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Modeling of FRP Strengthening Techniques in Concrete Infrastructure

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Modeling of FRP Strengthening Techniques in Concrete Infrastructure

Editor:
Riadh Al-Mahaidi



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Preface

Modeling of FRP Strengthening Techniques in Concrete Infrastructure

Fibre reinforced polymers (FRP) have become an effective solution to retrofit and reinforce existing and new concrete and masonry structures. This has taken the form of externally bonded reinforcement in existing structures and internally installed reinforcement to replace steel bars in new structures; the latter particularly suited to structures exposed to corrosive environments. Over the last two decades, the use of FRP materials in civil infrastructure has become increasingly dominant worldwide due to the unique properties they possess such as high strength to weight ratio and excellent corrosion resistance. In Australia, the use of externally bonded FRP reinforcement has found wide application in recent years. A good example of this is the retrofitting of the concrete approach viaducts of the West Gate Bridge in Melbourne. When completed by the end of 2010, it will be the largest FRP retrofitting project in the world.

This special publication of the ACI contains eight papers on the Modeling of FRP Strengthening Techniques in Concrete Infrastructure. They were presented at a special session of the ACI Fall Convention, October 2011 in Cincinnati, Ohio. They cover a wide range of innovative contributions from researchers in the United States, Australia, Canada and United Arab Emirates.

The first paper in this issue is by Chen and El-Hacha and presents a finite element investigation into ultra-high performance concrete beams under flexural loading. Next in the series is a paper by Oudah and El-Hacha which deals with numerical simulations of RC beams strengthened with prestressed near-surface mounted CFRP subjected to quasi-static loading. The structural response of RC girders strengthened with FRP composites in shear is investigated by You et al in the third paper using numerical simulations. The next paper by Moslehy et al focusses on the development of new constitutive relations for FRP-strengthened RC elements subjected to pure shear for implementation into finite element models. The fifth paper written by Hawileh et al uses a finite element model to analyze the cyclic response of RC beams strengthened in shear with both externally bonded and near-surface mounted FRP. The response of RC beams strengthened with externally bonded FRP under elevated temperatures is investigated numerically in the sixth paper by Naser et al. This is followed by the seventh paper by Ahmed et al which investigates a numerical procedure to generate interaction diagrams for circular columns wrapped with FRP. The last paper by Kalfat and Al-Mahaidi covers numerical simulations of bi-directional fiber patch anchors used to enhance the strength of FRP-to-concrete joints.

As the editor of this special issue, I would like to thank all the authors who contributed to this special issue I would like to take this opportunity to thank the authors for all their efforts in preparing these papers and to thank the reviewers for their valuable contribution to ensure that a high standard is maintained.

Riadh Al-Mahaidi
Editor

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INVESTIGATION OF HYBRID FRP-UHPC BEAMS UNDER FLEXURE WITH FINITE ELEMENT METHOD

Donna Chen and Raafat El-Hacha

Synopsis: This paper explores the investigation using finite element methods of experimentally tested hybrid FRP-UHPC (Ultra-High Performance Concrete) beams under flexural loading. A combination of mesh sensitivity and cohesive element parameter studies were performed through validation with experimental data. Good correlation was found between experimental findings and the finite element method, though higher stiffness was found in the latter case. It was found that an overall mesh size of 12.5 mm (0.50 in) was suitable for use in the model in order to allow for proper convergence. For the parameters at the GFRP-UHPC interface, it was found that a bond-slip ratio of 5 along with a bond strength of 5 MPa (0.725 ksi) were the best fit to experimental data and should be used in future studies. Additional investigation into the incorporation of considerations to allow for more damage accumulation in the finite element model was recommended.

Keywords: Bond, hybrid, fiber reinforced polymer, ultra-high performance concrete, bond, cohesive element, mesh sensitivity

ACI Member, **Raafat El-Hacha**, is an Associate Professor in the Department of Civil Engineering at the University of Calgary, Canada. He is a member of ACI Committee 440 “Fiber Reinforced Polymer Reinforcement” and chair of Sub-Committee 440-I “FRP Prestressed Concrete”. He is an associate member of ACI 437 “Strength Evaluation of Concrete Structures”. His research interests include strengthening concrete/steel structures and reinforcing/prestressing concrete structures with FRPs, and use of high strength materials in new construction.

Donna Chen is a Ph.D. student in the Department of Civil Engineering at the University of Calgary. She received her M.Sc. degree at the University of Calgary, Calgary, Alberta in 2011 and her B.Sc. degree at Queen’s University, Kingston, Ontario in 2007. Her research focuses on the performance of hybrid structural members using high performance materials, such as FRPs and ultra-high performance concrete, in bridge applications.

INTRODUCTION

The extension of research on Fiber Reinforced Polymers (FRPs) from the aerospace industry to civil engineering applications began approximately thirty years ago and since then, many findings and developments have been made. There are mainly three core streams for the use of FRP materials in structural applications, either for new construction, strengthening or rehabilitating structures. More specifically, FRP materials have been predominantly studied for the purpose of column confinement or to serve as internal or external reinforcements in flexural members or whole structural shapes. In this particular research program, FRP materials in the form of pultruded shapes as well as uni-directional sheets were used.

The use of FRP structural shapes as a primary load carrying structural element has both advantages and disadvantages. Due to its high strength-to-weight ratio, the resulting structural member has significant reductions in self-weight as well as overall dimensions. Nevertheless, structural shapes, which are typically made from Glass FRP (GFRP) have relatively low stiffness compared with conventional materials such as reinforced concrete and structural steel. This shifts the type of design methodology used from strength-oriented approaches to those that focus on serviceability requirements. In order to modify and enhance the stiffness of FRP structural members, several designs have been proposed by various researchers that utilize a combination of different materials in conjunction with the main GFRP structural shape.

Deskovic et al. [1, 2] investigated the design of a hybrid FRP-concrete beam that incorporated a thin-walled GFRP box section, a layer of concrete overtop and a thin sheet of Carbon FRP (CFRP) bonded on the tension side. The main design objectives were to increase the stiffness of the overall section, through the addition of both the concrete and CFRP, as well as to introduce an element of pseudo-ductility, where the failure strain of the CFRP material was chosen to be less than that for the GFRP material, therefore providing an advanced warning sign prior to failure of the overall section. Experimental testing under flexural loading for both static and fatigue loading conditions did exhibit fracture of the CFRP sheet prior to concrete crushing at ultimate failure. Stiffness was significantly increased in the hybrid beam, as compared with the GFRP box section alone. Under fatigue loading, it was found that performance was influenced by the load level applied, where sustained or alternating loads would result in a reduced effective stiffness over time.

Kitane et al. [3] studied the performance of a hybrid FRP-concrete bridge structure composed of three adjacent trapezoidal GFRP thin-wall shapes, bonded together as well as wrapped using FRP laminates. A thin layer of concrete was also cast directly below the top flange of the cross-section. Flexural testing under a combination of static and fatigue loading conditions showed that the addition of concrete did increase the stiffness of the hybrid structural member, while at the same time decreasing the amount of GFRP required in the cross-section. At ultimate failure, complete collapse did not occur and the following symptoms were noted: concrete crushing, compressive failure of the GFRP compressive flange, web buckling as well as delamination in the GFRP.

Cheng and Karbhari [4] examined the performance of steel-free hybrid bridge deck systems, which consisted of a FRP composite deck panel with alternating layers of uni-directional carbon and E-glass fibres. A combination of epoxy bonded carbon fibre rectangular stiffeners along with sand-epoxy paste shear ribs were also incorporated into the design to ensure composite action between the FRP deck panel and the cast-in-place concrete layer above. Flexural load testing of five deck specimens showed that the use of closely spaced stiffeners increased the load carrying capacity of the structural system whereas the presence of rib spacing did not have a significant impact on the performance of the composite deck panel.

Honickman and Fam [5] studied various configuration of hybrid girders that consisted of a trapezoidal pultruded

GFRP sheet pile sections and cast-in-place concrete. The concrete was cast within the interior of the GFRP sheet pile sections, which acted as stay-in-place formwork; in some specimens, a void was left in the bottom tension zone of the member where concrete would have negligible strength. Different bonding techniques between the GFRP and the concrete were examined, including the use of wet adhesives, bonded coarse aggregates and threaded steel rods. The hybrid girders were subjected to both positive and negative moment loading and it was found that interface debonding was the primary cause of ultimate failure, though more gradual failure did occur when both wet adhesives and shear rods were used at the bond interface.

Alongside with experimental investigation, research focus and effort has also been spent on the development of analytical and finite element models in order to better understand and predict the behavior of hybrid systems. Due to the importance of composite action and bond behavior in hybrid systems, a great deal of attention has been directed towards modeling of bond interfaces. Matzenmiller et al. [6] developed a constitutive model to describe the behavior of anisotropic FRP materials when undergoing damage. Li et al. [7] adopted the cohesive zone approach to predict mixed-mode bond behavior of adhesively bonded interfaces using combined results from previous Mode I and Mode II damage parameters. To ensure solution convergence, Gao and Bower [8] proposed the use of small viscosity in the cohesive interface parameters. Similarly, to facilitate model convergence, Diehl [9] investigated the incorporation of penalty stiffness in the cohesive zone method in order to create a model solely dependent on the critical energy release rate of the bond interface.

The investigation presented in this paper will discuss the development of a finite element model based on previous experimental testing performed on hybrid FRP-UHPC beams. Validation of the finite element model with experimental data will be performed with special attention to the bond interface parameters as well as a mesh sensitivity study.

RESEARCH SIGNIFICANCE

The validation, using the finite element method, of experimental results obtained from hybrid FRP-UHPC structural members, can be used towards future parametric studies and design optimizations, without the need for further experimental testing. The incorporation of contact elements allows for detailed examination and modeling of the behavior at material interfaces, which is a critical area of concern in hybrid structural members due to the occurrence of debonding.

FINITE ELEMENT MODEL ANALYSIS

Introduction

A 3-D finite element model of the hybrid FRP-UHPC beam tested experimentally was developed using ABAQUS 6.9. The cross-sections of the two types of tested hybrid beams are shown in **Fig. 1**. Due to the symmetry of the beam as well as the loading configuration, only one-fourth of the actual beam was modeled. A diagram of the completed quarter-beam model is shown in **Fig. 2**. The objective for the development of the finite element was to allow for parametric and optimization studies to be performed in the future. Therefore, mesh sensitivity analysis and validation with experimental results must first be confirmed.

Material Properties

The hybrid FRP-UHPC beam can be divided into distinct components. These include: the GFRP hollow box section beam, the UHPC layer, the CFRP/Steel FRP (SFRP) sheet, the steel plates at the loading points and the end supports and the epoxy adhesives used at the GFRP-UHPC and GFRP-CFRP or GFRP-SFRP interfaces. The material properties used to depict the behavior of the individual components were derived from the results of compression cylinder tests and tension coupon tests performed in accordance with ASTM standards [10, 12] along with comparisons with published manufacturer's specifications.

The GFRP hollow box section beam used in the hybrid beam was fabricated using the pultrusion process. The thin-walled beam has flange and web thicknesses equal to 11.1 mm (0.44 in), with an outer height and width of 228.6 mm (9 in) and 152.4 mm (6 in), respectively [13]. The GFRP hollow box section beam was modeled using 8-node shell continuum elements (SC8R). The GFRP hollow box beam was also modeled with the Hashin damage initiation criteria, with the value for the α coefficient taken equal to 1.0 [14]. The UHPC layer cast-in-place overtop of the GFRP hollow box section beam is an ultra-high performance material that behaves in a nearly linear-elastic manner up until ultimate compressive failure [15]. It was modeled with solid continuum elements (C3D8). The CFRP/SFRP sheet bonded to the bottom face of the GFRP hollow box section beam is an anisotropic material that behaves in a linear-elastic manner; they were modelled using SC8R

elements. The steel plates used at the loading points and end supports were made from structural steel. In the developed finite element model, the steel plates were modeled with solid continuum elements (C3D8). It behaves linear-elastically up until the yield point after which it exhibits large amounts of plasticity in the yield plateau prior to ultimate failure. For the purpose of this FE investigation, the plasticity of the steel plates was ignored and it was assumed that the steel plates would behave only linear-elastically; this is due to the fact that the applied load will not result in stress and strain levels in the steel greater than the yield point.

The anisotropic linear-elastic material properties used are given in **Table 1** [13]. The isotropic material properties for the UHPC, CFRP, SFRP and steel are shown in **Table 2** [16-19]. The epoxy adhesive used for bonding of the coarse silica sand aggregates in the bond interface system has the following properties: tensile strength of 48 MPa, tensile elongation limit of 1.9% and a modulus of elasticity equal to 3726 MPa when fully cured [20].

Modeling of Interface Interactions

For the majority of the model, merging of the meshes between the different material components/instances, which would allow for perfect bond to be provided, was not performed. This excludes the contact areas between the steel plates and the UHPC layer at the top of the hybrid beam and the steel plates and the CFRP/SFRP sheet at the bottom of the beam, where separation was judged to be unlikely and therefore perfect bonding was assumed. In order to correctly depict the contact behavior at bond interfaces, a thin layer of cohesive elements was used in conjunction with contact interactions to prevent element penetration, at both the GFRP-UHPC (top) and GFRP-CFRP or GFRP-SFRP (bottom) bond interfaces.

The mesh size used for the cohesive element layers was 0.25 mm (0.01 in), which is 4 times smaller than the CFRP sheet thickness (provided as equal to 1.0 mm to approximate zero-thickness behavior) and a minimum of 25 times smaller than typical mesh size of the GFRP flanges; this criterion was found to be within the recommended mesh ratio for achieving adequate convergence of the whole model [9]. The cohesive elements were modeled using the traction separation method. Previous testing performed on the bond interface under pure shear loading provided preliminary values for modeling of the cohesive elements [21]. However, it should be noted that these values are not completely representative of the actual bond behavior experienced in the hybrid beam due to the fact that the bond interface experiences mixed-mode damage when subjected to flexural loading. The main components for the modeling of cohesive elements using the traction separation method are: the fracture energy (G_c), the bond stress (σ) and the bond slip ratio (δ_{ratio}) based on the bond slip experienced at failure (δ_f) divided by the bond slip experienced at maximum stress (δ_o). A parametric study to determine the best approximations for the cohesive element traction separation damage values was performed and will be described in this paper.

Experimental Program and Main Results

One of each type of hybrid FRP-UHPC beam was tested experimentally under four-point flexural loading. The 2900 mm (114 in) long beam was subjected to static loading up until ultimate failure using displacement control mode at an increment of 1 mm/min (0.04 in/mm). The load set-up is shown in **Fig. 3**. A full discussion of the experimental results was provided in other publications [22-24] and the purpose of this paper is not to reintroduce those findings. The main intention is to use the experimental data to validate the results obtained from the finite element model analysis. Therefore, only the results required for comparison with the finite element models will be presented in this paper.

The primary modes of failure experienced by the hybrid beams occurred in the GFRP hollow box beam. They included: web shearing in the shear span, web buckling in the flexural span and longitudinal rupture at the flange-web interface. At ultimate failure of the hybrid CFRP-GFRP-UHPC beam, the maximum load applied was 341 kN (76.66 kip), with an associated maximum mid-span deflection of 48.9 mm (1.93 in). For the hybrid SFRP-GFRP-UHPC beam, the maximum load and deflections at failure were 383 kN (86.10 kip) and 50.5 mm (2.0 in).

Predicted Flexural Strength using Analytical Methods

In order to determine the expected flexural capacity of the hybrid FRP-UHPC beam, the following basic principles and assumptions were used:

- All materials used in the cross-section behaved linear-elastically until failure;
- Plane sections remained plane before and after bending through the cross-section until failure; and,

- Composite cross-sections can be described using an equivalent transformed cross-section based on the relative second moment of inertia of the individual material components.

The predicted capacity of the two hybrid beams is shown in **Table 3**.

Finite Element Analysis Test Matrix

Three test series with different variable parameters were investigated using the experimental data collected from the hybrid CFRP-GFRP-UHPC beam. The first series investigated the influence of varying bond slip ratios. The second series investigated the mesh sensitivity of the model at the two extreme values of bond ratios examined. Lastly, the third series investigated the bond strength parameter that best fit the experimental behavior of the hybrid beam using the previous parameters determined (bond ratio and mesh size) in the first two test series. The test matrix and their associated test IDs are presented in **Table 4**.

A final validation of the completed model was performed through comparisons between the finite element solution and the experimental behavior of the hybrid SFRP-GFRP-UHPC beam. This was to ensure the applicability of the developed model to different structural configurations and material properties.

COMPARISON OF RESULTS AND DISCUSSION

Series A – Determining the correct bond slip ratio

In this first test series, finite element models using bond slip ratios values within the range of 2 – 5 were compared with the experimental data. As can be seen from the load-deflection curves, shown in **Fig. 4**, all of the curves exhibited the same initial behavior, which did not deviate until a mid-span deflection of approximately 40 mm (1.57 in) was surpassed. After that point, it is evident that Test A1 showed the least curvature in the load-deflection curve, as compared with Test A4, which showed the greatest amount of non-linearity. Since experimental testing did not suggest the presence of significant debonding, a bond ratio of 5 was chosen as the most probable value for use in the finite element model.

Series B – Determining the mesh sensitivity

For this second test series, two additional mesh sizes were examined using the cohesive element parameters determined from the first two test series. The goal was to find the largest permissible mesh size that would allow for convergence of the finite element solution in order to reduce the computational time required. In total, the three mesh sizes investigated were 6.25, 12.5 and 25.0 mm (0.25, 0.5 and 1.0 in), tested with bond slip ratio values of both 2 and 5, to confirm that the behavior is the same for all bond ratio values irrespective of mesh size. The mid-span load-deflection curves are shown in **Fig. 5** and **Fig. 6**. The load-deflection curves using mesh sizes 6.25 and 12.5 mm (0.25 and 0.5 in), for both bond slip ratio, were very nearly identical, with the curves obtained using a mesh size 25.0 mm (1.0 in) demonstrating significantly different behavior. This finding indicates that convergence of the finite element model can be achieved using an overall model mesh size of 12.5mm (0.5in).

Series C – Determining the correct bond strength value

From the findings obtained from the three test series performed, it was ascertained that:

- The bond slip ratio should be equal to 5; and,
- The optimized mesh size for model convergence as well as reduced computational time is 12.5 mm (0.5 in).

A final, third test series using the above parameters were used in order to determine more precisely the actual strength of the bond interface between the GFRP and the UHPC layer.

Three different parameters sets were investigated in the finite element model, which tested a bond strength range between 4 – 6 MPa (0.58 – 0.87 ksi). The load-deflection curves for the third test series is shown in **Fig. 7**. Tests C1, C2 and C3 were seen to reach maximum load carrying capacity within the deflection-control testing parameters of the finite element model. Modeling the interface with a bond strength equal to 4 MPa (0.58 ksi) in Test C1 resulted in a maximum applied load of 358.9 kN (80.68 kip) with a mid-span deflection of 52.8 mm (2.08 in). Similarly, the peak load reached in Test C2 was 386.8 kN (86.96 kip) at a mid-span deflection of 57.0 mm (2.24 in); for Test C3, the peak condition occurred at a mid-span deflection of 60.6 mm (2.39 in) with an applied load of 413.4 kN (92.94 kip). In comparison, at a mid-span deflection of 62.8 mm, which was the upper

deflection limit used in the finite element model, the applied load carried by the hybrid beam were 304.1 kN (68.36 kip), 366.9 kN (82.48 kip), 411.0 kN (92.40 kip), 438.1 kN (98.49 kip) and 454.3 kN (102.13 kip) for Tests C1, C2, C3, C4 and B2, respectively.

Though, from first observations, it would appear that Test C1 most closely approximated the behavior exhibited during experimental testing, it should be carefully noted that ultimate failure of the hybrid beam did not occur due to interface debonding but rather due to web buckling as well as shear and flexural cracking. Therefore, the curve that most reflects the actual behavior of the hybrid beam would demonstrate a flexural capacity close to the theoretical value of 416 kN (93.52 kip). Since significant bond slippage was not observed in the hybrid beam during experimental testing, judging from the tests results in **Fig. 6**, it was found that Test C3, with a bond interface strength of 5 MPa (0.73 ksi), most closely represented the behavior of the beam. The behavior of the hybrid beam exhibited in Test C3 allows for adequate softening in the load-deflection curve while providing a maximum capacity that exceeds the load achieved during experimental testing, which still approximates the predicted flexural strength obtained through beam theory. This ensures that the FE model would not predict debonding failure prior to failure due to shear or flexure.

Validation with Experimental Data from Hybrid SFRP-GFRP-UHPC Beam

The developed model that was validated with the experimental data from the hybrid CFRP-GFRP-UHPC beam was modified in order to reflect the slightly wider UHPC flange (as shown in **Fig. 1**) as well as the different material property of the SFRP sheet. No variables other than those listed above were changed. The finite element model results, in comparisons with the experimental behavior, for both types of hybrid beams are shown in **Fig. 8**. Comparison between the experimental data and the finite element solution for the hybrid SFRP-GFRP-UHPC beam shows that the flexural response of the finite element solution was stiffer than the actual experimental behavior, to a greater degree than for the hybrid CFRP-GFRP-UHPC beam. The percent difference in the comparison for both types of hybrid beams is provided in **Table 5**.

FURTHER RESEARCH

In all of the tests performed using the developed finite element model, it can be noted that the modeled behavior showed greater stiffness than the actual experimental data near the end of the analysis. Though greater stiffness is to be expected in finite element solutions, particularly when using full integration methods, the noticeable difference in the slopes of the load-deflections curves suggest that further investigation will be required to incorporate additional softening into the finite element model. This would include investigations into the use of different damage models and material parameters. Additionally, the effect of mixed-mode behavior in the cohesive layer was not incorporated in this study and will be featured in future revisions.

CONCLUSIONS

The objective for the investigation described in this paper was to develop and validate a finite element model with experimental data obtained during testing of a specific hybrid beam. The mesh sensitivity study performed showed that a maximum overall mesh size of 12.5 mm (0.50 in) was acceptable for use in order to allow for adequate solution convergence. Further investigation into the cohesive element parameters indicated that, for the hybrid beam under examination, a bond slip ratio of 5 along with a bond strength of 5 MPa (0.73 ksi) should be used in further parametric studies. Additionally, further care and study should be taken to increase the damage accumulated in the model, in the effort to reduce the overall stiffness of the finite element model at higher load and deflection levels.

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Table 1 – GFRP material properties [13]

Material Property	Value	
	MPa	ksi
Longitudinal modulus	17200	2495
Transverse modulus	6890	999
Longitudinal tensile strength	207	30
Longitudinal compressive strength	207	30
Transverse tensile strength	48.3	7
Transverse compressive strength	103	14.9
Longitudinal shear strength	31	4.5
Transverse shear strength	31	4.5
Poisson's ratio	0.33	

Table 2 – Material properties for UHPC, CFRP, SFRP and steel

Material	Young's modulus		Poisson's ratio	Reference
	MPa	ksi		
UHPC	50000	7252	0.2	[16]
CFRP	95800	13895	0.2	[17]
SFRP	66100	9587	0.3	[18, 19]
Steel	200000	29008	0.3	

Table 3 – Predicted flexural capacity of hybrid beams

Beam	Flexural resistance		Maximum applied load under four-point loading		Mid-span deflection at maximum capacity	
	kN.m	lb.ft	kN	kip	mm	in
CFRP-GFRP-UHPC	239	176277	416	93.52	41.8	1.65
SFRP-GFRP-UHPC	279	205780	486	109.26	47.6	1.87

Table 4 –Finite element test matrix for hybrid CFRP-GFRP-UHPC beam

Test ID	G_c J/m ²	σ MPa	K_{eff} MPa/mm	δ_o mm	δ_f mm	δ_{ratio} mm
Series 1 – Mesh size = 25.0 mm (1.0 in)						
A1	480	6	75	0.08	0.16	2
A2	720	6	75	0.08	0.24	3
A3	960	6	75	0.08	0.32	4
A4	1200	6	75	0.08	0.4	5
Series 2A – Mesh size = 12.5 mm (0.50 in)						
B1	480	6	75	0.08	0.16	2
B2	1200	6	75	0.08	0.4	5
Series 2B – Mesh size = 6.25 mm (0.25 in)						
B3	480	6	75	0.08	0.16	2
B4	1200	6	75	0.08	0.4	5
Series 3 – Mesh size = 12.5 mm (0.50 in)						
C1	800	4	50	0.08	0.4	5
C2	900	4.5	56.25	0.08	0.4	5
C3	1000	5	62.5	0.08	0.4	5
C4	1100	5.5	68.75	0.08	0.4	5

* 1 Ly = 41840 J/m², 1 in. = 25.4 mm, 1 kip = 6.89 MPa

Table 5 – Comparison between experimental results and finite element solution

Beam	Applied load at 49 mm (1.93 in.) mid-span deflection				Percent difference (%)
	Experimental		Finite element		
	kN	kip	kN	kip	
CFRP-GFRP-UHPC	341	76.66	371	83.40	8.8
SFRP-GFRP-UHPC	375	84.30	422	94.87	12.5

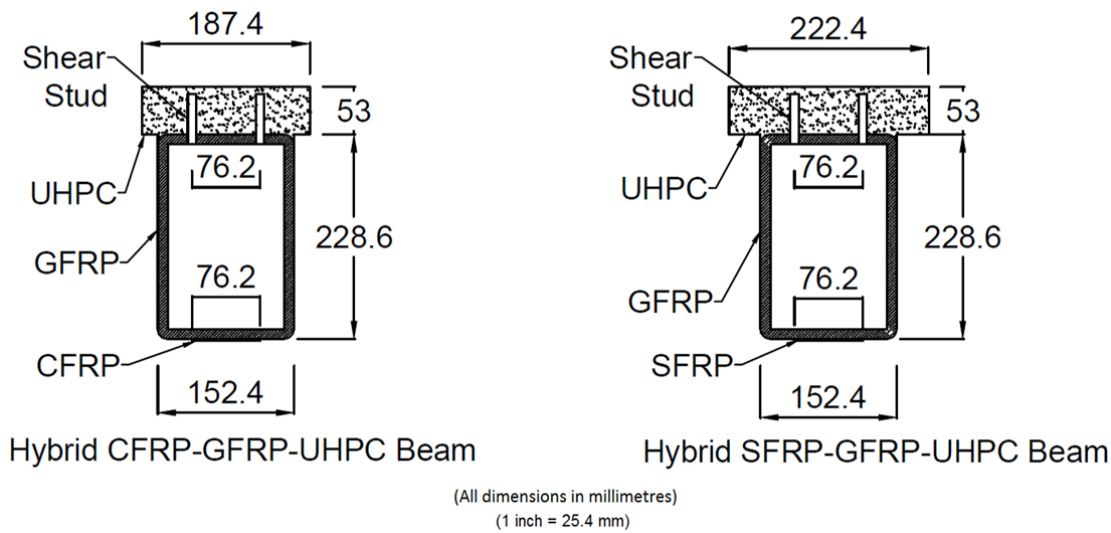


Fig. 1 – Cross-section of hybrid beam (Chen, 2011)

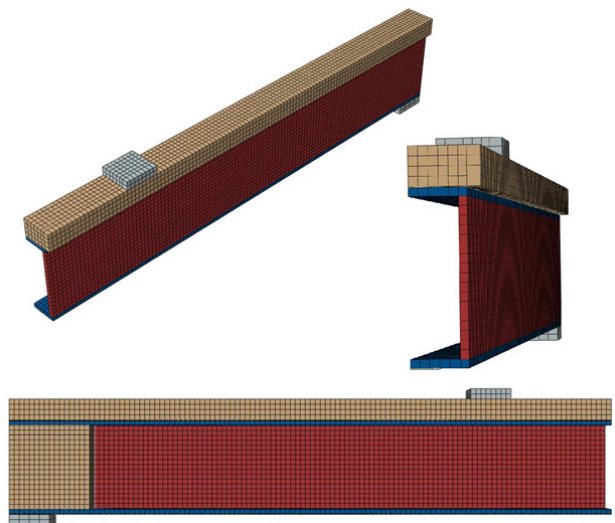


Fig 2. – Mesh arrangement of completed quarter-beam finite element model

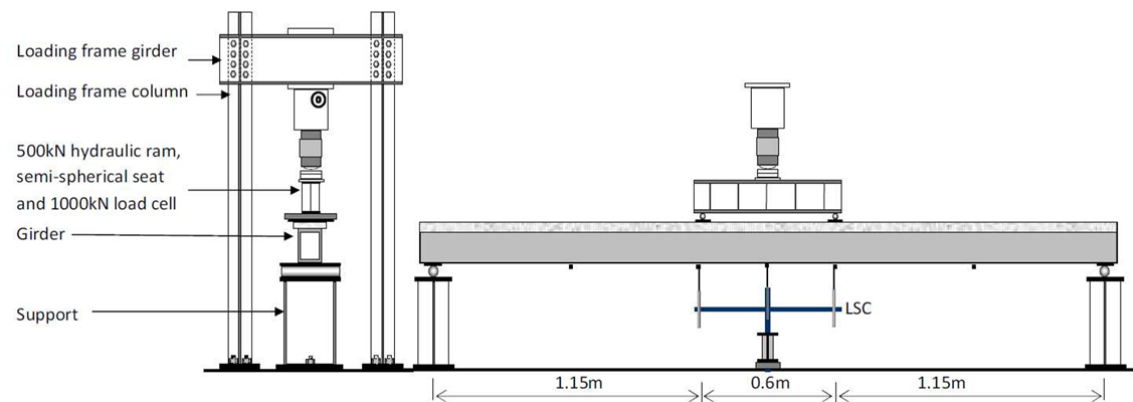


Fig. 3 – Experimental test set-up (Chen, 2011)

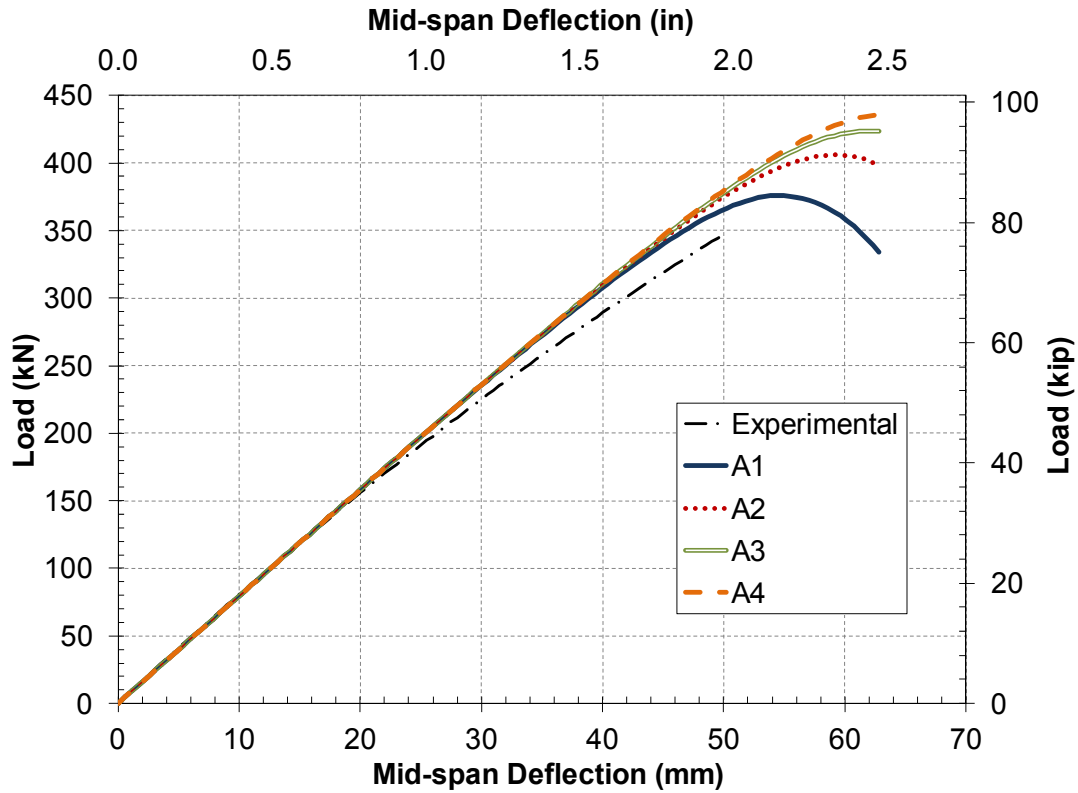


Fig. 4 – Comparison of finite element solutions with various bond slip ratios

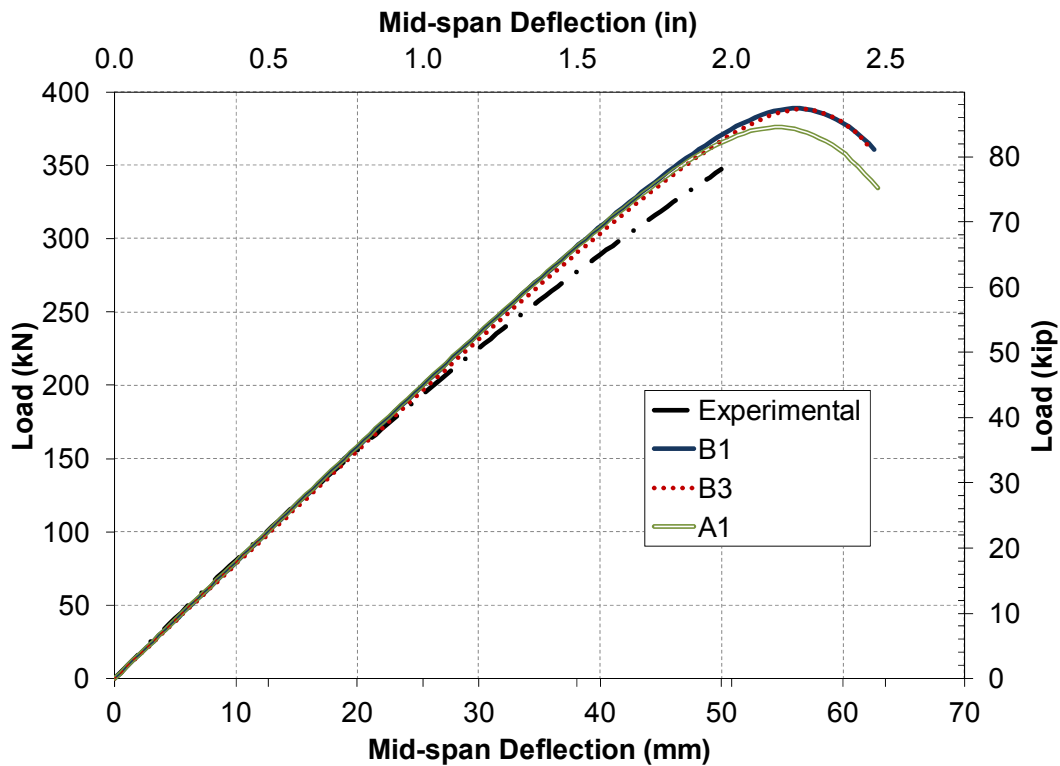


Fig. 5 – Mesh sensitivity analysis using different mesh sizes with a bond ratio of 2

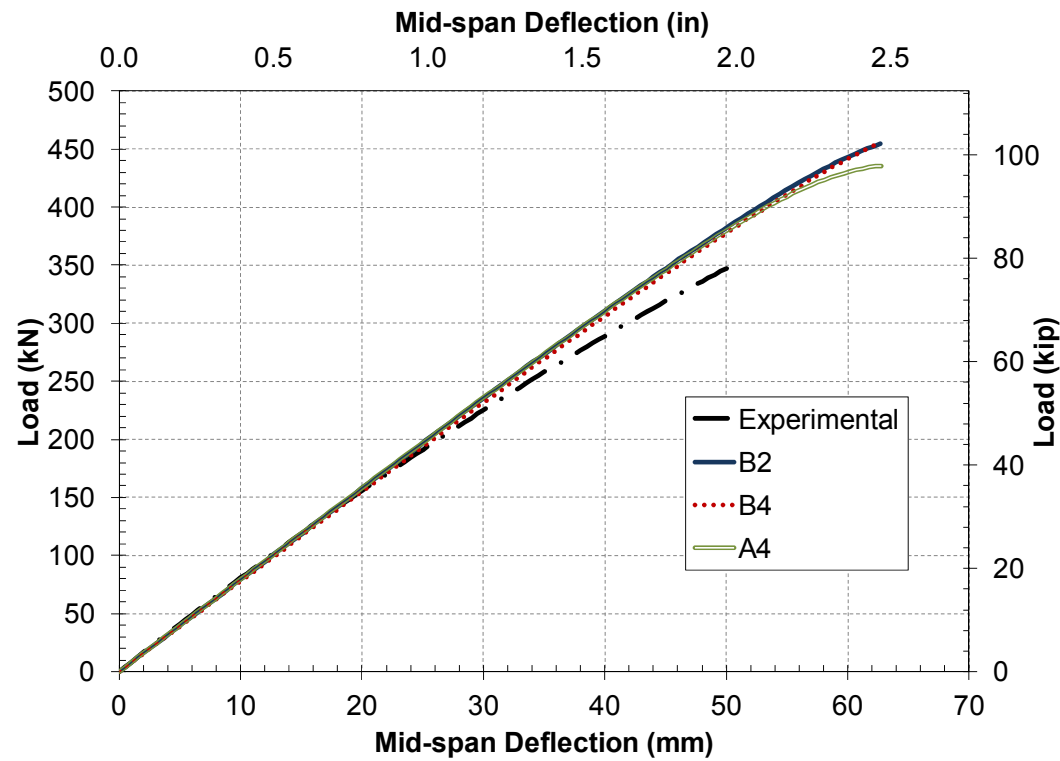


Fig. 6 – Mesh sensitivity analysis using different mesh sizes with a bond ratio of 5

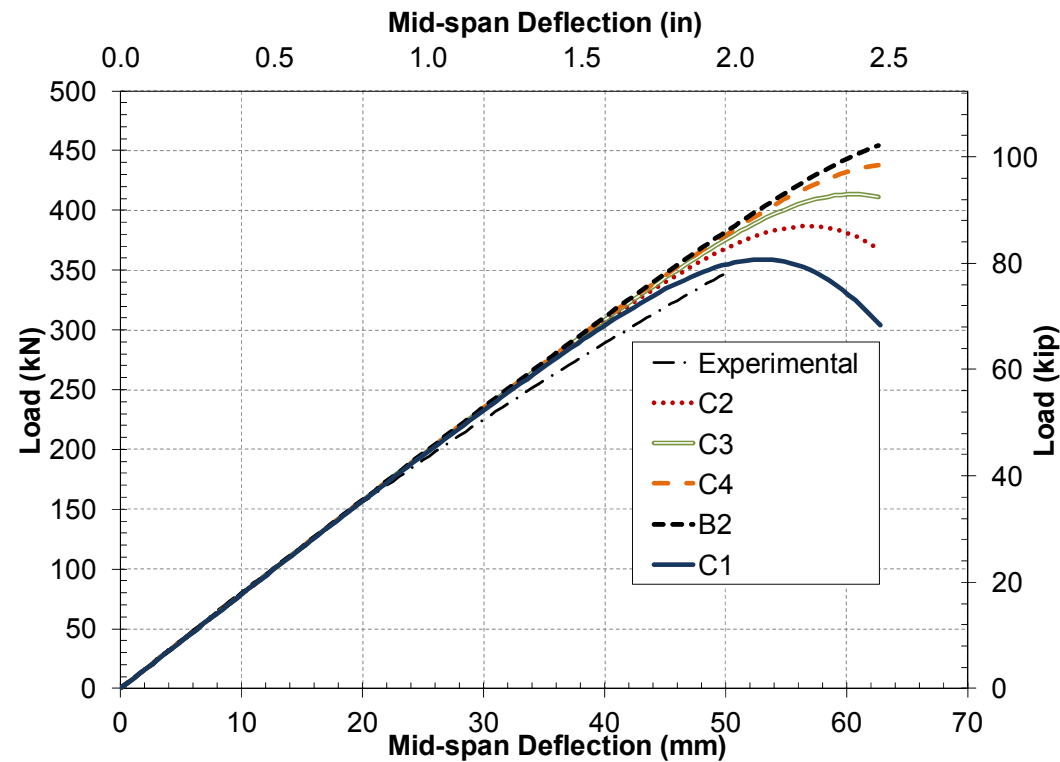


Fig. 7 – Comparison of FE solution with various cohesive element bond strength values

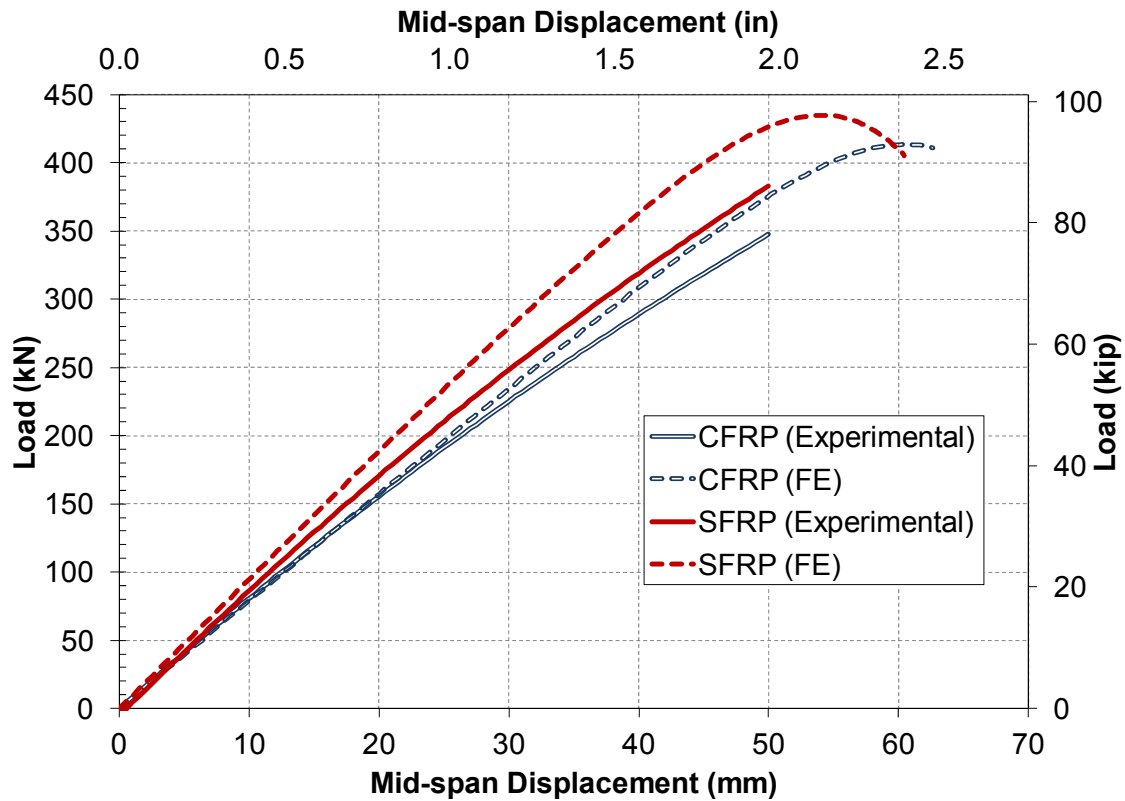


Fig. 8 – Comparison of FE solution with experimental results for both hybrid beam types

NON-LINEAR FINITE ELEMENT MODELING OF RC BEAMS STRENGTHENED USING PRESTRESSED NSM CFRP RODS SUBJECTED TO CYCLIC LOADING

Fadi Oudah and Raafat El-Hacha

Synopsis: Finite Element Method (FEM) models of Reinforced Concrete (RC) beams strengthened in flexure using prestressed Near-Surface Mounted (NSM) Carbon Fiber Reinforced Polymer (CFRP) subjected to quasi-static loading were developed and presented in this paper. One un-strengthened RC beam and four RC beams strengthened using NSM CFRP prestressed to various prestress levels were modeled. Comparisons of the load-deflection behaviors between the FEM models and the experimental test results indicate the good agreement in the loading branch. However, differences in the stiffness exist once the beams are unloaded. It seems that the concrete damage variables in tension and in compression play a paramount role in the response of the beams under cyclic loading. The possible modifications to the developed FE model are outlined as well as the future needed research.

Keywords: bond, CFRP, damage, degradation, fatigue, Near-Surface Mounted, strengthening.

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INTRODUCTION

The increasing use of Fiber Reinforced Polymer (FRP) in strengthening and design of RC structures demands the need to investigate the performance of these composite structures under variety of loading conditions such as static and fatigue loading. Near-Surface Mounted (NSM) FRP strengthening technique captured considerable interest in the past decade as an alternative method to the conventional Externally Bonded (EB) FRP strengthening due to the superior performance of the former one in terms of enhanced bonding properties, ease of installation, and protection against external damage¹. Furthermore, prestressing the NSM reinforcement showed promising results in improving the debonding resistance especially if proper anchorage is provided². Despite the large body of literature considering the static performance of NSM CFRP strengthened structures under monotonic loading and the emerging interest in their fatigue behavior, limited researchers examined their behavior analytically and numerically.

Oudah and El-Hacha³ developed a 2D Finite Element (FE) model of RC beams strengthened in flexure using prestressed NSM CFRP strips subjected to static four point bending. The developed models were validated against experimental testing and were further used in Oudah and El-Hacha⁴ to examine the effect of varying the prestress level on the ductility capacity of the strengthened beams. Evaluation against existing ductility models was also performed. Despite the simplicity of the developed models, they were capable to capture the significant variables and provide reasonable prediction of the behavior of the tested beams. Three dimensional (3D) FE models of RC beams strengthened using prestressed NSM CFRP were developed and validated against experimental test results⁵. The models were used to perform a parametric study concerning the different variables in the models⁶. Nonlinear FEM of RC beams strengthened with non-prestressed⁷ and prestressed⁸ NSM FRP reinforcement were also developed to investigate their behavior under static loading with emphasis on the bond performance at the epoxy-concrete interface. The efficiency of NSM CFRP laminates was assessed also by Kang et al.⁹. An analytical study on the groove depth revealed the existence of a critical depth beyond which the increase of the ultimate load becomes insignificant. Another FEM parametric study on RC beams strengthened using NSM rods was conducted by Soliman et al.¹⁰ in which comparisons between the FEM predictions and the experimental results revealed good agreement in terms of the load-deflection and the load-strain relationships. Several FE models were also developed to simulate pullout tests on RC prisms strengthened using NSM. Lundqvist et al.¹¹ examined the critical anchorage lengths for NSM strengthening technique using 3D nonlinear FE model. It was concluded that the numerical results are sensitive to the values of the fracture energy, the tensile strength, and the shape of the softening response of the concrete. Lundqvist et al.¹² also performed a probability study of FE analyses of NSM CFRP bonded to RC members where a special algorithm was devised to alter the groove geometry automatically while maintaining the dimensions of the elements of interest. Furthermore, nonlinear 3D analysis was performed on anchorage tests to study the failure behavior in NSM strengthened RC prisms. The results showed that the failure mode is a combined failure in the concrete and the adhesive¹³.

An extensive research project was carried out at the University of Calgary to investigate the effect of fatigue and quasi-static loading on the behavior of RC beams strengthened using prestressed NSM CFRP reinforcements experimentally, analytically, and using the FEM. The experimental results are found in Oudah¹⁴ and Oudah and El-Hacha^{15,17}, the analytical prediction model can be found in (Oudah and El-Hacha^{16,18}). This paper includes the FEM investigation of the tested beams through developing 3D models using the FEM software package ABAQUS.

RESEARCH SIGNIFICANCE

This study is the first to examine the behavior of RC beams strengthened using prestressed NSM CFRP rods subjected to cyclic loading using FEM. The different variables influencing the response under cyclic loading are outlined and investigated. Research needs are also included to draw the road map for future researchers to improve the developed model.

BEAMS TO BE MODELED

Five beams were constructed and tested under four point bending; one un-strengthened beam (B0) and four beams (B1, B2, B3 and B4) strengthened using NSM CFRP rods prestressed to prestress strains of 0, 3325 $\mu\epsilon$, 7244 $\mu\epsilon$, and 10232 $\mu\epsilon$, representing 0%, 20.8%, 45.3%, and 63.8% of the CFRP rod ultimate tensile strain reported by the manufacturer, 1.6%. The beam details are shown in **Fig. 1**. All beams were strengthened using 9.5 mm (0.37 in.) CFRP rods and were designed to increase the strength of the un-strengthened beam by 63%. The percentage increase in the ultimate strength of the strengthened beams as compared with the un-strengthened beam is an arbitrary percentage increase that is determined based on the geometry and the ultimate strength of the CFRP reinforcement used in this research. The testing regime consisted of the application of three types of loading on each of the tested beams: Quasi-Static (QS) loading, fatigue loading, and post-fatigue monotonic loading. Only the initial QS testing was modeled in this paper. The QS testing consisted of the application of three cycles; A, B, and C. The load limits were applied such that a stress range of 125 MPa (18130 psi), based on CSA¹⁹, is induced in the tension steel in the first cycle, A, with an upper stress limit of 70% of f_y , where f_y is the steel yield stress. In the first cycle, A, the beams were loaded monotonically until the strain in the tension steel reached a stress equivalent to $0.7f_y$ and then unloaded until the strain in the tension steel reached a stress equivalent to $0.42f_y$. The loads at which the beam reached the maximum and minimum limits were set as the upper and lower load limits, respectively, for the cycles B and C. Table 1 includes the target, actual, and effective prestress strains of the strengthened beams. The upper load limits and the load ranges are also included in Table 1. For more information about the significant test results of the QS, fatigue and post-fatigue monotonic test results, the reader is advised to refer to Oudah¹⁴ and Oudah and El-Hacha¹⁷.

FINITE ELEMENT MODEL

Types of elements

Fully integrated eight-node linear brick elements, C3D8, were used to model the concrete beam, steel plates, and epoxy block. Full integration was used, instead of reduced integration, in order to avoid potential hourglassing problems (i.e. zero potential energy even though the element is being deformed). Two-node linear truss elements, T3D2, were used to model the steel cage while two-node linear beam elements, B31, were used to model the steel bolts and the CFRP rods. A circular shape profile was used to model the CFRP rod and the steel bolts.

Material Constitutive Modeling

The Damage Plasticity Model (DPM) was used to model the concrete material. The DPM is designed for applications in which the concrete is subjected to monotonic, cyclic, and/or dynamic loading. It allows also for user control of the stiffness recovery effect unlike the other concrete material models in ABAQUS which is the Smeared Cracking Model (SCM). The two main failure mechanisms are tensile cracking and compressive crushing of the concrete material. The degradation of the global stiffness, when unloading, is taken into account through two degradation parameters; one for concrete tensile behavior, D_t , and the other is for concrete compressive behavior, D_c ²⁰. The damage variables D_t and D_c can take values from zero, representing the undamaged state, to one, representing the total damage.

Several analytical expressions can be found in the literature describing the softening behavior after cracking. The tension stiffening behavior implemented in this study, which is recommended by Coronado and Lopez (2006), is described by a bi-linear curve as shown in **Fig. 2 (a)**. Two parameters were needed to determine the tensile response after cracking; concrete tensile strength, f_{ct} , and concrete fracture energy in tension, G_F . Both f_{ct} and G_F were estimated using the following equation by CEB-FIP²¹:

$$f_{ct} = 1.4 \left(\frac{f'_c - 8}{10} \right)^{\frac{2}{3}} = 3.04 \text{ MPa (0.44 psi)} \quad (1)$$

$$G_F = (0.0469 d_a^2 - 0.5d_a + 26) \left(\frac{f'_c}{10} \right)^{0.7} = 92 \text{ Nm/m}^2 \text{ (0.53 lb in/in}^2\text{)} \quad (2)$$

where,

f'_c = concrete compressive strength and d_a = maximum aggregate size, 20 mm (0.79 in).

The concrete stress, f_c , versus concrete strain, ε_c , relationship under uni-axial compression was determined using the following expression developed by Loov²²:

$$f_c = f'_c \left(\frac{Ax}{1 + Bx + Cx^n} \right) \quad (3)$$

where,

$$A = 1 + B + C \quad \text{and} \quad C = \frac{1}{n-1} \quad \text{and} \quad x = \frac{\varepsilon_c}{\varepsilon_o}$$

The constants in the equation were determined based on the experimental test results of concrete cylinders such that $f'_c = 40 \text{ MPa (5802 psi)}$, $\varepsilon_o = 0.003$, $B = 0.707$, and $C = 6.4$. The graphical presentation of the stress-strain curve is shown in **Fig. 2 (b)**. The modulus of elasticity of the concrete, E_c , was determined using the following equation by CSA²³:

$$E_c = 4500 \sqrt{f'_c} = 28,460 \text{ MPa (4128 ksi)} \quad (4)$$

The Poisson's ratio and the density of concrete were taken as 0.2 and 2400 kg/m^3 (150 lb/ft^3), respectively²⁴. The angle of dilatancy was chosen to be 30 degree as recommended by Coronado and Lopez²⁵.

Voyiadjis and Taqieddin²⁶ developed a model to predict the concrete damage variables D_t and D_c . The proposed model contains seventeen parameters describing the mechanical properties and the characterization of concrete plasticity. The model, shown in **Fig. 3**, was applied to predict D_t and D_c of a typical concrete: $f'_c = 27.6 \text{ MPa (4000 psi)}$ and $f_{ct} = 3.48 \text{ MPa (505 psi)}$. Furthermore, other researchers assumed linear degradation of D_t and D_c with respect to the increase in the strain values²⁷. The concrete compressive strength had a negligible effect on the behavior of D_c ²⁸. Based on the above literature review and due to the lack of test variables regarding the plasticity of the concrete used in this research, the damage variable versus the compressive strain and the tensile displacement relationships were idealized as shown in **Fig. 4**. A bi-linear relationship was assumed for the case of D_t while a linear relationship was assumed for D_c .

The stress levels in the steel reinforcements were always in the elastic range; hence, the steel reinforcements and the CFRP rod were modeled as linear elastic materials. Tension coupon tests were conducted to determine the mechanical properties of the 10M and 15M steel rebars according to ASTM A370²⁹. The modulus of elasticity and the yield stress of the 10M steel rebars were 183 GPa (26.5 ksi) and 458 MPa (66427 psi), respectively. The modulus of elasticity and the yield stress of the 15M steel rebars were 180 GPa (26.1 ksi) and 440 MPa (63817 psi), respectively. Steel Poisson's ratio and density were taken as 0.3 and 7800 kg/m^3 (487 lb/ft^3)²⁷, respectively. Tension coupon tests were conducted in accordance with ASTM D3039/D3039M³⁰ on the CFRP rod to determine their mechanical behavior. The modulus of elasticity of the CFRP rod, determined experimentally, was 116 GPa (16824 ksi) and the ultimate tensile strain was 2.5%. The CFRP Poisson's ratio and density were taken as 0.3³¹ and 1500 kg/m^3 (93.6 lb/ft^3), respectively. Two types of epoxies were used; Type 1 was used to bond the CFRP rod to the steel anchor while Type 2 was used as groove filler. The tensile strength and the ultimate strain of Type 1 were 30 MPa (4351 psi) and 1.5%, respectively. The tensile strength and the ultimate strain of Type 2 were 24.8 MPa (3597 psi) and 1%, respectively. The Poisson's ratio and density for both epoxies were taken as 0.4³² and 1650 kg/m^3 (103 lb/ft^3), respectively.

Boundary conditions

The beam has two planes of symmetry. One plane of symmetry is the x-y plane cutting beam in half longitudinally. The other plane of symmetry is the y-z plane cutting beam in half transversely, thus, only one fourth of the beam was modeled. Planes of symmetry were held at the internal faces of the beams due to the fact that only one quarter of the entire beam was modeled. At a plane of symmetry, the displacement in the direction perpendicular to the plane was held at zero throughout the entire analysis. The translational degree of freedom in the vertical direction was also held at zero at the location of the support. Experimental test results^{1,33,34,35} and nonlinear finite element modeling^{3,5,9,11} showed that full bond is utilized at the CFRP-epoxy and the epoxy-concrete interfaces prior to yielding of the steel reinforcement. The cyclic experimental test results of the modeled beams¹⁴ did not show signs of debonding at the CFRP-epoxy / epoxy-concrete interfaces. Thus, a perfect bond was assumed, i.e. there was full transfer of load between CFRP and epoxy and between epoxy and concrete, with the application of tie constraints at the epoxy-concrete interface in order to allow for changes in mesh density, **Fig. 5 (a)**. The perfect bond assumption is further examined in proceeding paragraphs.

Geometry of beams

Three dimensional FE models were developed using ABAQUS 6.9 software. After constructing the different parts of the beam, all parts were assembled and constrained to form the RC strengthened beam as shown in **Fig. 5 (b)**. The nodes at the CFRP rods inside the anchors were merged with the nodes of the epoxy filling the anchors while the rest of the CFRP elements were embedded, their translation degrees of freedom were constrained by the response of the nodes, inside the epoxy block. Similarly, the steel cage was embedded inside the concrete beam such that its movement was constrained by the response of the whole model. The steel bolts, attaching the steel anchor to the bottom face of the beam, were also treated as embedded elements inside the steel anchor and the surrounding concrete elements. It should be noted that the mesh configurations shown in **Fig. 5 (b)** is the optimum mesh density obtained from mesh sensitivity analysis where several mesh configurations were examined¹⁴.

Loading

The prestressing was applied to the CFRP reinforcement such that the stress values are applied uniformly over the element. The applied prestress corresponded to the effective strain values and are included in **Table 1** multiplied by the modulus of elasticity of the CFRP rod. Equally spaced amplitude loading was used to apply the QS loads since it facilitates an easy application of cyclic loads. A schematic representation of the amplitude loading is shown in **Fig. 6**.

DISCUSSION AND COMPARISON

Load-deflection behavior

The load-deflection curves obtained from the FEM and from the experimental testing are compared in **Fig. 7**. The FEM and the experimental test results of the prestress camber, Δ_i , cracking load, P_{cr} , cracking deflection, Δ_{cr} , and deflection at upper load limit, Δ_{up} are summarized in **Table 2**. It is noted that Δ_{cr} and Δ_{up} were measured from zero deflection. The prestress cambers of the FE models correlate very well with the experimental test results as indicated by the low percentage differences shown in **Table 2**. Considering the relatively low percentage differences of P_{cr} , Δ_{cr} , and Δ_{up} , along with the load-deflection behaviors, the load-deflection responses of the FEM models were in good agreement with the experimentally tested beams during the loading branch of the first cycle. However, differences in the responses exist when the beams were first unloaded. This issue is elaborated in the following paragraphs. Experimentally, the load-deflection slope decreased once the beams were cracked. Nevertheless, concrete in the tension zone still contributes to the overall stiffness of the beam in the form of tension stiffening, which tends to degrade towards the yielding of the steel when the beam becomes fully cracked. Upon first unloading, the stiffness of the experimentally tested beams was enhanced and its value can be estimated by interpolating between the stiffness of the un-cracked state, where the concrete is fully effective in tension, and the fully cracked state, where there is no tension stiffening. This observation is in agreement with other experimental cyclic testing results of RC beams strengthened using various types of FRPs^{36,37,38,39,40}. The contribution of the concrete in tension degraded significantly upon first reloading and subsequent unloading and reloading cycles which

caused degradation of beam stiffness. The stiffness of the unloading branch at the end of the third cycle, C, was high and then gradually softened as the applied load approached zero. The residual slip between the reinforcement and the concrete caused the cracks to remain open, hence, results in permanent deflection. Under flexural bending, the energy input is partially dissipated in concrete cracking and steel yielding whereas the rest is stored as strain energy that can be recovered when the beam is completely unloaded after it reaches the ultimate moment. All beams in this study were cycled in the service load range, after cracking and before yielding, and hence, energy lost when unloading the beams was due to the progressive concrete tensile cracking⁴¹.

The stiffness of the FE models when first unloaded was greatly higher than that of the experimentally tested beams and they were almost identical to the un-cracked stiffness. This observation implies that the energy lost in cracking by the FEM overestimated the actual energy loss. The hysteresis loops by the FE models were also narrower than those determined experimentally. For more insight into the difference in the load-deflection responses when first unloading, the stiffness, calculated as the slope of the load-deflection curve, change at the second cycle, B, with respect to the first cycle, A, of the FE models and the experimentally tested beams are plotted in **Fig. 8 (a)**. It is observed that the change of stiffness is independent of the prestress level and that it is much higher in the case of FE models as compared with the tested beams. Experimentally, insignificant difference exists between the stiffness of the control beam and the strengthened beams. However, larger difference exists in the case of the FE models.

The increase in deflection is one of the most important parameters that should be investigated when studying the effect of cyclic loading on the performance of RC beams. The changes in deflection at the third cycle, C, with respect to the first cycle, A are plotted in **Fig. 8 (b)**. Experimentally, the deflection changes of all the strengthened beams were lower than that of the un-strengthened beam. However, the deflection change increased almost in a linear fashion with respect to the increase in the prestress level. Analytically, a general trend indicates that the deflection change is independent of the prestress level and that all the strengthened beams experienced almost the same deflection change as the control un-strengthened beam.

The possible reasons behind the difference in the response between the FE models and experimentally tested beams are summarized in the following points:

- The adopted values of the concrete damage in tension, D_t , and concrete damage in compression, D_c , do not reasonably represent the actual behavior of the concrete.
- Loading the beams in load control rather than displacement control.
- The adopted equally-spaced amplitude loading.
- The available DPM developed by Lee and Fenves⁴² and implemented in ABAQUS is not capable of capturing the response accurately when RC beams are unloaded prior to yielding.

Strain distribution

The experimental and the FEM strain distributions across the mid-span depth at the upper load limit in the first cycle, A, are compared in **Fig. 9**. The percentage differences of the strains in top fiber, top steel, bottom steel, and CFRP rod at the upper load limit are included in **Table 3**. Hairline cracks at the tension face of beam B0 were observed prior to testing. The existence of these cracks contributed to the high strain in the tension steel determined experimentally as compared with the strain obtained from the FE model. The strain distributions of the strengthened beams, in general, are in good agreement with those obtained experimentally. The low concrete strain observed at the upper load limit, within the elastic range, means that the concrete under compression was slightly damaged when subjected to cyclic loading and that the damage was mostly due to the degradation of the concrete tensile stiffness. Hence, the variable D_c did not influence significantly the response of the beams unlike the variable D_t which contributed to behavior of the beams under cyclic loading.

The effect of cyclic loading on the strain values of the tension steel and the CFRP rod is of significant importance since the design of RC members subjected to cyclic loading is based on the stress range, hence strain range, in the tension reinforcements^{19,43,44}. The change in the tension steel strain in the third cycle, C, with respect to the first cycle, A, at the upper load limit is shown in **Fig. 10 (a)**. The experimental strain changes were significantly lower than the strain change of the control un-strengthened beam which indicates the efficiency of the strengthening system in reducing the strain increase under cyclic loading. However, the strain changes obtained from the FEM were almost the same as that of the un-strengthened beam. Despite the significant difference in the strain change of the un-strengthened beam determined experimentally and analytically, the strain changes of the strengthened beams

obtained from the FEM models have the same trend as those determined experimentally, especially at high prestress levels. This observation indicates the suitability of the used tension stiffening model as well as the embedded constraint option used to model the beams.

The validity of the perfect bond assumption at the epoxy-concrete interface was examined by plotting the change in the CFRP strain at the first cycle, A, with respect to the third cycle, C, as shown in **Fig. 10 (b)** (strain gauge attached to the CFRP in B2 was damaged after the first cycle). Experimentally, the change of the CFRP strain in beams B1 and B3 indicates that the CFRP strain increased with the increase in the number of cycles, and hence, no debonding occurred. Nevertheless, at high prestress strains, beam B4, debonding seems more likely to occur under cyclic loading. Comparing the strain changes obtained from the FE models and the experimental testing, yield the conclusion that perfect bond assumption is valid for prestress level up to $6899\mu\epsilon$ after which debonding at the epoxy-concrete interface is more likely to occur.

Crack patterns

Fig. 11 shows the isometric views of the beams at complete unloading after the first cycle, obtained by visualizing the plastic strain evolution along the length of the beam. The degree of cracking is shown by the scaled colors; blue for no cracking and red for full cracking. These cracks increased in height and width in the following cycles.

Examining the crack patterns leads to the following observations:

- The cracks are closely distributed for the case of the strengthened beams as compared with the un-strengthened beam, beam B0.
- Beam B4 experienced the lowest number of cracks as compared with the other beams.
- The cracks have shear tendency in the shear span.
- In general, no major cracks are observed at the location of the steel anchor.

RESEARCH NEEDS

The developed model was validated in the elastic range. However, more research is needed to improve its behavior when subjected to cyclic loading. Following are some of the points that should be considered in future research:

- Parametric study regarding different concrete damage models is needed to investigate the effect of changing the concrete damage in tension variable, D_t , and the concrete damage in compression variable, D_c .
- Parametric study is needed to examine the effect of the nature of the applied loading, whether load-control or displacement control, on the response of the beams under cyclic loading.
- Parametric study is needed to examine the different types of amplitude loading on the performance of the beams; equally-spaced, periodic, and smooth step.

CONCLUSIONS

In this paper, 3D FE models of RC beams strengthened in flexure using prestressed NSM CFRP rods subjected to quasi-static cycles were developed and compared with experimental test results. Following are the conclusions drawn from this study:

- In general, the load-deflection responses of the FE models are in good agreement with the experimentally tested beams during the first loading stage. Hence the model can be used to predict precisely the response of RC beams strengthened using prestressed NSM CFRP rods subjected to monotonic loading. However, the stiffness of the FEM models when first un-loaded is higher than that of the experimentally tested beams and it is almost identical to the un-cracked stiffness, and hence, more research is needed to improve the cyclic response.
- The perfect bond assumption is valid since the experimental and FEM CFRP strain results are almost the same.
- Similar to the tested beams, all cracks in the FE models are developed during the first cycle. However, they tend to increase in height and width in the following quasi-static cycles.
- The difference in strain values between the experimentally tested beams and the FE models decreases with respect to the increase in the prestress level.

Despite the increasing research in the area of cyclic performance of RC beams strengthened using FRP, limited work examined their performance analytically and numerically. The developed model described in this paper is considered to be among the earliest work that attempts to simulate the cyclic performance of FRP RC strengthened beams using the FEM.

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Table 1 – Prestress strains, upper load limits and load ranges

Beam ID	Prestress strain ($\mu\epsilon$)			Upper load (kN) [kip]	Load range (kN) [kip]
	Target	Actual*	Effective**		
B0	NA	NA	NA	44.1 [9.9]	26.7 [6.0]
B1	0	0	0	65.3 [14.7]	33.9 [7.6]
B2	3400	3325	3260	73.6 [16.5]	35.3 [7.9]
B3	6800	7244	6899	87.0 [19.6]	39.4 [8.9]
B4	10200	10232	9177	88.9 [20.0]	33.9 [7.6]

* Prestress strain achieved at the end of the prestressing process

** Prestress strain after all losses (7 days monitoring period)

Table 2 – Comparison of experimental and FEM load-deflection results

Beam ID	Δ_i (mm) [$\times 10^{-2}$ in]			P_{cr} (kN) [kip]			Δ_{cr} (mm) [$\times 10^{-2}$ in]			Δ_{up} (mm) [$\times 10^{-2}$ in]		
	Exp.	FEM	Diff. (%)	Exp.	FEM	Diff. (%)	Exp.	FEM	Diff. (%)	Exp.	FEM	Diff. (%)
B0	NA	NA	-	11 [2.5]	11 [2.5]	0	1.27 [5.0]	1.4 [5.5]	10	14.0 [55.1]	13.6 [53.5]	-2.9
B1	NA	NA	-	14 [3.1]	15 [3.4]	7	1.7 [6.7]	1.1 [4.3]	-35	19.0 [74.8]	17.2 [67.7]	-9.5
B2	0.61 [2.4]	0.43 [1.7]	-30	28 [6.3]	25 [5.6]	-11	2.17 [8.5]	1.4 [5.5]	-35	17.3 [68.1]	17.2 [67.7]	-0.6
B3	0.92 [3.6]	0.90 [3.5]	-2	36 [8.1]	34 [7.6]	-6	2.56 [10.1]	1.8 [7.1]	-30	18.5 [72.8]	18.1 [71.3]	-2.2
B4	1.04 [4.1]	1.19 [4.7]	15	44 [9.9]	41 [9.2]	-7	3.29 [13.0]	2.3 [9.1]	-30	17.3 [68.1]	16.2 [63.8]	-6.4

Table 3 – Comparison of experimental and FEM strain results at mid-span cross section at the upper load limit in the first cycle

Beam ID	Top fiber strain ($\mu\epsilon$)			Top steel strain ($\mu\epsilon$)			Bottom steel strain ($\mu\epsilon$)			Mid-CFRP strain ($\mu\epsilon$)		
	Exp.	FEM	Diff. (%)	Exp.	FEM	Diff. (%)	Exp.	FEM	Diff. (%)	Exp.	FEM	Diff. (%)
B0	-913	-615	-33	-433	-429	-1	1629	986	-39	NA	NA	-
B1	-809	-646	-20	-541	-484	-10	1741	1410	-19	1625	1931	19
B2	-793	-672	-15	-590	-555	-6	1718	1730	1	1591	2024	27
B3	-732	-760	4	-593	-555	-6	1755	1800	3	2128	2201	3
B4	-954	-726	-24	-821	-538	-35	1978	1816	-8	2981	2025	-32

Non-Linear Finite Element Modeling of RC Beams Strengthened using Prestressed NSM CFRP Rods Subjected to Cyclic Loading

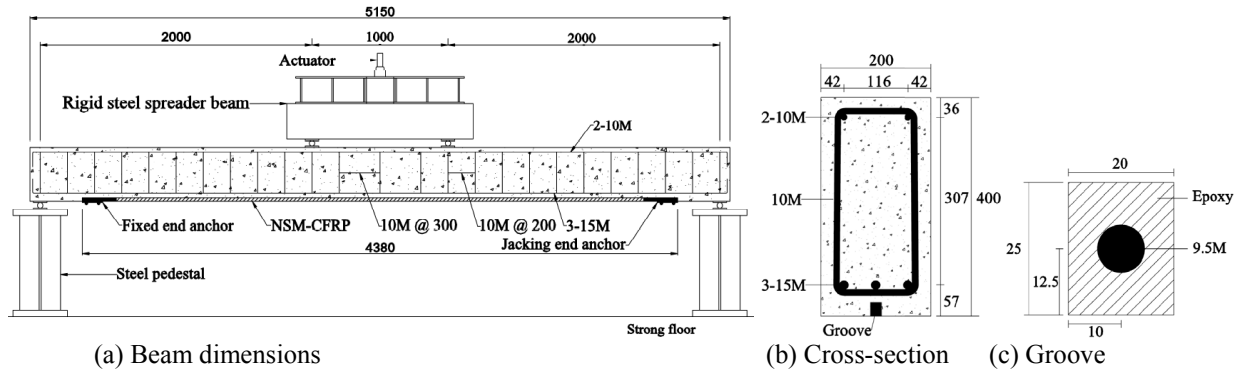


Fig. 1– Beam details

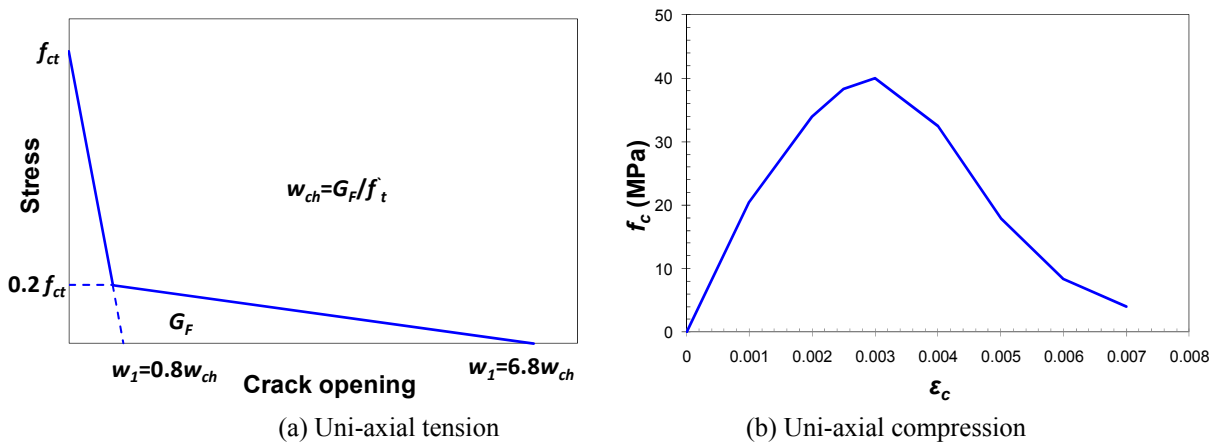


Fig. 2 – Concrete behavior

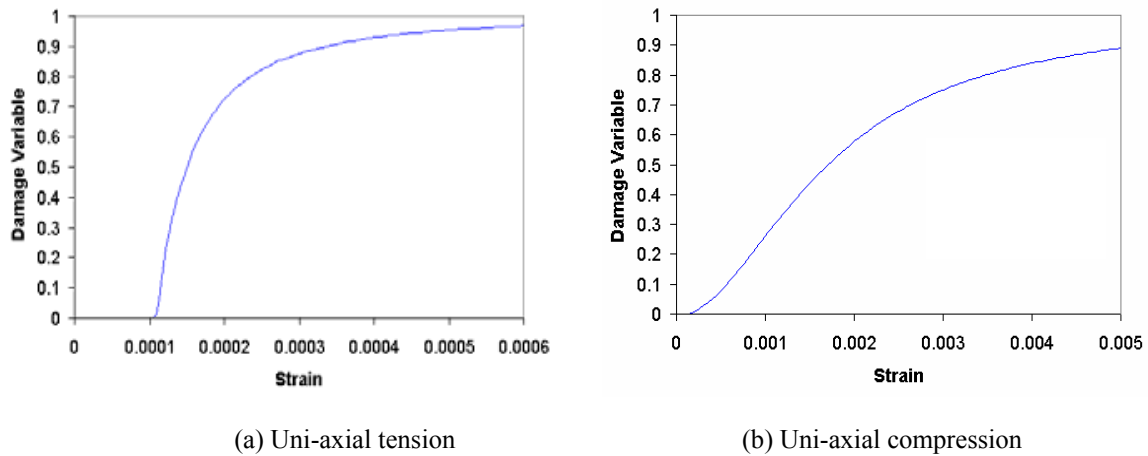


Fig. 3 – Behavior of the damage model by Voyiadjis and Taqieddin (2009)

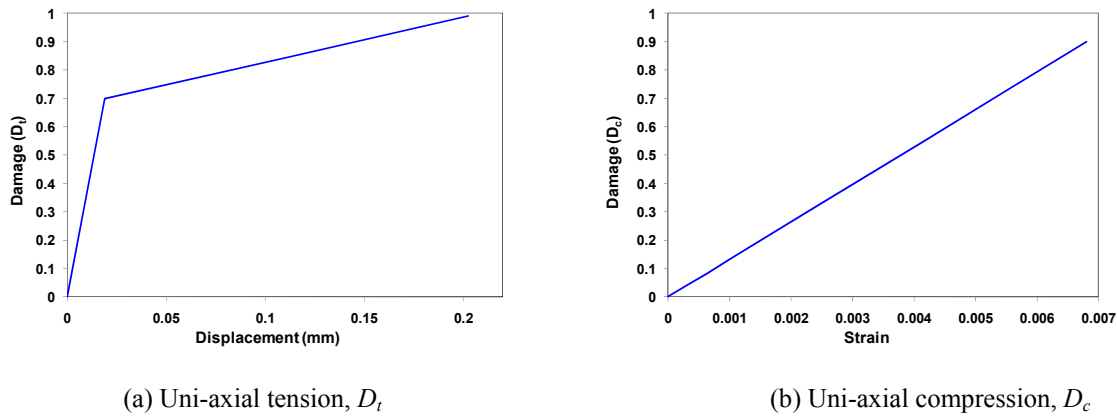


Fig. 4 – Concrete damage

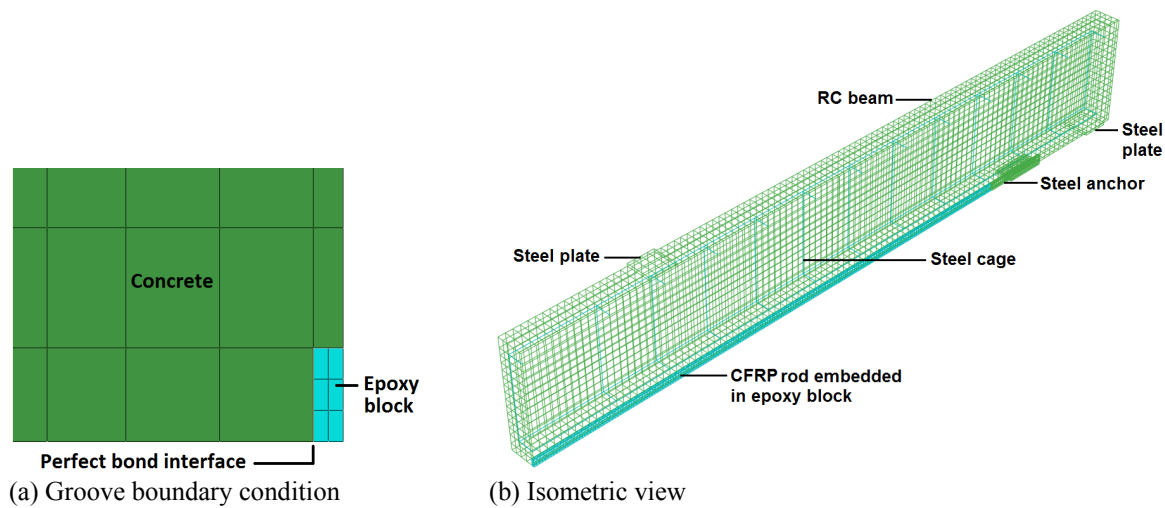


Fig. 5 –Modeled beam

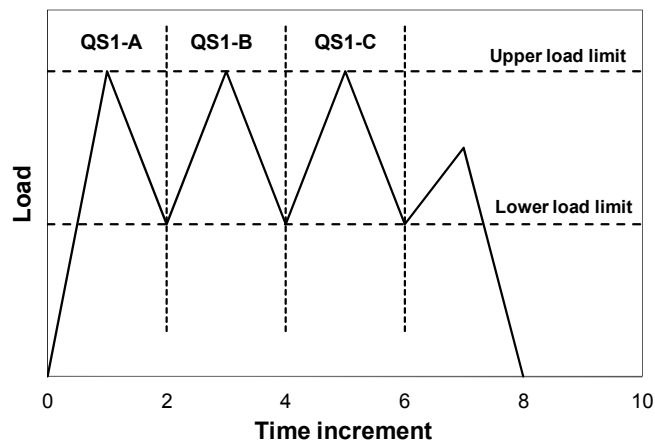


Fig. 6 – Schematic representation of the applied amplitude loading

Non-Linear Finite Element Modeling of RC Beams Strengthened using Prestressed NSM CFRP Rods Subjected to Cyclic Loading

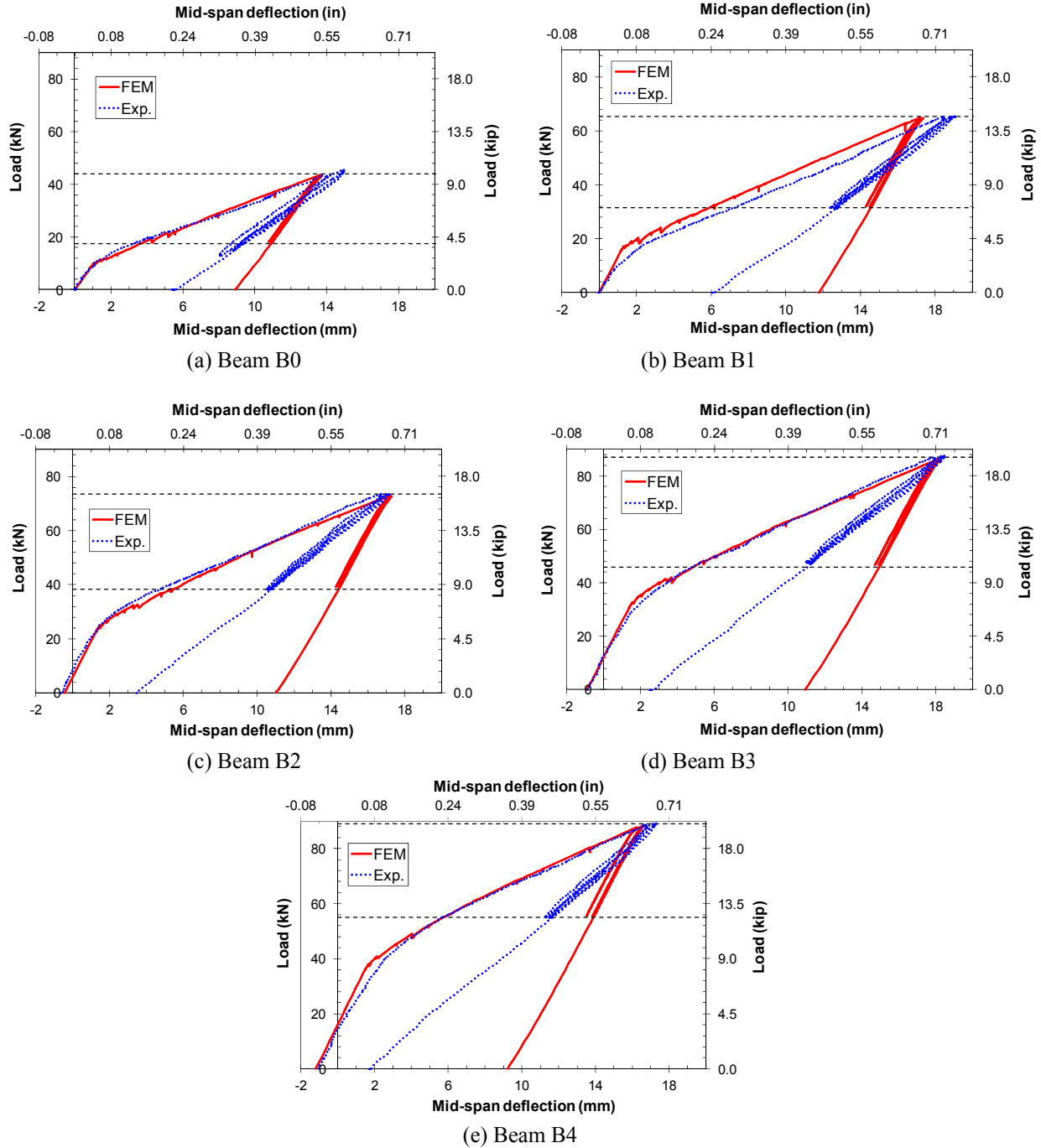


Fig. 7 – Comparison of experimental and FEM load-deflection behavior

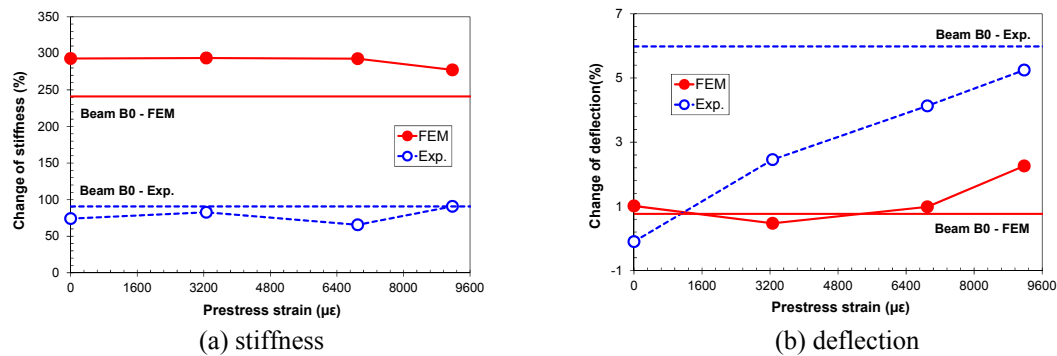


Fig. 8 – Change of stiffness and deflection between cycles A and B vs. prestress strain

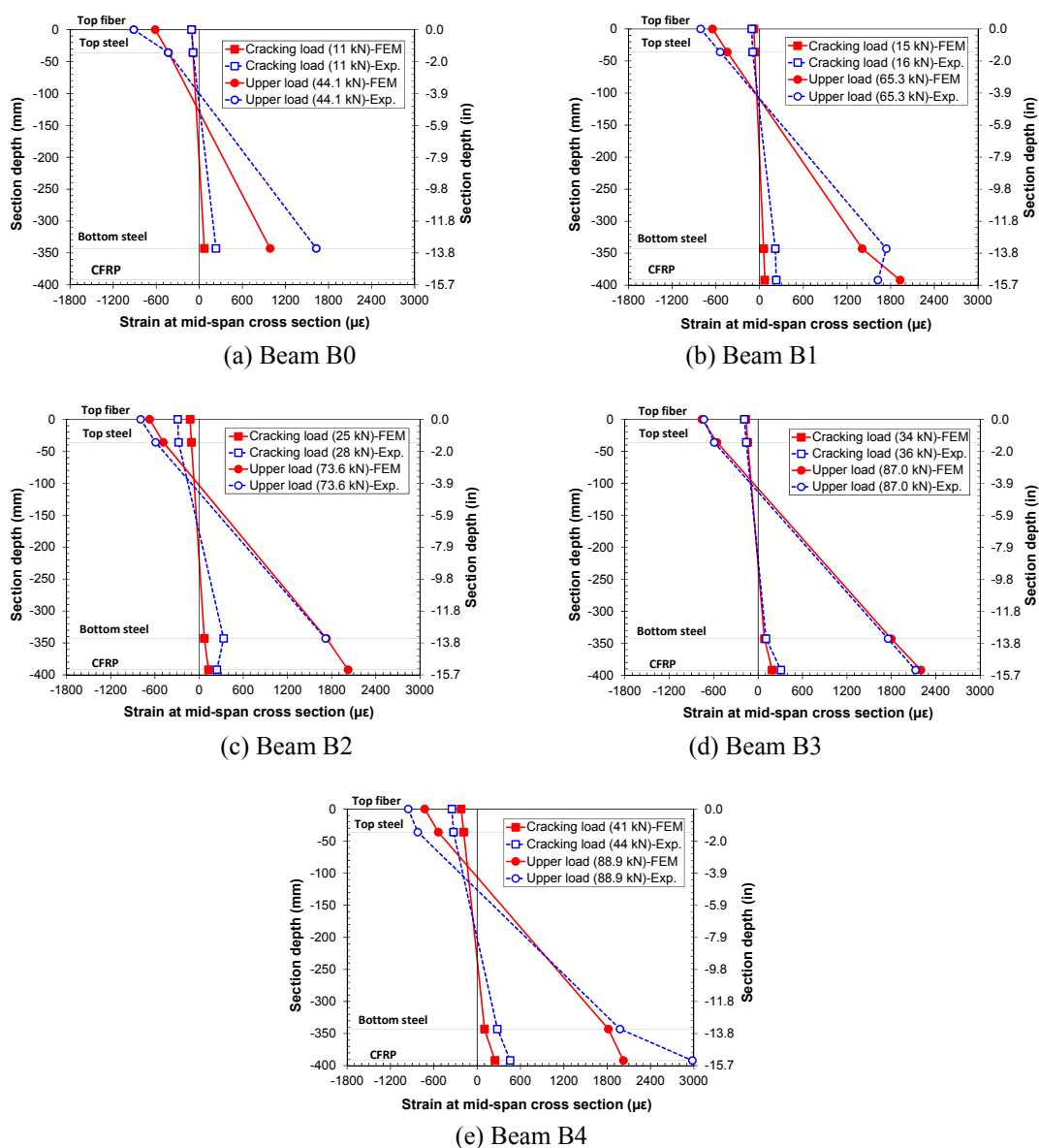


Fig. 9 – Comparison of experimental and FEM strain distribution across mid-span section

Non-Linear Finite Element Modeling of RC Beams Strengthened using Prestressed NSM CFRP Rods Subjected to Cyclic Loading

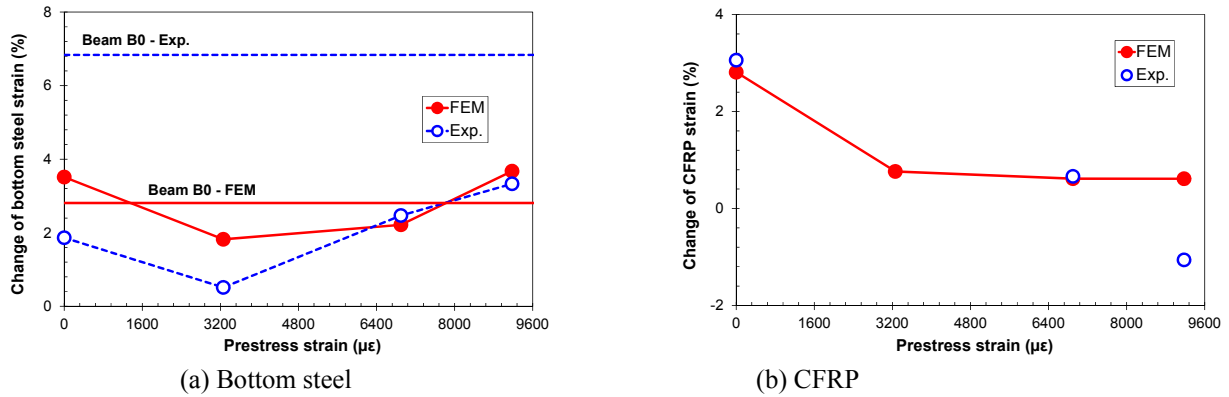


Fig. 10 – Change of strains between cycles A and C vs. prestress strain

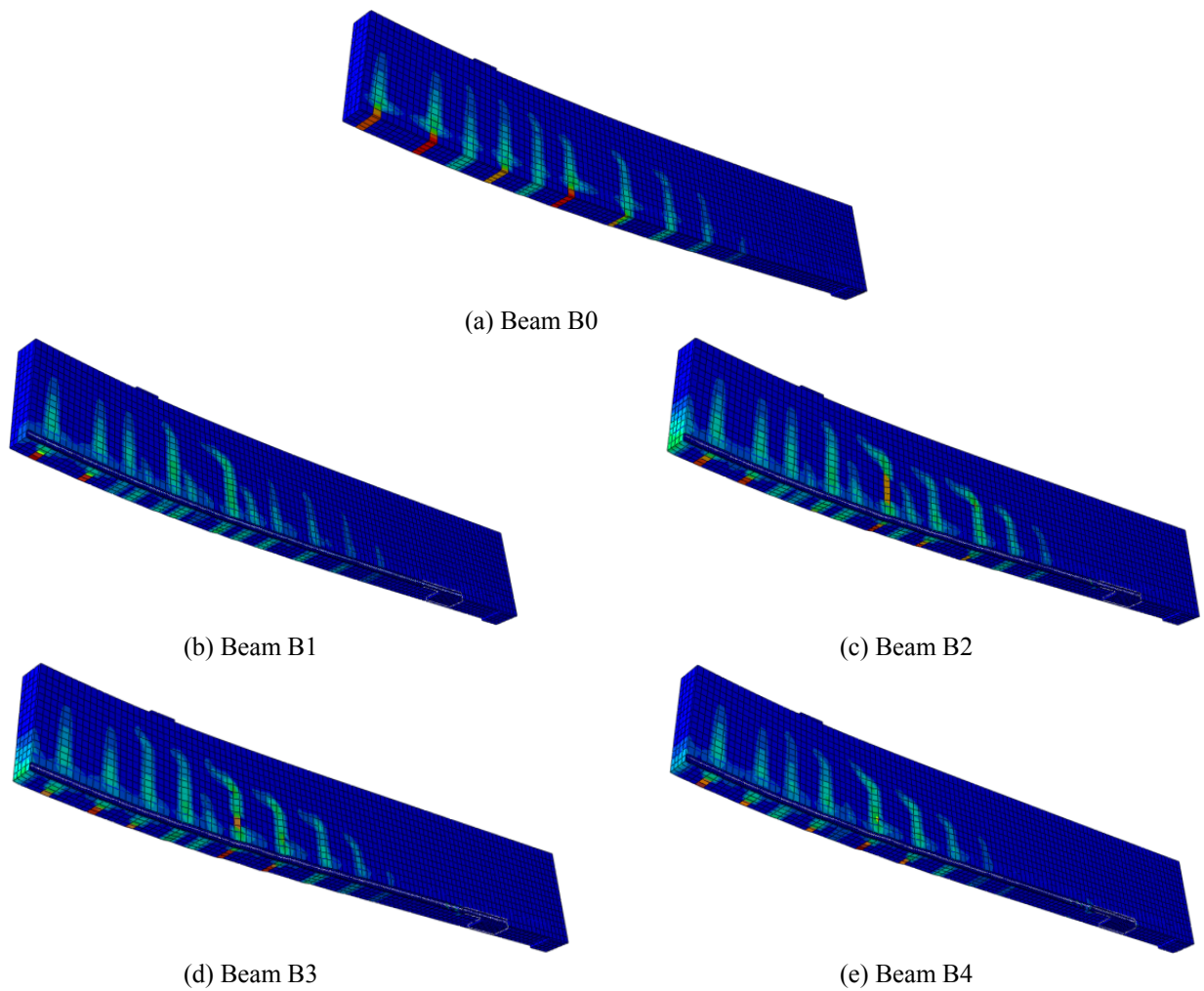


Fig. 11 – FEM Isometric view of the crack pattern at complete unloading

NUMERICAL SIMULATION OF FRP SHEAR-STRENGTHENED REINFORCED CONCRETE GIRDERS

Young-Min You, Ashraf Ayoub, Sang-Wook Bae, and Abdeldjelil Belarbi

Synopsis: The structural response of reinforced concrete (RC) girders strengthened with fiber reinforced polymer (FRP) composites in shear is investigated in this study, using a rationally-developed three-dimensional finite element model that was calibrated through comparison with test results. Analysis of reinforced concrete structures dominated by shear requires careful consideration in selecting the appropriate elements and material models. This task is more prominent in RC girders strengthened with FRP composites due to the difficulties of characterizing the corresponding properties and failure modes. Thus, the novel attributes of the proposed model is in the description of the three-dimensional constitutive material laws for each component. In the proposed model, the softening behavior of concrete under a triaxial state of stress was accounted for. The effect of the out-of-plane stress behavior of the FRP-concrete interface was carefully evaluated, which currently cannot be measured during experiments. In addition, the use of mechanical anchors to improve the bond behavior was properly simulated. Furthermore, the damage mechanism and progression of failure were carefully monitored. The model was shown to provide a good level of correlation with experimental data, and could therefore be used to conduct extensive parameter studies.

Keywords: FRP composites; Shear strengthening; Three-dimensional nonlinear FE analysis.

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INTRODUCTION

The use of fiber-reinforced polymer (FRP) composite materials in civil engineering as an economical method for strengthening of deficient concrete structures dates back to the 1970s. There exists an extensive background in the literature on how to use FRP to increase the moment capacity of structures, but studies showed a lack of knowledge in accurately predicting the behavior of reinforced concrete (RC) structures strengthened in shear with FRP sheets. Many of the proposed models to predict the gain in shear capacity from FRP strengthening were proposed, but these studies resulted in large discrepancies and produced substantial errors when compared to experimental results. A recent research study¹ revealed that only 49 experimental tests were carried out to investigate the shear strengthening effect of FRP through approximately 500 specimens. These tests, however, did not encompass the effect of the numerous parameters that can influence the overall behavior of the strengthened member. These include the tensile strength of concrete, longitudinal and transverse reinforcement ratio, shear span-to-depth ratio, size of the member, and axial forces among others. An efficient approach to properly evaluate the effect of these parameters on the structural behavior and failure mode is through well-defined finite element (FE) modeling. Therefore, the objective of this work is to develop accurate FE models that can be used to supplement the weak archival data for FRP- strengthened concrete beams in shear.

The accuracy of FE modeling in simulating the behavior of concrete structures in general strongly depends on the adopted material constitutive models as well as the modeling techniques. This task is particularly more complex in the case of shear strengthening with FRP sheets due to the failure modes, such as delamination and debonding. Initial FE studies on shear-strengthened beams assumed perfect bond between concrete and FRP²⁻⁷, while more recent studies used an interface element to account for bond-slip effects⁸⁻¹². Accordingly, this paper aims at developing a generally applicable three-dimensional nonlinear finite element model for FRP shear-strengthened RC girders. The model incorporates accurate material laws, and a rational background theory. In addition, the model adopts a phased analysis technique, in which the different loading stages such as pre-cracking, are accounted for. The developed FE model in this study was calibrated through comparison with available test results for both local and global parameters. Good agreement was observed between the analytical results and test data.

EXPERIMENTAL PROGRAM

This paper is an extension of the work by You et al.¹³, who analyzed prestressed concrete girders tested by Belarbi et al.¹ The proposed model considers the special case of RC beams strengthened in shear with FRP composites. The test program of Belarbi et al.¹ includes 8 full-scale RC girders designed to mimic the beams from a field study¹⁴, and strengthened with FRP composites in a variety of configurations to investigate various effects of FRP shear strengthening. Each girder designed and constructed to provide two distinct test regions. The primary test parameters include the effects of: pre-existing damage (cracks), transverse reinforcement (stirrup) ratio, FRP strengthening scheme, and methods of FRP anchorage. The effects of pre-existing damage were investigated in the

form of shear cracks created from applying load prior to FRP strengthening. Table 1 represents the test matrix according to these parameters; and the corresponding material properties are summarized in Table 2 and Table 3. The nomenclature of the specimen denominations indicates the beam type (RC = reinforced concrete T-beam), stirrup spacing in inches (8 in. (200 mm) or 12 in. (300 mm)), the CFRP strengthening configuration (S90 = strips at 90° and S45 = strips at 45°), the presence and type of mechanical anchorage (NA = no anchorage, DMA = discontinuous mechanical anchorage plates, SDMA = sandwiched discontinuous mechanical anchorage plates, and HS = horizontal CFRP strips), and any additional test parameters (PC = pre-existing cracks, Cor = corrosion damage, HM = negative moment).

Figure 1(a) shows the test set-up, while Figure 1(b) illustrates the cross sectional dimensions of test girders for the positive and negative moment regions. The test setup was a three point static loading configuration with each girder being designed to have two distinct test regions, one on each end of the girder. The second test region is tested by relocating the supports and loading frame accordingly. The different types of mechanical anchorage systems are depicted in Figure 2 along with the configuration of the FRP strengthening scheme.

CONFIGURATION OF FINITE ELEMENT MODEL

Figure 3 shows a representative FE model for test girders RC-12-S45-HA and RC-12-S90-SDMA-PC, which include all elements and modeling techniques used in the other FE models used in this study. The finite element program DIANA (Displacement Analyzer) was used to establish the FE models. Test Girder RC-12-S45-HA, differentiates its configuration from the other specimens due to the diagonal strengthening scheme of FRP sheets as shown in Figure 3. The eight-node isoparametric solid brick element (HX24L in DIANA) was chosen to model the concrete in the un-strengthened regions and at the loading plates, while a six-node isoparametric solid wedge element (TP18L in DIANA) was used for the concrete in the strengthened region of the test girder. The 90° FRP sheets of RC-12-S90-SDMA-PC were modeled using four-node quadrilateral orthotropic flat shell elements (Q20SF in DIANA), while a three-node triangular isoparametric flat shell element (T15SF in DIANA) was used for RC-12-S45-HA. Both shell elements take into account the wrinkling effect, which is characterized by the fact that compression stresses cannot be resisted by FRP. Because of the different geometries of the mother elements, an eight-node quadrilateral interface element (Q24IF in DIANA) was utilized with the brick element, while a six-node triangular interface element (T18IF in DIANA) was used with the wedge element. These interface elements were also used to model the mechanical anchorage regions by adjusting the material properties of interface elements. The averaged normal and shear stress distribution under the plates, which are derived from the mechanical properties of anchors provided by the manufactures and the geometrical conditions of plates, were assigned to the anchorage interface elements. Because the concept of zero thickness of interface was adopted in the FE models, the interface regions could not be observed in the final FE model. Thus, the figure of interfaces and FRP sheets illustrated in Figure 3 is the one obtained during the process of FE modeling. As shown in Figure 3, FRP sheets are created first at their strengthened locations with an arbitrary gap from the concrete surface. This is for easier modeling of the interface region between two layers. Next, interface elements are created using corresponding nodes in the concrete and FRP, respectively. It should be noted that the side and bottom layers of FRP sheets are separated at that stage. Finally, the FRP sheets are moved to their final position with an assumption of zero thickness resulting in overlapping nodes at the same location. To avoid disconnection of FRP sheets at the corners, the separated nodes of bottom and side FRP sheets are merged together as described in Figure 3. The reinforcing steels were modeled as embedded or smeared bar elements and assuming the perfect bond between concrete and reinforcing steels. To simulate the pre-cracking and FRP strengthening stages, a phased analysis was performed. The three phases of RC girders strengthened with FRP before and after pre-cracking used in this study are illustrated in Figure 4.

The FE model developed in this study adopted the rotating crack model concept using Thorenfeldt's¹⁵ uni-axial model for the concrete compressive behavior and an exponential softening relation for the concrete tensile behavior as commonly adopted in smeared crack models. In particular, the softening effect for concrete compressive strength induced by lateral cracking perpendicular to the principal compressive direction was considered using the softening coefficient proposed by Vecchio & Collins.¹⁶ Since this softening coefficient was derived and calibrated for two-dimensional problems, the extension to the three-dimensional model¹⁷ was incorporated in this study. The reinforcing steel in the FE models was assumed to follow an elastic-perfectly plastic behavior, while the FRP sheet was assumed linear elastic up to failure along the orthotropic directions of FRP sheets. A bilinear bond stress-slip relationship¹⁸ was applied for the shear behavior of interface using the given properties: