied
- $\kappa_a$  is the efficiency factor for FRP reinforcement in determination of  $f'_{cc}$ , which for circular cylinders is 1
- $\Psi_f$  is the strength reduction factor, which is 0.95 for fully wrapped sections.

**Table 4 – Average tested values versus theoretical calculations of the prepreg CFRP system when cured in room temperature and under-water**

| Systems Tested         | Avg. Tested $f'_{cc}$ | Theoretical $f'_{cc}$  | % Difference in theoretical vs tested value | % Increase in compressive strength of concrete |
|------------------------|-----------------------|------------------------|---------------------------------------------|------------------------------------------------|
| <b>Control</b>         | 8,026psi<br>(55.3MPa) | N/A                    | N/A                                         | 0%                                             |
| <b>Configuration 1</b> | 9,989psi<br>(68.9MPa) | 10,022psi<br>(69.1MPa) | 0.3%                                        | 20%                                            |
| <b>Configuration 2</b> | 9,895psi<br>(68.2MPa) | 10,000psi<br>(68.9MPa) | 1%                                          | 19%                                            |
| <b>Configuration 3</b> | 9,186psi<br>(63.3MPa) | 9,724psi<br>(67.0MPa)  | 5%                                          | 13%                                            |

The results, summarized in Table 4, are shown to be matching the experimental values. Compression test results are also in agreement with the tensile material properties of the composite since the input for theoretical calculations include the material properties of the composite. Since the percent difference in theoretical versus tested values for all

## Advantages of Prepreg FRP Systems for Extending Service Life of Concrete Members in Wet Environments

systems is less than 5%, it can be seen that the two tests are in agreement with each other and the same confinement theory as described in ACI 4402R-08 may be applied for calculations underwater.

### 1,000hr durability dolly pull-off test results

Table 5 below summarizes the test results of the durability study of the prepreg CFRP system, bonded to concrete substrate. The failure mode of all samples is shown in Figure 5. As shown in the results table the system was able to retain >90% of its bond strength to concrete when exposed for 1,000hrs to the various environments. The primary mode of failure was in the substrate.

**Table 5 – 1,000hr durability bond strength testing**

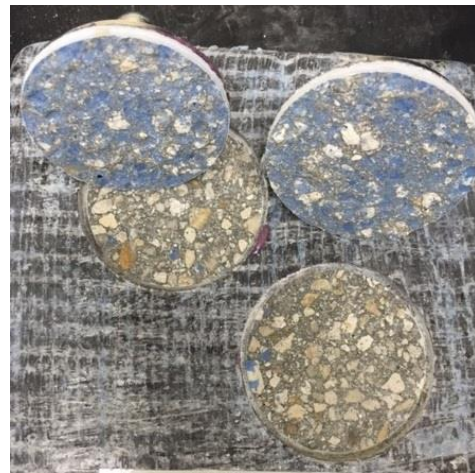
| <b>Environments</b>          | <b>CFRP system<br/>felt (psi)</b> | <b>Retained bond<br/>strength (%)</b> |
|------------------------------|-----------------------------------|---------------------------------------|
| <b>Control</b>               | 376                               | N/A                                   |
| <b>Water resistance</b>      | 356                               | 95%                                   |
| <b>Salt water resistance</b> | 348                               | 93%                                   |
| <b>Alkali resistance wet</b> | 399                               | 106%                                  |
| <b>Dry heat resistance</b>   | 379                               | 101%                                  |



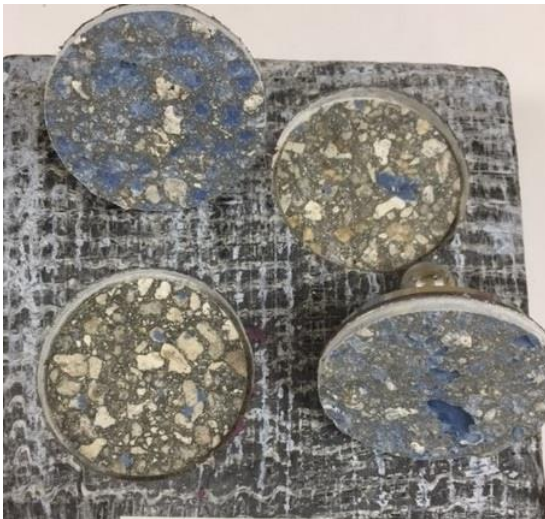
**(a) Control sample failure mode**



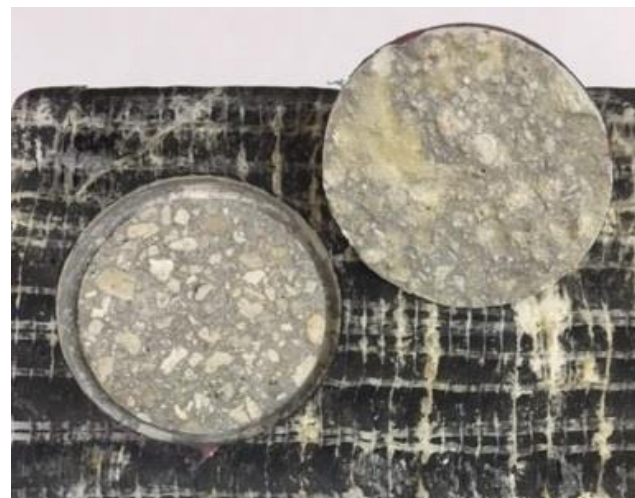
**(b) Water resistance samples' failure mode**



**(c) Salt water resistance samples' failure mode**



**(d) Alkali resistance samples' failure mode**



**(e) Dry heat resistance samples' failure mode**

**Figure 5 – Failure mode of bond strength samples: (a) control, (b) 100% humidity, (c) salt water, (d)alkali, and (e) dry heat environment**

### **SUMMARY AND CONCLUSIONS**

Tensile and confinement testing of a new prepreg system has been completed demonstrating that wet lay-up system may be used for splash-zone applications to strengthen and protect reinforced concrete. The material properties results are in agreement with concrete confinement equations currently in ACI 440.2R-08, which show that this system is able to strengthen concrete even when applied and cured under water. The anticipated loss in tensile strength and modulus of the system when applied and cured underwater is 2% and 10%, respectively. Bond strength tests of concrete samples wrapped with the prepreg system, cured underwater, and exposed to salt, alkalie, dry, and 100% humidity environment retained > 90% of their initial strength.

### **ACKNOWLEDGEMENTS**

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## CFRP REPAIRING SYSTEM AT OPENINGS IN REINFORCED CONCRETE T-BEAMS CRACKED BY IMPACT LOADS

Nazar Oukaili and Abeer Al-Shammari

**Synopsis:** This study presents the experimental and theoretical response of 10 simply supported reinforced concrete T-beams with multiple circular web openings to concentrated static and impact loading. Carbon fiber reinforced polymer (CFRP) was used for strengthening some of the beams with web openings. Five beams were exposed to static loading up to failure while the other beams were subjected to repeated impact loading until the shear crack width reached 0.3 mm (0.012 in.). The residual static strength of these beams was then determined. Test variables included the number of web openings and the height of weight drop during impact tests. Static loading test results indicated that unstrengthened beams with four and six openings have strength less than those without openings by 30% and 41%, respectively. However, strengthening using CFRP installed on beams web around openings resulted in an increase in the strength capacities ranging between 32% and 93%. Impact load data indicated that the beam with four openings did not show a significant increase in midspan deflections as compared with the specimens with no web openings. In comparison, the beam with six openings showed a 75% increase in the maximum midspan deflections compared with the beam with no web openings.

**Keywords:** CFRP strengthening, finite element, impact loading, multiple web openings, RC beams

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## INTRODUCTION

In some cases, it is necessary to create multiple openings in the web of a beam to permit the installation of utility ducts and pipes in order to reduce the floor height. This may also results in savings in material and construction costs, especially in multistory buildings. Creating openings in the web of a beam reduces the beam's cross-sectional area and therefore decreases its shear capacity while increasing deflection. If openings are planned before casting the beam, internal reinforcing steel bars can be placed around the openings to provide the required strength. Alternately, strengthening using external steel plates or fiber reinforced polymer (FRP) materials can be used to restore the original capacity. The combination of the low weight, high strength-to-weight ratio, and ease of installation of fiber reinforced polymer (FRP) materials is the main advantages of this type of material. Moreover, since FRP materials have high resistance to corrosion, they are satisfactory in terms of durability requirements. For the current study, carbon fiber reinforced polymer (CFRP) fabric was used for beams strengthening.

There are many cases in which a structural member may undergo impact or dynamic loading that reduces its capacity. Repeated impact loading. The residual strength of such member depends on several parameters governing the structural response. The residual strength of the structural member is usually of primary concern because the remaining service of the structure may depends on its ability to withstand additional impact and/or static loading.

El Maaddawy and Sherif (2009) investigated the use of FRP sheets for shear strengthening of reinforced concrete deep beams with square openings. Test parameters included the opening size, location, and the presence of the CFRP sheets. The shear strength gain caused by CFRP sheets was in the range of 66–71% when the opening was located at the mid-point of the shear span. When the opening was located at the top of the beam, the shear strength gain was a maximum (72%) where most of the shear force was carried by the bottom chord that was fully wrapped with CFRP. Only a strength gain of 35% was recorded for the beam with bottom openings because most of the shear force was carried by the top chord that had a U-shaped CFRP sheet. Hassan et al. (2015) used Finite Element Analysis in order to study reinforced concrete beams with opening strengthened using FRP. Fifty-seven beams were analyzed using finite element program ANSYS V12. Strengthening of all beams with opening came out to six types of different scheme around the opening using fiber-reinforced polymer (FRP). They concluded that the failure loads are mainly affected by strengthening scheme and are slightly affected by spacing or number of layers. El-kareim and El-sayed (2017) investigated the reduction of shear strength due to the introduction of an opening in loaded RC beams. El-kareim and El-sayed demonstrated that for an existing building, creating openings in RC beams caused a reduction in shear force where for RC beams with circular openings have diameters equal to 0.25d and 0.50d (d is the effective depth), the shear forces were equal to 65% and 37% from the shear force from the control tested RC beams, and for created openings under loading, more reduction occurred.

## RESEARCH SIGNIFICANCE

While there have been multiple studies recently published on the behavior of reinforced concrete beams under dynamic loads, none of these studies investigated the behavior of reinforced concrete T-beams with multiple web openings under impact loading. The current research studies the effect of the number of circular web openings and the height of drop of a mass during the impact test on the capacity and deflection of concrete beams. The experimental program investigated the effects of static and impact loading and responses of reinforced concrete beams with four or six circular openings. The loss of structural capacity due to the creation of the multiple web openings and the strength increase with using external CFRP fabric around the openings to restore the original behavior is also investigated. A

three dimensional finite element model was generated using, ANSYS 12.1 software to simulate and analyze the tested beams. In order to verify the analysis results and to be able to predict behavior of other test variables without testing, these analysis results were compared with the experimental results and showed good agreements.

### EXPERIMENTAL PROGRAM

#### Details of Test Beams

The experimental program consisted of testing two groups of simply supported reinforced concrete (RC) T-beams. The five beams of group (I) were tested under static loading and the five beams of group (II) were tested under impact loading. Details of the test beams are given in Table 1. Two of these beams were manufactured as solid beams with no openings, four beams with no strengthening around openings, and the remaining four beams were reinforced around openings with CFRP fabric. The two solid beams were used as reference (control specimens), one for static testing and the other for impact testing. The beam designation adopted is as follows:

B = beam, S = static, I = impact, W = without openings, O = with openings, 4 or 6 = number of web openings, and F = carbon fiber, which was used for strengthening at openings. All openings were circular with a diameter of 110 mm (4.3 in.). The ratio of the opening diameter to the web depth was 48%, which was sufficient to cause a sizeable reduction in shear capacity and enable evaluation of the efficiency of the strengthening technique. The distance from the support to the beginning of the first opening was 130 mm (5 in.), which is equal to about half the effective depth of the beam. In general, beams with multiple web openings may experience failure in the post (the element between two openings) when the width of the post is smaller than 3/8 of the depth of the web (ACI 426R-74 1973). In this study, the width of the post was selected to be about 5/8 of the web depth. In order to investigate the shear behavior of the test specimens, the solid control beam was designed to fail in shear. The specimens were designed with flexural capacity that exceeds their shear capacity. Specimen Designs were achieved per ACI 318-11 (2011). A schematic diagram of the test specimens showing dimensions and reinforcement details is depicted in Figure 1 (a). The length of each test specimen was 2000 mm (78.7 in.) with an effective span of 1800 mm (71 in.) and effective depth (d) of 256 mm (10 in.). The tension steel reinforcement consisted of two bars of 20 mm diameter (3/4 in.), while the compression reinforcement consisted of four bars of 6 mm diameter (1/4 in.). Stirrups consisted of 4 mm diameter (1/6 in.) bars placed at 130 mm (5 in) center-to-center. This reinforcement for test beams is shown in Figures 1(b), 1(c) and 1(d). Properties of the concrete, the steel reinforcement, and fiber reinforced polymer used in this study are given in Table 2.

#### CFRP Strengthening Configuration

Beams BSO4F, BSO6F, BIO4F, and BIO6F were strengthened using unidirectional CFRP fabric with, which was placed around the openings in two layers. The first layer consisted of horizontal and vertical strips of CFRP fabric placed around openings, while the second layer consisted diagonal strips of CFRP fabric, as shown in Figure 2. The concrete substrate was roughened and then cleaned prior to CFRP installation. An epoxy resin adhesive with an average thickness of 1 mm (0.04 in.) was applied to the substrate prior to CFRP installation.

#### Static Test Set-up and Instrumentation

Static beam specimens were tested using simple supports configuration under a monotonic concentrated load applied at midspan. The beams were loaded up to failure using a 3000 kN (675,000 lb) capacity hydraulic universal testing machine. All measurements, such as beam deflections, strains in steel bars, and strains in concrete were recorded twice, immediately after the application of the load and after 10 minutes from load application. Two dial gages of 0.01 mm (0.0004 in.) accuracy were used to measure vertical displacement at mid and quarter span. Electrical strain gauges were used to measure the strains in steel and concrete. Cracks were monitored along the span of the beams. Maximum crack width, number of cracks, and average crack spacing were measured sequentially during loading. Figure 3 shows the static test setup and instrumentation locations for the test specimens.

#### Impact Test Set-up and Instrumentation

A drop mass impact loading frame was used for impact tests as shown in Figures 4 and 5. Each beam was subjected to a drop mass of 24.5 kg (54 lb), which was dropped freely one time only at beams' midspan from seven different heights: 100, 250, 400, 750, 1000, 1500, and 2000 mm (3.94, 9.84, 15.75, 29.53, 39.37, 59.06, and 78.74 in.). The impact drop heights (H) represents different impact (input) energy =  $0.5 m v^2$  (where: m = mass; and v = impact velocity =  $\sqrt{2gH}$ ). It was easier to change the height of the drop (H) rather than the dropping mass (m) to induce higher energy levels. The impact load was then repeated with a drop height of 1000 mm (39.37 in.) until the width of

the shear crack reached the pre-determined serviceability limit of 0.3 mm (0.012 in.) crack width. The drop mass system consisted of three parts: a steel cylindrical shaft, a dynamic load cell (MLC213 model) and a stainless steel impact head with a curved tip. The RC beam was simply supported over a span of 1800 mm (71 in.) with specially designed support devices, two steel clamps, which used to restrain the vertical movement of the beam at the supports. In order to assure that the beam was still simply supported, these clamps were pinned to the supports to allow rotation during beam loading. To permit an easier rotation, round steel bars were welded to the supports and to the clamps where they touch the beam.

The contact force developed between the mass and the RC beam was measured using the dynamic load cell, which was rigidly connected to the drop mass system between the steel shaft and the impact head. The midspan deflection response of the RC beams was measured using linear variable differential transducers (LVDT). Strains at different locations were measured using electrical strain gauges. Signals from the load cell, strain gauges, and LVDTs were recorded using digital data acquisition system.

### **Residual Load Carrying Capacity Test**

The residual load carrying capacity of the impact beams was determined by static loading in which each beam loaded at midspan up to failure. The ultimate loads were recorded and compared with the ultimate loads of the beams which were subjected to static loading only.

## **STATIC TEST RESULTS**

### **Cracking Behavior, Failure Loads and Mode of Failure**

A summary of first cracking loads, ultimate loads, and mode of failure are presented in Table 3. The loads recorded did not include the self-weight of the specimens. During each test, cracks were marked on one face of the specimen at each load increments. Figure 6 shows crack patterns for all specimens. The solid control beam, BSW, failed in shear mode. This specimen showed several flexural tension cracks at early stages of loading, at 40 kN (9000 lb), then at about 40% of the ultimate load, a number of inclined cracks initiated from these cracks to form flexural-shear cracks. At the same load level, diagonal tension cracks appeared near the mid-height of the beam, between the existing inclined cracks and the supports, and propagated toward the point of loading and the supports. Splitting cracks initiated from the diagonal tension crack (failure crack) that were oriented along the longitudinal reinforcement at an applied load of about 75% of the ultimate load. As the applied load increased, these cracks widened and continued to propagate toward the point load and toward the supports. A brittle shear failure of the control beam BSW occurred at a load of 160 kN (36,000 lb).

Unlike the solid beam, beams BSO4 and BSO6 showed fewer flexural cracks at 40 kN (9000 lb) load, and these cracks remained inactive. Several diagonal cracks initiated from the openings, passing through the flexural cracks and extending toward the point of the applied load and the supports. A sudden shear failure occurred at 112 kN (25,000 lb) and 95 kN (21,000 lb) for specimens BSO4 and BSO6, respectively. Beam BSO4F showed several flexural cracks at 20 kN (4500 lb) load. Several inclined cracks initiated from flexural cracks to form flexural-shear cracks at 40 kN (9000 lb) load, which continued to grow in size and propagated toward the point load and supports until failure occurred at a load of 147.5 kN (33,000 lb). The presence of these cracks was limited to the unstrengthened regions of the beam. Cracks could not be identified for Beam BSO6F since the applied CFRP covered almost the entire surface area of the beam. At failure, buckling of CFRP fibers was noticed, at concrete crushing locations, which appeared in an advanced stage of loading. Ultimate load was 182.5 kN (41,000 lb).

### **Load- Deflection Relationships**

The load versus mid-and quarter-span deflection curves for all beams are presented in Figures 7 and 8. From these figures, it can be seen that the introduction of openings into these beams causes an increase in the deflection values and a decrease in their stiffness at similar load levels. This behavior depends on the number of openings. Strengthening with CFRP installed around the openings increased beam stiffness and decreased the measured deflections at similar levels of loading. In addition, the beam specimens strengthened with CFRP behaved in a more ductile manner.

## IMPACT TEST RESULTS

### **Cracking, Failure Mode and Number of Drops**

The first visible crack was a vertical crack which appeared under the point of the impact on the flange of all tested beams. This was followed by a flexural crack which started at the soffit of the beam and propagated upward. Shear cracks appeared after several drops from a height of 1000 mm (39.37 in.). These cracks started near mid-height of the beam and propagated diagonally with an inclination angle of 40°– 60°. All beams demonstrated this behavior with the exception of two specimens: BIO6, which did not show web flexural cracking, and BIO6F, which could not be examined for cracking since CFRP covered almost the entire web area.

For all beams, the impact test was stopped when the width of the shear crack reached 0.3 mm (0.012 in.). A crack detection microscope was used for this purpose. The number of drops required to reach the serviceability limit state were recorded and the results are presented in Figure 9. Compared with the solid beam (BIW), it is concluded that the number of drops required to form shear cracks with a 0.3 mm (0.012 in.) width decreased when four and six openings were created in specimens BIO4 and BIO6, by about 30% and 86%, respectively. When opening regions were strengthened with CFRP fabric in specimens BIO4F and BIO6F, the number of drops required to form shear cracks with a 0.3 mm (0.012 in.) width increased by 85% and 125%, respectively.

### **Impact Response**

Impact load-time histories were recorded for each beam for drop heights of 100, 250, and 400 mm (3.94, 9.84 and 15.75 in.) and the deflection-time histories were recorded for each beam for the seven drop heights. Impact force- time history results for specimen BIW for different drop heights are presented in Figure 10. From this figure, it can be seen that the measured impact loads are characterized by two parts: the first part is the impact force induced by the dropping the mass, which represents the incidental wave with comparatively high amplitude; the second part is the inertia force produced by the vibration of the beam. This part has comparatively low amplitude. These results are similar to was presented in previous studies (Kishi et al. 2001; Bhatti et al. 2009; Fujikake et al. 2009). As the height of drop increases, the peak of the first part of the impact force increases. Figure 11 shows midspan deflection-time history for the BIW specimen from different drop heights. From these results, it can be noticed that the maximum midspan deflection increases with the increase of drop height.

Figure 12 shows comparisons of the maximum impact force for specimens BIW, BIO4, and BIO6. This figure indicates that the maximum impact force values of specimen BIW are slightly larger than those of specimens BIO4 and BIO6. This indicates that the impact and inertia forces depend on the structural stiffness: the higher the stiffness, the larger the impact and the inertia forces. Figure 13 shows comparisons of the maximum transient midspan deflections for the seven impact-tested beams. From these figures it can be concluded that the maximum midspan deflections increase as the height of the drop is increased with the increase being almost linear for the first three drop heights. From these figures it can also be concluded that the beam with four openings does not show an obvious increase in the maximum midspan deflections, while the beam with six openings clearly shows a 75% greater increase in the maximum midspan deflections in comparison to the solid beam. Strengthening of around openings by CFRP fabric can overcome this increase in the deflection values, where the strengthening technique improves the deflection behavior of the specimen with four openings by reducing the maximum deflection value.

## RESIDUAL STATIC STRENGTH OF THE IMPACTED BEAMS

The residual static strengths of the impacted beams were obtained using static tests. The test results were then compared with the strength of beams tested under static loading only. The test results are presented in Table 4 and Figure 14. From these results it can be concluded that the percentage of the residual static strength decreases with the increase of the number of drops to which the beam is subjected. It can also be seen that the maximum difference between the percentages of the residual static strength of impacted beams does not exceed 15%. This may be caused by the failure limit that considered for all beams tested under impact loading, namely (when the width of shear crack reaches 0.3 mm (0.012 in.)). Figure 15 shows the cracking patterns due to static and impact loading of the. During the

static tests, all beams showed additional cracks, which propagated vertically and diagonally and increased in width until failure of the beam. All beams failed in shear mode, as that for the pure static tested beams.

## FINITE ELEMENT MODELING

### **Element Types and Modeling**

Test beams were modeled with SOLID65 elements with eight nodes, three degrees of freedom (DOF) at each node, and translations in the nodal x, y, and z directions. The element is capable of plastic deformation and cracking in three orthogonal directions. LINK8 elements capable of plastic deformation with three dimensional spar element with two nodes and three degrees of freedom were used to represent the steel reinforcement. Similarly, a layered structural solid element SOLID185 was used to model the CFRP fabric. This element allows for up to 250 different material layers with different orientations and orthotropic material properties in each layer. Solid45 element with eight nodes and three degrees of freedom per node was used for modeling the steel plates at supports and loading locations to prevent stress concentration issues and to provide stress distribution over the support and loading areas.

By taking advantages of the symmetry of the beams, a quarter of the full beam was modeled. This approach significantly reduced computational time and computer hard space requirements. Perfect bond was assumed between different materials and the self-weight of the beams was considered in analysis. Figure 16 shows volumes created in the ANSYS program.

### **Material Properties**

Concrete: SOLID65 elements are capable of predicting the nonlinear behavior of concrete material using a smeared crack approach (William and Wranke 1975). Compressive and tensile strength and modulus of elasticity of concrete used for ANSYS modeling are given in Table 2. Poisson's ratio for concrete was assumed to be 0.2 (Bangash 1989). Shear transfer coefficients for modeling shear transfer at cracks were varied from 0.0 to 1.0, where 0.0 represents a smooth crack with complete loss of shear transfer and 1.0 represents a rough crack with no loss of shear transfer. The shear transfer coefficients for open and closed cracks selected were equal to 0.3 and 0.5, respectively.

Steel Reinforcement: elastic modulus and yield stress for the steel reinforcement used in the ANSYS program follow the design material properties used for the experimental investigation (Table 2). The steel for the finite element models is assumed to be an elastic-perfectly plastic material and identical in tension and compression strengths. A Poisson's ratio of 0.3 is used for the steel reinforcement.

CFRP composites: input data needed for the FRP in the finite element model is given in Table 5.

### **Meshing, Boundary Conditions and Static Loading**

A convergence study was carried out to determine an appropriate mesh density. A mesh size of 20 mm (0.787 in.) was selected based on the study. Because only one quarter of the entire beam was modeled, planes of symmetry at the internal faces and rollers were used to show the symmetry conditions. In the support region, vertical movement was restricted while rotation was permitted to avoid excessive cracking of concrete elements at this region. The external load was applied to the centerline of the beam on the edge of the modeled beam. The external applied load was represented by the equivalent nodal forces on the top nodes of the centerline of the flange as a line load with width equal to the width of the web to avoid flange failure. Figure 17 shows finite element mesh, boundary conditions, and static loading.

### **Static Finite Element Analysis**

Static analysis type was utilized in this part of study. The Newton-Raphson method was used to compute the nonlinear response of the modeled beams. The application of the loads up to failure was done incrementally as required by the Newton-Raphson procedure, and the total load was divided into a series of load increments (load steps). Within each load step, a maximum of (100) iterations was used. A convergence failure can indicate a physical failure of the structure, or it can merely be the result of some numerical problem in the finite element model. Since in all cases, convergence was achieved before reaching the maximum (100) iterations; that means, failure for each of the models is achieved when the solution for a minimum load increment fails to converge (physical failure).

## **Impact Finite Element Loading and Analysis**

The impact (transient) loading is a special class of dynamic loading. Such a load consists of a single principal impulse of relatively short duration. Damping has much less importance in controlling the maximum response of a structure to impact loads than for periodic or harmonic loads because the maximum response to a particular impulsive load will be reached in a very short time, before the damping forces can absorb much energy from the structure (Clough and Penzien 2003). For this reason only the undamped response to impact loads will be considered in this analysis.

Transient dynamic analysis (or time-history analysis) is a technique used to determine the dynamic response of a structure under the action of any general time-dependent loads. This type of analysis can be used to determine any time-varying displacements, strains, stresses, or forces. Three methods are available in the ANSYS program to perform a transient dynamic analysis: full, mode superposition, and reduced. ANSYS analysis was done using the reduced method which condenses the problem size by using master degrees of freedom DOF and reduced matrices. After the displacements at the master DOF have been calculated, the ANSYS program expands the solution to the original full DOF set. Multiple load steps are usually required to specify the load history in a transient analysis. The first load step was used to establish initial conditions and the second and subsequent load steps were used for the transient loading. Figure 18 shows the master DOF, load step, and boundary conditions.

## **STATIC ANALYSIS RESULTS**

### **Failure Loads**

Table 6 shows a comparison between failure loads of the experimental beams and the theoretical loads from the finite element models. It is seen that the ANSYS models overestimated the strength of the beams since the finite element method deals with the modeled members as a rigid body, demonstrating stiffer and stronger behavior than the real one.

### **Load-Deflection Relationships**

Comparisons between load-midspan deflection relationships for the experimental and finite element analysis results are presented in Figure 19. From these figures it can be seen that the load-deflection relationships from the ANSYS program agreed with the experimental relationships. The ANSYS program predicts the beam to be stiffer than the experimental one for most specimens; this behavior is due to the assumed perfect bond between concrete and steel reinforcement.

## **IMPACT ANALYSIS RESULTS**

Midspan deflection-time histories, as measured during tests and recorded from finite element analyses, are compared in Figure 20. In general, the agreement is good and the plots have similar trends. However, except for specimen BIO6, the peak deflections were overestimated. One of the reasons for this behavior is the applied impact force. In tests, loss in the impact force was expected, while in finite element analysis, the full impact was applied (damping was ignored as an assumption of the analysis).

## **SUMMARY AND CONCLUSIONS REMARKS**

Based on the study presented herein, the following conclusions can be drawn:

### **Static Tests**

- (1) Introducing four or six circular web openings with a diameter of 48% of the web depth in the shear zone of reinforced concrete T-beams reduced the strength capacity of these beams by 30% and 41%, respectively, increases beam deflections, and resulted in significant shear cracks around openings.
- (2) The application of the CFRP strengthening technique to beams with openings presented in this study greatly decreases beams' deflections, reduced cracks around openings, and increased the ultimate load capacity of these beams.

- (3) The increase in shear capacity of T-beams with four and six strengthened circular web openings strengthened with CFRP fabric was 32% and 93%, respectively. The load carrying capacities of these beams were 92% and 114% of the solid beams.
- (4) Shear failure at the opening regions of the CFRP strengthened beams occurred due to a combination of concrete shear cracking and buckling of FRP reinforcement.
- (5) Finite element method using the ANSYS program was used to simulate T-beams with multiple web openings. The general behavior of the finite element models represented by the load-deflection plots at midspan showed a good agreement with the experimental test data. However, the finite element models showed slightly more stiffness than the test results. The effects of bond slip between the concrete and steel reinforcing, and micro cracks occurring in the actual beams were excluded in the finite element models, contributing to the higher stiffness of the finite element models.
- (6) Failure loads from the finite element analyses were slightly higher than the ultimate loads from the experimental results by 3% - 5%.

### **Impact Tests**

- (1) The maximum impact force values (measured by the load cell) of the solid beam were slightly larger than those of beams with four and six openings. This indicates that the impact and inertia forces depend on the structural stiffness: the higher the stiffness, the larger the impact and the inertia forces.
- (2) The impact force (measured by the load cell) and midspan deflections of the reinforced concrete beams increase proportionally with increases in drop height.
- (3) The beam with four openings did not show a clear increase in the maximum midspan deflections, while the beam with six openings showed a 75% greater increase in the maximum midspan deflections than those of the solid beam. Strengthening of specimens around openings by CFRP fabric can overcome this increase in the deflections.
- (4) The number of drops required to form shear cracks of 0.3 mm (0.012 in.) width, the suggested serviceability limit, decreased when four and six openings were created by about 30% and 86%, respectively, when compared with the solid reference beams. This number of drops increased when the opening region was strengthened by CFRP fabric.
- (5) Numerical models using finite element analysis using the ANSYS program were developed to determine the maximum midspan deflection, which is an important index for evaluating damage levels of the reinforced concrete beams subjected to impact loading. The numerical models were shown to be in good agreement and the plots have similar trends to the experimental curves. However, the predicted peak deflections were overestimated.

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**Table 1—Details of the experimental specimens**

| Group I | Beam symbol | Number of openings | Type of strengthening at opening regions | Group II | Beam symbol | Number of openings | Type of strengthening at opening regions |
|---------|-------------|--------------------|------------------------------------------|----------|-------------|--------------------|------------------------------------------|
| Static  | BSW         | Without openings   | ---                                      | Impact   | BIW         | Without openings   | ---                                      |
|         | BSO4        | 4                  | No strengthening                         |          | BIO4        | 4                  | No strengthening                         |
|         | BSO6        | 6                  | No strengthening                         |          | BIO6        | 6                  | No strengthening                         |
|         | BSO4F       | 4                  | CFRP fabric                              |          | BIO4F       | 4                  | CFRP fabric                              |
|         | BSO6F       | 6                  | CFRP fabric                              |          | BIO6F       | 6                  | CFRP fabric                              |

**Table 2—Material properties of the experimental specimens**

| Material    | Diameter or thickness, mm (in) | Yield strength, MPa (psi) | Compressive strength, MPa (psi) | Tensile strength, MPa (psi)    | Modulus of elasticity, GPa (ksi)               |
|-------------|--------------------------------|---------------------------|---------------------------------|--------------------------------|------------------------------------------------|
| Concrete    | ---                            | ---                       | 27 (4000)                       | 3.2 (464)                      | 23 ( $3 \times 10^6$ )                         |
| Steel       | 20 (0.8)                       | 600 (87,000)              | ---                             | 747 (108,000)                  | 200 ( $29 \times 10^6$ )                       |
|             | 6 (0.24)                       | 420 (61,000)              | ---                             | 570 (83,000)                   |                                                |
|             | 4 (0.16)                       | 400 (58,000)              | ---                             | 553 (80,000)                   |                                                |
|             | 8 (0.32)                       | 450 (65,000)              | ---                             | 602 (87,000)                   |                                                |
| CFRP fabric | 0.131 (0.005)                  | ---                       | ---                             | 4300 (62,000)                  | 238 ( $35 \times 10^6$ )                       |
| Epoxy resin | ---                            | ---                       | ---                             | 30 (4400)<br>at 7 days (+23°C) | 4.5 ( $0.7 \times 10^6$ )<br>at 7 days (+23°C) |

**Table 3—Summary of static test results**

| Specimen | First cracking load $P_{cr}$ , kN (lb) | Failure load $P_u$ , kN (lb) | $P_u/P_{u,BSW}$ , (%) | Shear capacity increase, (%) | Crack and failure mode                                              |
|----------|----------------------------------------|------------------------------|-----------------------|------------------------------|---------------------------------------------------------------------|
| BSW      | 18 (4050)                              | 160 (36,000)                 | 100                   | ---                          | Diagonal tension crack (shear failure)                              |
| BSO4     | 15 (3400)                              | 112 (25,000)                 | 70                    | ---                          | Shear cracks at opening regions (shear failure)                     |
| BSO6     | 12.5 (2800)                            | 95 (21,500)                  | 59                    | ---                          | Shear cracks at opening regions (shear failure)                     |
| BSO4F    | 20 (4500)                              | 147.5 (33,000)               | 92                    | 32                           | Flexural-shear cracks (shear failure)                               |
| BSO6F    | 27.5 (6200)                            | 182.5 (41,000)               | 114                   | 93                           | Diagonal tension cracks and buckling of CFRP fabric (shear failure) |

**Table 4—Residual static strengths of specimens subjected to impact load**

| Static tested beam symbols | Strength of static tested beams, kN (lb) | Impacted beam symbols | Static strength of impacted beams, kN (lb) | No. of drops exerted on impact tested beams | Residual static strength, (%) |
|----------------------------|------------------------------------------|-----------------------|--------------------------------------------|---------------------------------------------|-------------------------------|
| BSW                        | 160 (36,000)                             | BIW                   | 125 (28,000)                               | 60                                          | 78                            |
| BSO4                       | 112 (25,000)                             | BIO4                  | 92.5 (21,000)                              | 42                                          | 82.5                          |
| BSO6                       | 95 (21,500)                              | BIO6                  | 81.5 (18,500)                              | 8                                           | 85.5                          |
| BSO4F                      | 147.5 (33,000)                           | BIO4F                 | 110.5 (25,000)                             | 111                                         | 75                            |
| BSO6F                      | 182.5 (41,000)                           | BIO6F                 | 130 (29,000)                               | 135                                         | 71                            |

**Table 5—Summary of CFRP composite properties**

| FRP description           | Elastic modulus, GPa (ksi)          | Major Poisson's ratio | Tensile strength, MPa (psi) | Shear modulus, GPa (ksi)                 | Thickness of fabric, mm (in) | Ultimate elongation, (%) |
|---------------------------|-------------------------------------|-----------------------|-----------------------------|------------------------------------------|------------------------------|--------------------------|
| Woven carbon fiber fabric | $E_x = 238$<br>( $35 \times 10^6$ ) | $\nu_{xy} = 0.22$     | 4300<br>(624,000)           | $G_{xy} = 12.5$<br>( $1.8 \times 10^6$ ) | 0.131<br>(0.0052)            | 1.8                      |
|                           | $E_y = 19$<br>( $3 \times 10^6$ )   | $\nu_{xz} = 0.22$     |                             | $G_{xz} = 12.5$<br>( $1.8 \times 10^6$ ) |                              |                          |
|                           | $E_z = 19$<br>( $3 \times 10^6$ )   | $\nu_{yz} = 0.3$      |                             | $G_{yz} = 7.5$<br>( $1.1 \times 10^6$ )  |                              |                          |

**Table 6—Comparison between experimental and finite element failure loads**

| Specimen | Experimental loads, kN (lb) | Theoretical loads, kN (lb) | Difference, (%) |
|----------|-----------------------------|----------------------------|-----------------|
| BSW      | 160 (36,000)                | 165 (37,000)               | 3               |
| BSO4     | 112 (25,000)                | 116 (26,000)               | 3               |
| BSO6     | 95 (21,500)                 | 100 (22,500)               | 5               |
| BSO4F    | 147.5 (33,000)              | 151 (34,000)               | 3               |
| BSO6F    | 182.5 (41,000)              | 190 (43,000)               | 4               |

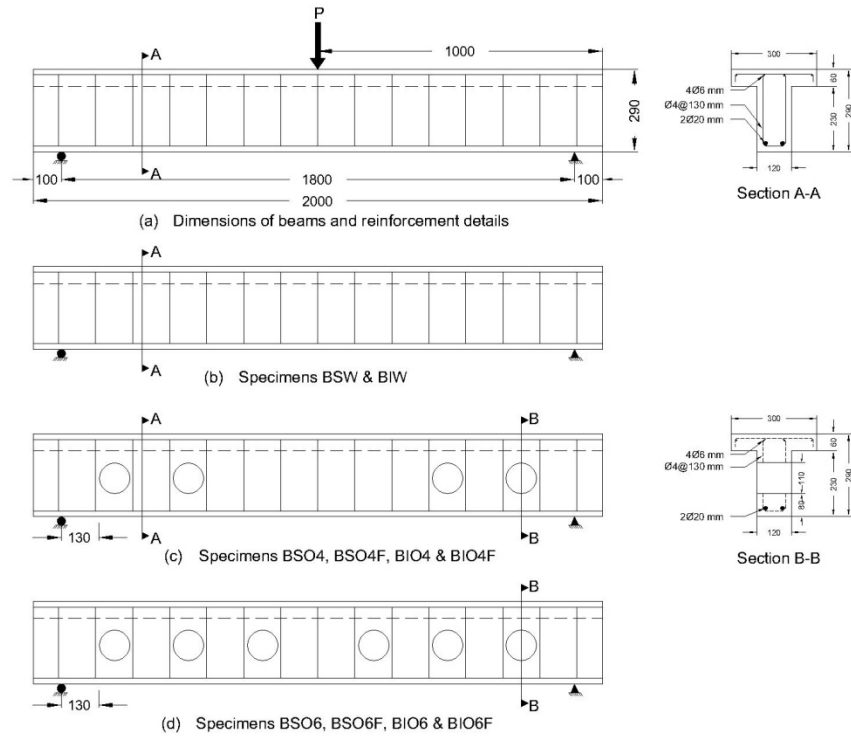


**Fig. 1—Details of beam specimens (unit: mm)**  
[1 mm = 0.0394 in.]



**Fig. 2—Alignment of CFRP fabric**

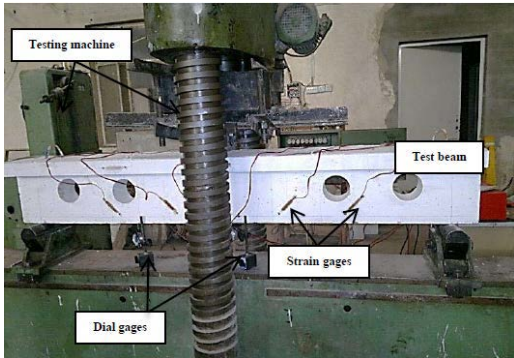


**Fig. 3—Universal static testing machine and instrumentation**

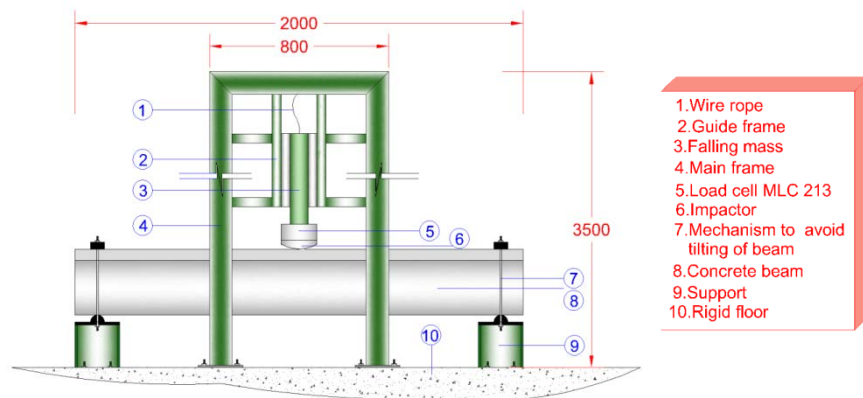


**Fig. 4—Experimental set-up for the dropping mass impact test (unit: mm)  
[1 mm = 0.0394 in.]**



**Fig. 5—Experimental set-up for the dropping mass impact test**



**Fig. 6—Crack patterns for static tested beams**

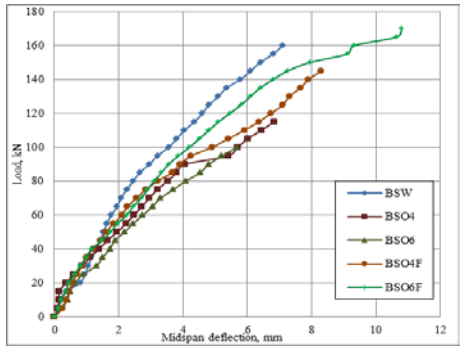


**Fig. 7—Load-midspan deflection relationships**  
[1 kN = 225 lb and 1 mm = 0.0394 in.]

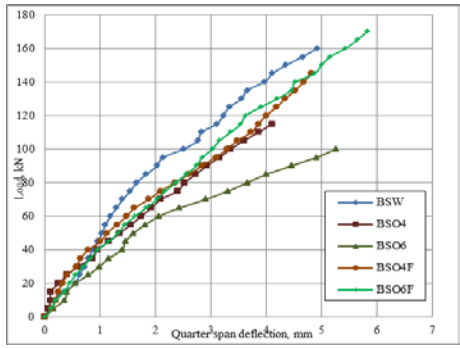


**Fig. 8—Load-quarter span deflection relationships**  
[1 kN = 225 lb and 1 mm = 0.0394 in.]

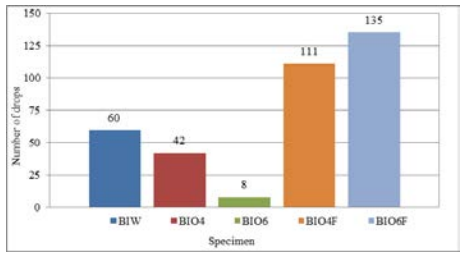


**Fig. 9—Number of blows required to reach the serviceability limit state for tested beams**

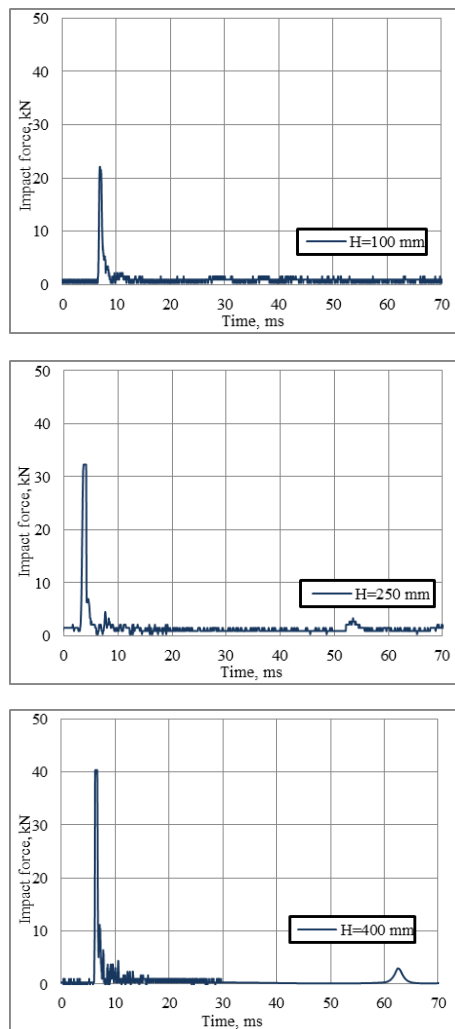


**Fig. 10—Impact force-time history for specimen BIW**  
[1 kN = 225 lb and 1 mm = 0.0394 in.]

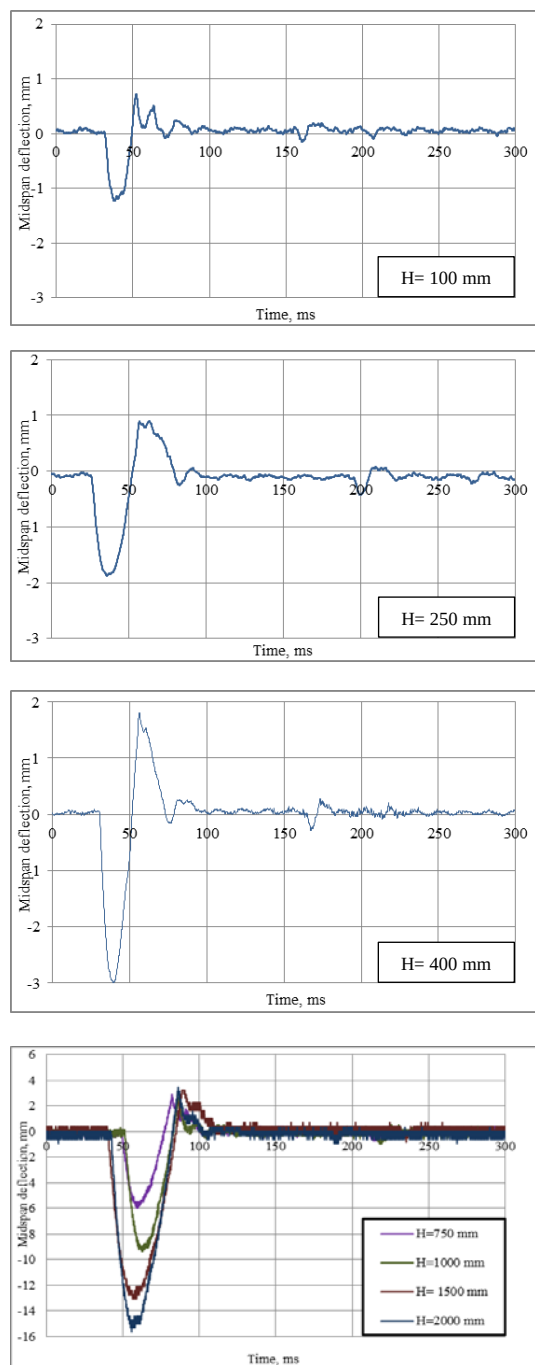


**Fig. 11—Midspan deflection-time history for BIW specimen from different drop heights  
[1 mm = 0.0394 in.]**

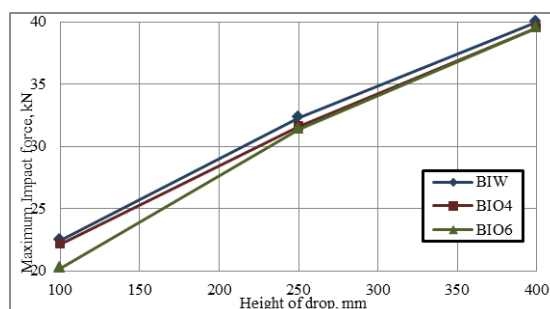


**Fig. 12—Maximum impact force of specimens BIW, BIO4 and BIO6 for different drop heights**  
[1 mm = 0.0394 in. and 1 kN = 225 lb]

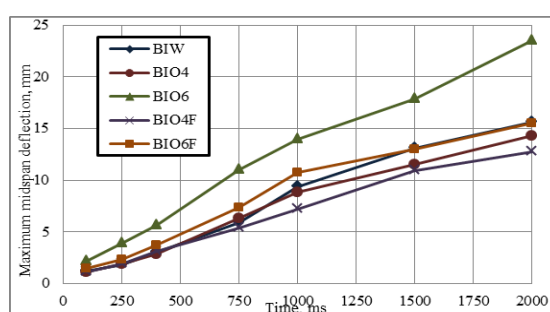


**Fig. 13—Maximum midspan deflections of specimens for different drop heights**  
[1 mm = 0.0394 in.]

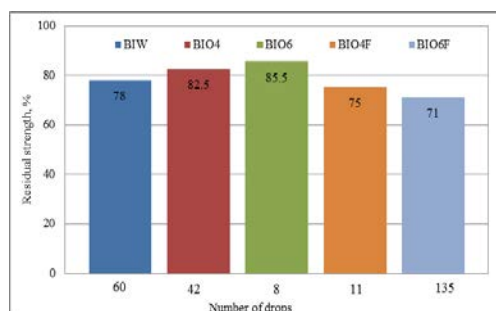


**Fig. 14—Residual strength-number of drops relationship for the impacted beams**



Fig. 15—Crack patterns for impact tested beams after static tests

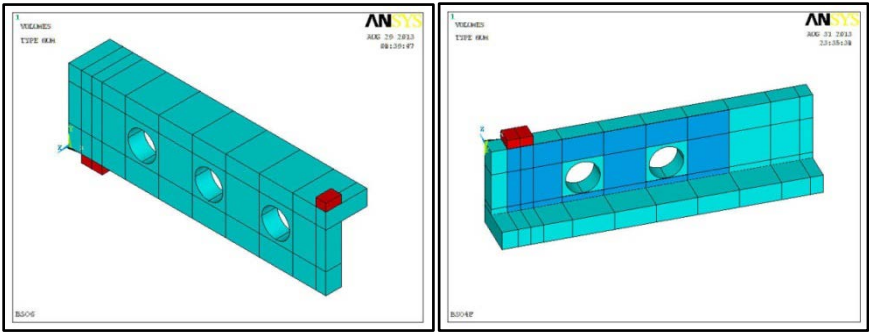


Fig. 16—Volumes created in ANSYS program for specimens BSO6 and BSO4F

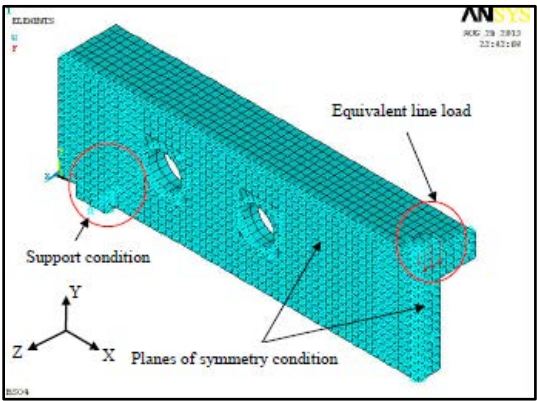


Fig. 17—Mesh, boundary conditions, and static loading

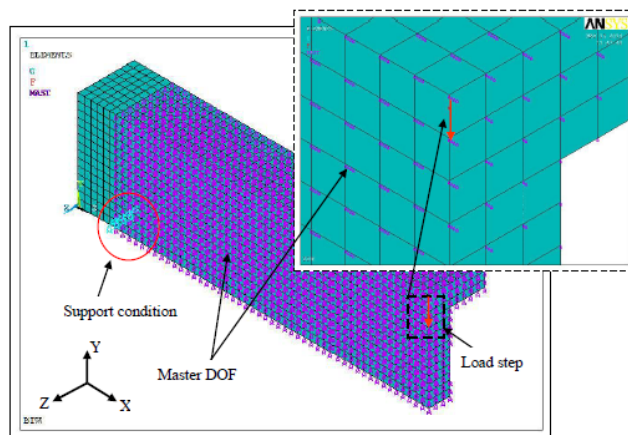


**Fig. 18—Master DOF, impact load step, and boundary conditions**

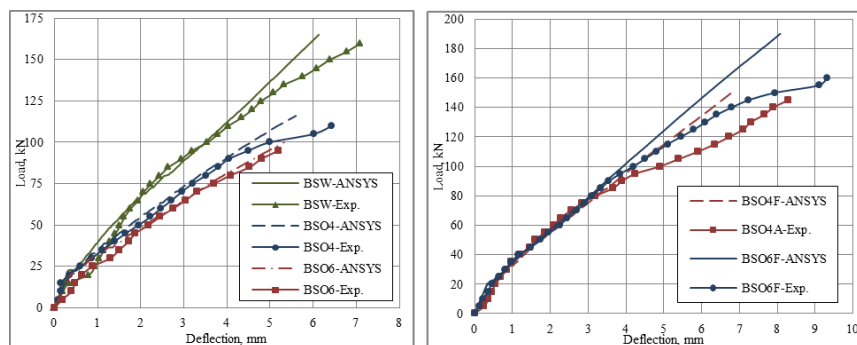


**Fig. 19—Predicted and experimental load-midspan deflection relationships  
[1 mm = 0.0394 in. and 1 kN = 225 lb]**

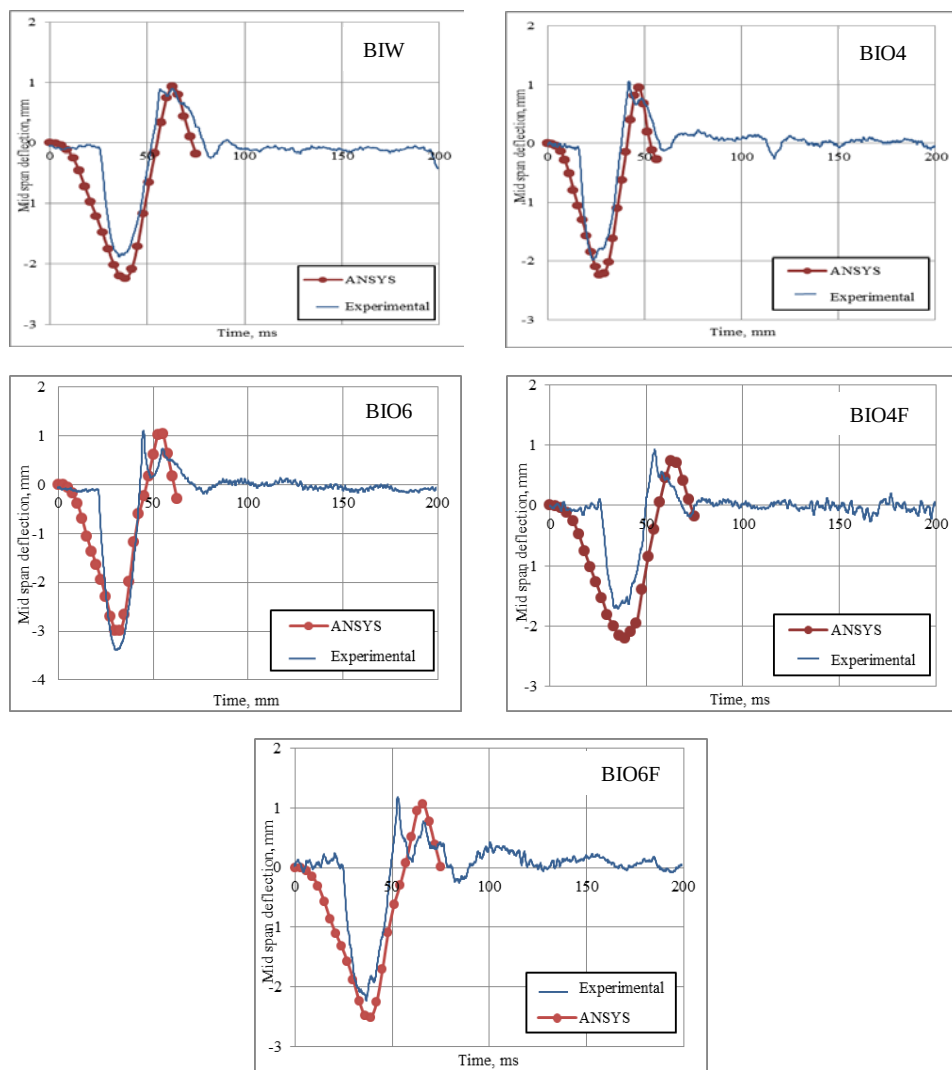


**Fig. 20—Experimental and theoretical midspan deflection-time history**  
[1 mm = 0.0394 in.]

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## Novel Composites Jacket for Repair of Concrete Columns

Allan C. Manalo, Ali Abdulkareem Mohammed, and Ginghis Maranan

**Synopsis:** This paper presents the development of a new type of fiber reinforced polymer (FRP) composites jacket and the evaluation to demonstrate the application of this pre-fabricated composite repair system for rehabilitation of deteriorating concrete columns. The novelty of this fiber composite strengthening system is that it is quick and safe to install due to the easy-fit and self-locking mechanical joint. An intensive characterisation of the mechanical properties of the composite resin for the interlocking teeth and locking key of the jacket was conducted. Similarly, finite element (FE) simulation on the behaviour of the FRP jacket subjected to internal pressure was performed and verified through full-scale experimental testing. Finally, the behaviour of concrete columns with simulated steel corrosion defect and repaired with a composite jacket was investigated. The results showed that the provision of FRP jacket restored the axial capacity of concrete columns with 25% and 50% corrosion damage up to 99% and 95%, respectively.

**Keywords:** composite jacket, joint, repair system, concrete structures, pile rehabilitation, FE simulation

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## INTRODUCTION

Retrofitting, strengthening, and repairing of aging infrastructures are becoming a major task and complex problem faced by engineers and researchers nowadays [1]. In most cases, it is more economical to rehabilitate these existing infrastructures than to replace and/or rebuild new ones. In year 2000 alone, the US State Department spent almost \$5 billion to remediate corroded RC bridges [1]. Similar cost was spent to maintain the highway bridges in Europe and Canada [2]. On the other hand, the corrosion damage costs the Australian economy more than \$13 billion per year [3]. Thus, engineers and researchers are continuously looking for a repair system that could offer a long-term repair solution and low-cost maintenance infrastructure.

The rehabilitation of existing structures is commonly performed using the same materials (wood, concrete, and steel) used in the original construction. This is not sensible since the structure will be subjected to the same forces and environmental conditions that caused its deterioration and the repair cycle will be unending [4]. In the last 20 years, fiber reinforced polymer (FRP) composite materials have become an extremely versatile option to strengthen or rehabilitate existing structures especially in harsh marine environment. This material has been a preferred option because of their superior properties such as high strength, corrosion resistance, lightweight, high fatigue resistance, nonmagnetic, high impact resistance, and durability [5].

Deterioration of reinforced concrete (RC) piles in marine environment is mainly due to corrosion of reinforcing bars. In this case, the large volume of corroded steel induces lateral forces in the surrounding concrete. When these lateral forces are greater than the tensile strength of the concrete, cracking and spalling of concrete would occur. Among the possible upgrading strategies, the use of FRP wraps or jackets to prevent this expansion through its induced lateral resistive forces, is gaining widespread acceptance [6]. Considerable research has been reported in the last decade on the mechanical behaviour and failure of FRP wrapped RC columns. These studies on FRP confinement of column structures include repairing and retrofitting of damaged columns using FRP system [7-10] and strengthening of columns using FRP composites [11-13].

The fabrication of externally bonded FRP confining system can be categorized into two methods, the wet lay-up and the pre-fabricated system. In the wet lay-up system, FRP composite jacket is prepared on site where trained workers are essential. This repair system requires a smooth surface and rounded corners of the column before the FRP wrapping is applied. Liquid resins are then prepared on-site which are then impregnated into the fiber and rolled evenly to ensure a balanced distribution and to remove the bubbles. While this repair system is ideal when immediate rehabilitation is necessary, the impregnation of the fiber is difficult to execute and monitor if done underwater. On another hand, prefabricated FRP composite jacket repair systems are produced in the manufacturing plants to achieve a high quality and uniform product. An example of this type of repair system is the FRP composite shells used to repair wood piles as reported by Lopez-Anido et al. [14]. The system requires a minimum of two FRP composite shells which are held together with circumferential metal straps. These metal straps were found usually damaged and severed which lead to opening of the composite shells and losing the effectiveness of the repair. Thus, the ability of the pre-fabricated composite repair system to provide structural continuity and confinement is always a concern. There is a need therefore to develop an innovative and effective jointing system for an FRP jacket which can provide continuity and confinement.

## RESEARCH SIGNIFICANCE

A new type of FRP jacket with an easy-fit and self-locking mechanical jointing system has been developed. However, there are still no reported studies demonstrating the application of this pre-fabricated composite repair system for pile rehabilitation. Investigation of the interlocking teeth and joint key of the FRP jacket including the characterisation of the mechanical properties of the composite materials used in the joint is therefore

important. Similarly, an assessment of the capacity and behaviour of the FRP jacket subject to internal pressure using Strand7 finite element (FE) software [15] and experimental verification is important to determine the viability of this new rehabilitation technique. Finally, the compression test of concrete columns repaired with an FRP jacket is critical to determine the efficiency and structural reliability of this new composite repair method.

## THE NOVEL COMPOSITE REPAIR SYSTEM

A new type of a pile repair system and concrete formwork jacket manufactured from fiber composite materials with an innovative mechanical joining system (Fig. 1 - left). The repair system is comprised of the (1) existing damaged structure, (2) prefabricated composite jacket, and (3) grout as infill to the annulus between the structure and the jacket. The novelty of this repair system is that it is quick and safe to install due to the easy-fit and self-locking mechanical jointing system [16]. This joint system involves two interlocked edges that quickly mesh together like the teeth of a zipper. A locking key is placed between the interlocking teeth and slid or levered only one pitch length into place, wedging the joint edges together with a uniformly loaded force along the entire length or circumference of the joint. This makes it more reliable than strapped/lap joints. This joint design was inspired by the way clams and other seashells attach themselves to rock ledges using anchors through hundreds of small filaments as depicted in Figure 1 (right). These filaments can produce a very strong hold when their strength is added together.

The repair system works by wrapping the prefabricated and flexible FRP jacket around the pile above or below the waterline and placing the joint key vertically along the seam to lock the jacket producing a cylindrical confinement. This simple assembly process can be carried out underwater or alternatively above waterline. The finished assembly is then lowered in the water up to the depth where repair is required. A standard hose is fitted at the bottom of the installed jacket to up-fill annulus with water displacing grout, resin or concrete as required. After grouting, the hose is removed and the installation is done. Aside from meeting the strength requirements, the composite jacket serves as a shield to prevent deterioration due to environmental attacks such as marine borers, chloride penetration and water waves.

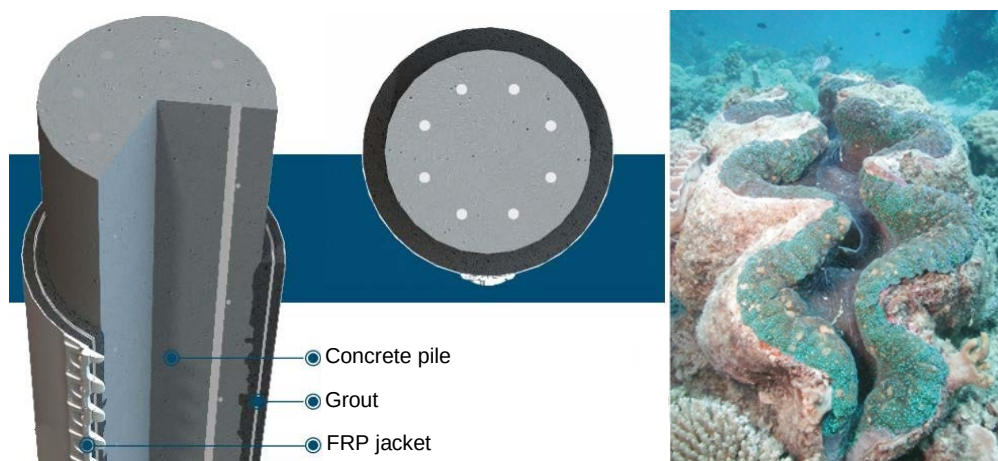


**Fig. 1** – The composite jacket repair system [16] (left) and the clam concept (right).

## CHARACTERISATION OF THE JOINT MATERIALS

One of the key components of the novel composite jacket is the innovative jointing system which locks the FRP jacket and provides the structural continuity of the repair system. This joint (locking key and interlocking teeth) is made up of high strength composite resin which is moulded to appropriate geometry. As the structural integrity of the repair system relies on the joint, an experimental investigation was conducted to determine the most appropriate material for the joint. Test of coupon specimens was performed following the ISO and ASTM standards [17-20] to evaluate the flexural, tensile, compressive, and shear properties of the considered joint materials. These properties were then utilised as input into the FE simulations on the behaviour of FRP jacket.

### Materials

Three types of thermoset composite resins are considered for the joint. These are the microfiber infused resin (MIR), high performance epoxy vinyl ester resin (EVE) and microfiber reinforced high performance vinyl ester resin (M-EVE). The sample panels were produced by the industry partner and cut according to sizes. Six specimens each were prepared and tested in order to obtain the strength and stiffness properties of the composite

resin. Each specimen was labelled with an identifying number in preparation for the test. Figure 2 shows the actual set-up for the characterization of the mechanical properties of the composite resins.

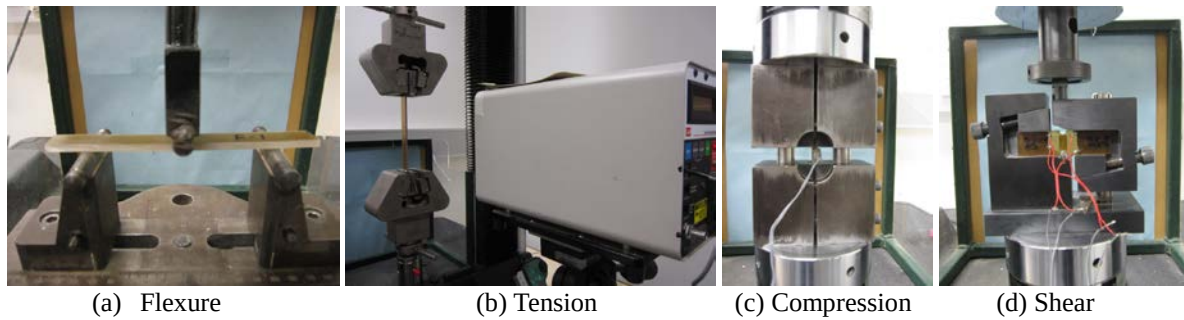


**Fig. 2** - Test set-up for the characterization of joint materials

### Mechanical properties of composite resins

The average mechanical properties and standard deviations of the different microfiber resins are summarized in Table 1. Based on the results, the variability of the modulus and strength values for all the tested specimens is less than 10%. It is also evident from the results that M-EVE resin has the best strength and stiffness properties among the 3 composite resins considered. The stiffness of M-EVE resin is more than 4 GPa (580,150 psi) while MIR and EVE have a stiffness of only 3.3 GPa (478,624 psi). The strength of M-EVE resin is also the highest. As an example, the flexural strength of M-EVE resin is around 110 MPa (15,954 psi) while MIR and EVE are only 67 MPa (9,717 psi) and 89 MPa (12,908 psi), respectively. These better properties of M-EVE resin are due to the introduction of microfibers in the vinyl ester resin which produces a composite with higher strength and stiffness. However, the addition of microfibers also results in a more brittle failure behaviour for M-EVE than MIR and EVE resins. These results can be clearly seen in Table 1 where M-EVE resin failed at the highest sustained stress but at the lowest strain. The results are consistent with the findings of Song and Zheng [21] where blending wheat gluten matrix with methylcellulose microfibers improves the tensile modulus of the composites. They also found out that the composites become more brittle as the volume of microfibers increases. Generally, the tensile strength of the resin with microfibers should increase [22] however it was not observed in the experiment. It is assumed that this result is due to the very short fibers used which did not contribute in the crack retardation of the composites and the strength behaviour is governed by the matrix. It is also observed that the shear and tensile strength are lower than their flexure and compressive strength properties which indicate that the composite resins are more susceptible to shear and/or tension type of failure when subjected to load.

**Table 1** - Mechanical properties of microfiber composite resins

| Test        | Property               | MIR               |                | EVE               |                | M-EVE             |                 |
|-------------|------------------------|-------------------|----------------|-------------------|----------------|-------------------|-----------------|
|             |                        | Average           | Std Dev        | Average           | Std Dev        | Average           | Std Dev         |
| Flexure     | Modulus, MPa (psi)     | 2724.83 (395,203) | 58.83 (8,532)  | 2770.50 (401,827) | 51.88 (7,524)  | 4330.71 (628,116) | 241.51 (35,028) |
|             | Peak Stress, MPa (psi) | 67.50 (9,790)     | 1.71 (248)     | 89.45 (12,974)    | 1.91 (277)     | 110.88 (16,082)   | 7.92 (1,149)    |
|             | Strain at Peak (%)     | 3.89              | 0.26           | 5.41              | 0.22           | 2.80              | 0.25            |
| Tensile     | Modulus, MPa (psi)     | 3379.00 (490083)  | 92.79 (13458)  | 3365.00 (488052)  | 127.99 (18563) | 4083.04 (592195)  | 47.75 (6926)    |
|             | Peak Stress, MPa (psi) | 44.78 (6495)      | 1.55 (225)     | 61.75 (8956)      | 0.60 (87)      | 60.02 (8705)      | 2.78 (403)      |
|             | Strain at Peak (%)     | 2.44              | 0.08           | 4.55              | 0.37           | 1.39              | 0.03            |
| Compression | Modulus, MPa (psi)     | 3145.00 (456144)  | 570.40 (82730) | 2927.00 (424525)  | 52.33 (7590)   | 4463.32 (647350)  | 162.60 (23583)  |
|             | Peak Stress, MPa (psi) | 81.59 (11834)     | 3.00 (435)     | 96.65 (14018)     | 3.93 (570)     | 116.80 (16940)    | 4.40 (638)      |
|             | Strain at Peak (%)     | 3.09              | 0.10           | 4.08              | 0.05           | 4.04              | 0.13            |
| Shear       | Modulus, MPa (psi)     | 930.13 (134904)   | 80.97 (11744)  | 908.72 (131799)   | 59.74 (8665)   | 1085.11 (157382)  | 59.11 (8573)    |
|             | Peak Stress, MPa (psi) | 35.65 (5171)      | 2.24 (325)     | 45.45 (6592)      | 2.04 (296)     | 42.26 (6129)      | 1.02 (148)      |
|             | Strain at Peak (%)     | 0.71              | 0.08           | 0.75              | 0.05           | 0.51              | 0.04            |

### BEHAVIOUR OF FRP JACKET UNDER INTERNAL PRESSURE

Finite element (FE) analysis was performed to investigate the overall behaviour and the failure mechanism of the FRP jacket when subjected to internal pressure. The application of internal pressure is done to simulate the radial expansion of the FRP jacket under the effect of confining pressure in a repaired column under axial compression. Strand7 finite element program [8] was used in the simulation works.

#### Description of the FE model

The FE simulation on the behaviour of the FRP jacket was performed using a 2-pitch model as shown in Figure 3 (left). The diameter of the jacket is 650 mm (25.6 inch) with a height of 400 mm (15.7 inch). The jacket is made up of 3.0 mm (0.12 inch) thick triaxial E-glass fiber-polyester composites and has an effective Young's Modulus of 29 GPa (4206094 psi). Similarly, the 0.5 mm (0.02 inch) thick SG 230 HV Methacrylate epoxy adhesive with a Young's Modulus of 0.55 GPa (79770 psi) between the FRP jacket and joint is included in the development of the FE model.

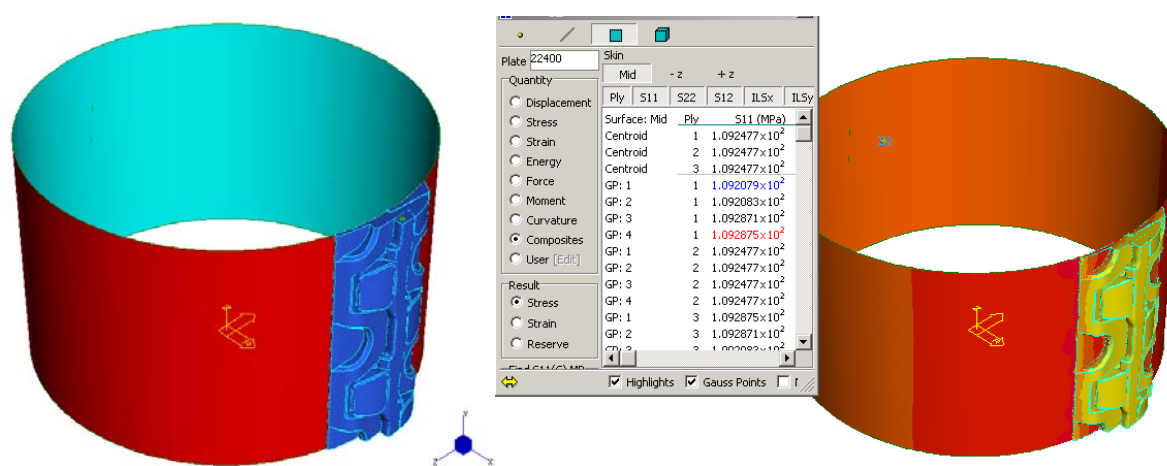


**Fig. 3** - FEM model of FRP jacket (left) and circumferential stress (S11) at 1.0 MPa (145 psi) internal pressure (right)

Three FE models with different joint and joint key materials were analysed. Models A, B and C represent the FRP jacket with MIR, EVE and M-EVE resins for the joint, respectively. Due to the complexity of the repair system geometry, the model was created first using the Solidworks and was converted using the Solid Automeshing tools in Strand7 to automatically generate high quality finite element meshes. In all FE models, the joint and joint key, and epoxy adhesive are modelled using Tetra4 brick elements while the FRP jacket is modelled using Quad4 plate elements. Similarly, an internal pressure of 1 MPa (145 psi) is applied on the interior surface of the jacket elements. The top and bottom of the model are restrained to move up and down (fixed at DZ) and to rotate about the radial and tangential direction (RR and RT). The jacket is allowed to move in the radial direction only. These restraint conditions are adapted in all the FE models.

Analysis was conducted using the linear static solver in Strand7. The prediction of failure of the jacket was performed by automatically load stepping the applied internal pressure of 1 MPa (145 psi). The level of stresses in each component of the jacket was then checked at each load step and compared with the strength properties of materials established from test of coupons. In this study, the jacket is considered to fail when the allowable stress of the joint materials, summarized in Table 1, is/are exceeded. The possible failure mechanisms of the FRP jacket were also determined. Similarly, the radial deformation was recorded and analysed to determine the radial expansion of FRP jacket due to confining effect.

#### Evaluation of stress level in FRP jacket

The stress level in the FRP jacket at an applied internal pressure of 1 MPa (145 psi) was theoretically evaluated and compared with the results of the FE simulation. Thin-walled cylinder formula was used to calculate the circumferential/hoop stress in the jacket and to validate the applicability of the developed FE model in predicting the behavior of FRP jacket. The assumptions in the theory are applicable since the thickness of the jacket is less than one over two hundredth (1/200) of its internal diameter. The circumferential stress in FRP jacket is calculated using Equation 1.

$$\sigma_h = \frac{pD}{2t} \quad (1)$$

where  $p$  is the internal pressure,  $D$  is the internal diameter of the jacket, and  $t$  is the thickness of the FRP jacket. The calculated hoop stress at an applied internal pressure of 1.0 MPa (145 psi) is 108.33 MPa (15712 psi). This is comparable with the hoop stress in the FRP jacket of 109.2 MPa (15838 psi) determined from the FE simulation (Fig. 3 - right).

#### Failure behaviour of FRP jacket at varying internal pressure

The level of stresses in the interlocking teeth and key of the joint is analysed at each load step and compared with the strength properties of materials to determine the level of applied internal pressure where failure will occur in the FRP jacket. The stress profile of the interlocking teeth and locking key for Model C is shown in Figure 4. The results of the FE simulation indicated that high stress levels in the FRP jacket can be found in the interlocking teeth and locking key of the joint. The results also showed that the axial tensile stress in the interlocking teeth and the shear stress in the locking key will exceed the strength of the composite resin at an applied internal pressure of 14 MPa (2030 psi) as shown in Figures 4a and 4b, respectively. On the other hand, the bending stress in the locking key will exceed the strength of the composite resin at an applied pressure of 20 MPa (2900 psi). In all FE models, debonding failure of the adhesive layer will occur at an applied internal pressure of 22 MPa (3190 psi). Based on this level of stresses, it anticipated that the failure of the composite jacket will more likely occur due to the tensile failure of the interlocking teeth or shear failure of the locking key.

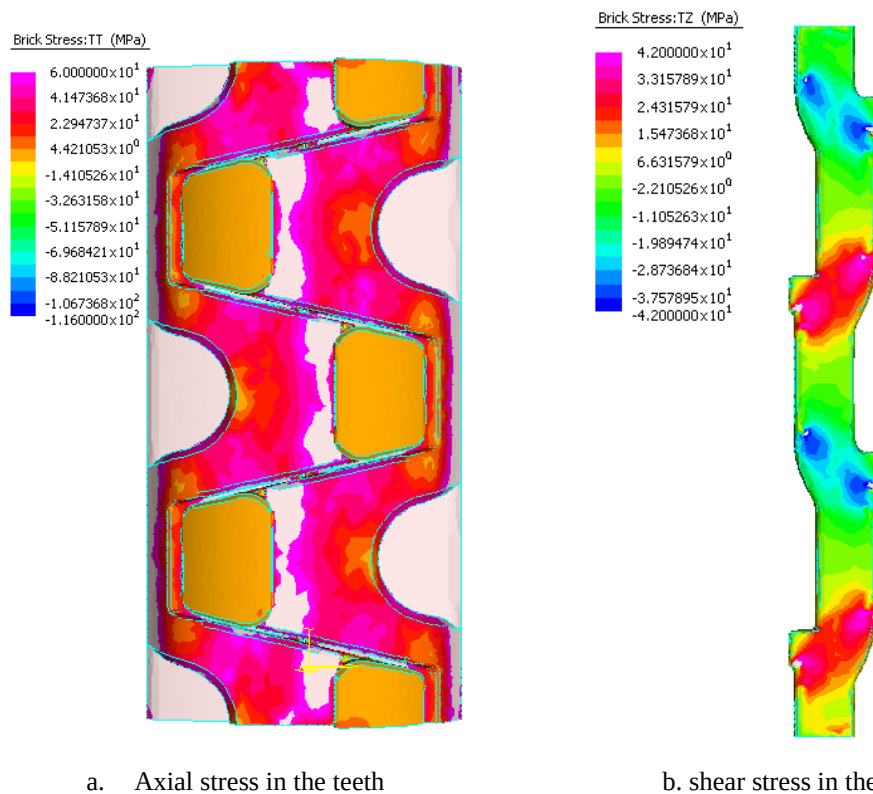


Fig. 4 - Stress profile in the teeth and key in Model C at an applied internal pressure of 14 MPa

Table 3 summarizes the failure mechanism and the level of internal pressure that will induce this type of failure. The results of the FE simulation indicate that the locking key will most likely fail due to shear failure. This type of failure will occur at an applied pressure of 4 MPa (580 psi), 6 (870 psi) and 14 MPa (2030 psi) for Models A, B and C, respectively. This finding is concurrent with the results of the test of coupons, wherein the three types of composite resin exhibited low shear strength and tensile strength. Interestingly, Model C or the FRP jacket with joint made from M-EVE resin will concurrently fail by shearing of the locking key and tensile failure of the interlocking teeth. Based on these results, it can be concluded that the interlocking teeth and locking joint made from microfiber reinforced high performance vinyl ester resin will provide the best structural continuity for the FRP jacket and a high confining pressure to the repaired column. The results also indicated that no edge debonding will occur on the epoxy adhesive when failure of the joint occurs indicating that a reliable structural bond between the joint and the jacket can be achieved using SG230 HV Methacrylate adhesive.