The First International Conference on Basalt Fibers and Composites (ICBFC-2019)
Nov 16-18 2019, Nanjing & Hengshui, China

NUMERICAL MODELING OF STRUCTURAL PERFORMANCE OF RC BEAMS STRENGTHENED WITH FIBER CONCRETE JACKETS

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Keywords: Numerical model, Strengthening, RC beam, Steel fiber concrete, bond interface.

ABSTRACT

Bond between steel fiber-reinforced concrete (SFRC) jackets and concrete beams plays an important role in controlling the structural performance of strengthened reinforced concrete (RC) structures. The main objective of this study as part of the research is to create and execute a detailed three dimensional (3D) finite element (FE) model to evaluate the structural performance of RC beams strengthened with SFRC jackets. In the proposal model, both material and geometric nonlinearity were taken into consideration throw choosing appropriate model. After that, the numerical FE simulations were compared with experimental tests of other investigators on specimen strengthened with SFRC jackets. On overall, the predicted FE peak loads, mid-span deflection, longitudinal steel strain, pattern of cracks and mode of failure responses agreed quite well with the corresponding measured experimental tested data at all stages of loading. The numerical results showed that the performance of beams strengthened with SFRC jackets is dependent on bond conditions between beams and the surrounding jackets. Therefore, the developed FE model is suitable as a practical and economical tool for accurate modeling and analysis of strengthening of RC beams with SFRC jackets.

1 INTRODUCTION

Recently, strengthening with fiber concrete jackets are receiving increased attention due to its superior performance compared to various methods. Strengthening with fiber concrete jacket is a promising technique, as it doesn't include steel bars for the jacket which decrease restrictions for jacket thickness comparing to conventional RC jacketing. FRC jacketing provides more ductile failure for RC elements rather than steel plates jacketing which exhibit sudden failure and suffers from corrosion. Moreover, it has more fire resistance rather than FRP wrapping [1, 12].

The objective of this paper is to develop a 3D nonlinear FE model that can accurately predicts the load-carrying capacity and response of RC beams strengthened with SFRC jackets using the FE ANSYS code version 13 [4]. The developed FE models have been validated by comparing the predicted ultimate load and mid-span deflection with the measured experimental data obtained from previous research carried out by Hassanean et al. [2, 3].

2 EXPERIMENTAL DETAILS

The numerical analysis reported herein is based on the experimental results conducted by Hassanean et al. [2, 3] that evaluated the development of steel fiber-reinforced self-consolidating concrete, (SFRSCC). The SFRSCC jackets were used for both strengthened and repairing of RC beams subjected to short time repeated loading.

The experimental program consisted of five strengthened RC beams in addition to two an un-strengthened specimen to serve as a control beam. The beams had a rectangular cross-section having a width and depth of 300 mm and 120 mm, respectively. The longitudinal compression steel reinforcement was two 10 mm diameter bars in the compression zone. In addition, 6 mm-diameter steel stirrups were used as transverse reinforcement. The details of the strengthening specimens are presented in Table 1.

Table.1: Dimensions, reinforcement and used materials tested beams [2,3].

Series	Beam	F_{cuj} MPa	F_{cu} MPa	t_j mm	V_f %	S.C	F_y MPa	S mm	A_s mm^2
1	BO	-	20	-	-	-	600	-	201
	BE1	60	20	30	0.75	-	600	-	201
	BE2	60	20	30	0.75	S.L	600	80	201
	BE5	60	20	30	0.75	D.L	600	80	201
	BE7	60	20	30	0.75	S.t	600	80	201
2	BOs	-	25	-	-	-	400	-	201
	BO2	80	25	50	1.5	S.t	400	20	201

3 NUMERICAL MODEL DETAILS

3.1. MODELING STRATEGY

A 3D-FE beam model was created using ANSYS code version 13 [4] based on the previously given details for concrete geometry and reinforcements; see Fig. 1. In the proposed FE model, element types, material models, bond–slip laws, and boundary conditions were carefully nominated and employed to simulate the performance of RC beams strengthened with SFRC jacket. From the element library of ANSYS code [4], four elements were selected to simulate the behavior of the beam specimens. 3D 8-node solid structural element (SOLID65) was used to model the concrete of the beams and the SFRC jacket. The solid element has eight nodes with three degrees of freedom at each node. This element models concrete cracking in three orthogonal directions, concrete crushing and treats the nonlinear behavior of concrete (plastic deformation). (SOLID 45) element was used for the steel plates at the supports for the beam. Longitudinal steel bars and steel stirrups were represented by 3D 2-node structural bar elements (LINK180). To consider sensitivity of the beam behavior to the effects of several bond conditions, the spring element (COMBIN39) of zero length was used to connect the nodes of the concrete beam to the nodes of the jacket using the generalized force– deflection curves. (COMBIN39) element is used for the simulation of bond-slip for the adhesive layer between old concrete and new layer. The element is defined by two node points and a generalized force-deflection curve. The element simulates a spring with a virtual length that has longitudinal or torsion behavior in up to three directions at each node.

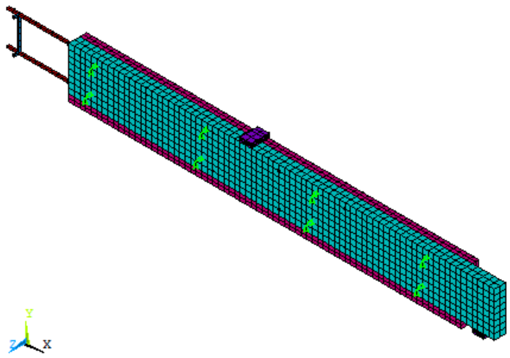


Fig. 1: Typical 3D FE model of simulated SFRC beam specimens.

3.1.2. MATERIAL MODELS

Cracking of the concrete in the tensile zones, nonlinearity of the concrete in compression zones, and plasticity of the steel reinforcement were taken into account to simulate the causes for nonlinearity. To simulate the material properties of the Solid65 elements, the procedure used in the work of Wolanski [8] on RC flexural beam was adopted. The nonlinear plastic behavior of concrete in compression was defined using the MacGregor model [5], while the concrete tensile stress–strain response was modeled using the ACI model [6]. In both models, (f_c) is the maximum concrete compressive strength and (εo) is the corresponding axial strain = 2 f_c/E_c, (ft) is the ultimate tensile strength = 0.62 $f_c \times e^{0.5}$, and Ec is the modulus of elasticity of concrete. The concrete poison's ratio was assumed equal to 0.2. The open and closed shear coefficients, which are typically in the range of zero to 1.0, were taken as 0.3 and 0.9, respectively. Additionally, the constitutive concrete model by William and Warnke [9] was used to define when failure will occur in concrete elements. It worth mentioned that the concrete of the beam was simulated with elastic concrete material properties having the same modulus of elasticity and Poisson's ratio of the concrete of the beam. The nonlinear response of the steel reinforcing bars was assumed to be bilinear elasto-plastic with a strain–hardening ratio of 0.01 [7]. Steel Poisson's ratio was specified as 0.3. The jacket was modeled as nonlinear plastic material with modulus of elasticity in tension and compression. Ascending and descending branches of the stress-strain curve should be implied, and the equations should represent both ascending and descending branches of the curve. The equation based on physically significant parameters that can be experimentally determined [10]. The addition of steel fibers into concrete increased both its crack stress and ultimate tensile strength. The effect of tension strength is taken into consideration in the model [11]. Finally, the rigid steel plates of loading were simulated with elastic steel material properties having a modulus of elasticity and Poisson's ratio of 200 GPa and 0.3, respectively.

3.1.3. BOND–SLIP MODELING

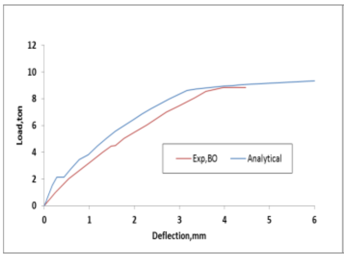
This method used multi-linear spring elements to connect the nodes of the beam and the jacket. These spring elements are unidirectional elements with a nonlinear generalized force against deflection capability. The longitudinal behavior was modeled by a uniaxial tension–compression element with up to three degrees of freedom at each node. These were translations in the nodal x, y and z directions. Two node points and a generalized force against deflection curve characterize this element. The behavior of the spring element was defined using the model [13] and the shear stress was calculated using equation as indicated below.

$$\left(\frac{\tau_f}{\tau_{fud}}\right)^4 - 0.50 \times \left(\frac{\tau_f}{\tau_{fud}}\right)^3 = 0.30\,\mathrm{s} - 0.03 \quad \text{Where } \tau_{\mathrm{fud}} = 0.40 \times (f_c^{\,2} \times \sigma_c)^{1/3}$$

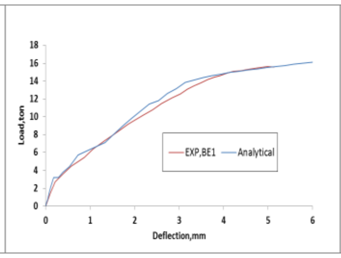
4. VERIFICATION OF THE FINITE ELEMENT MODELS

In order to appraise the developed FE model and test its validity, a comparison between the FE numerical simulations and experimental results of Hassanean et al. [2, 3] have been carried out. Figure

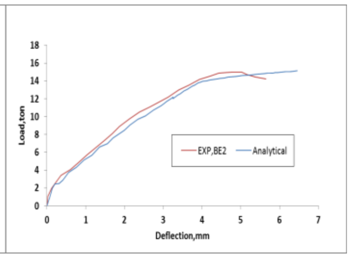
2 shows a comparison of load mid-span deflection of tested beams, figure 3 shows a comparison of cracking pattern and mode of failure of tested beams. Finally figure 4 shows a comparison of load mid-span and main steel strain of the tested beams.



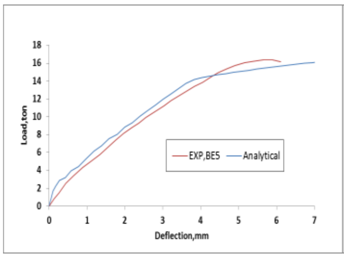
BO control beam [2].



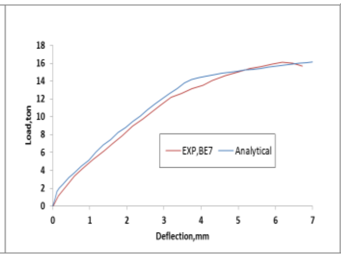
BE1 beam without shear connector [2].



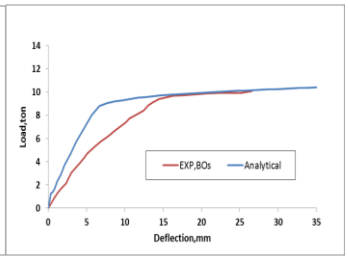
BE2 beam with S.L shear connectors [2].



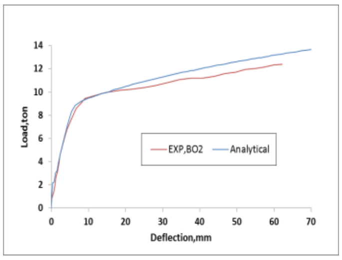
BE5 beam with D.L shear connectors [2].



BE7 beam with S.t shear connectors [2].

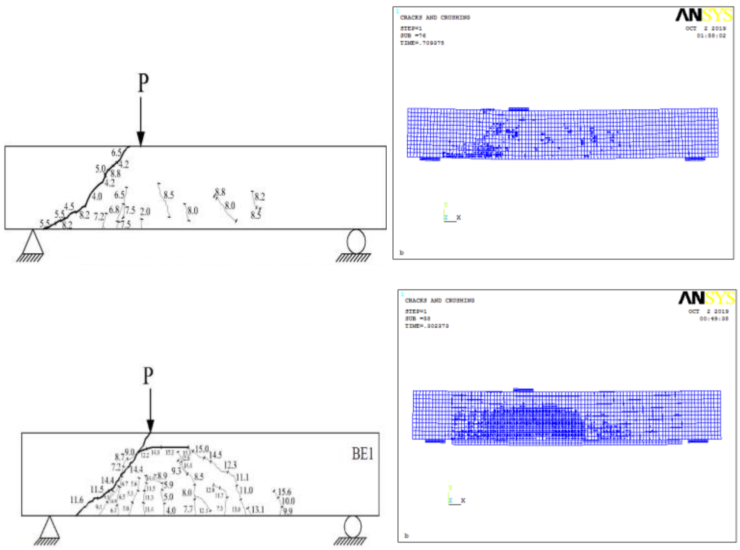


BOs control beam [3].



BO2 beam with S.t shear connectors [3].

Fig. 2. Comparison of the load-mid-span deflection response for specimens presented in [2, 3]



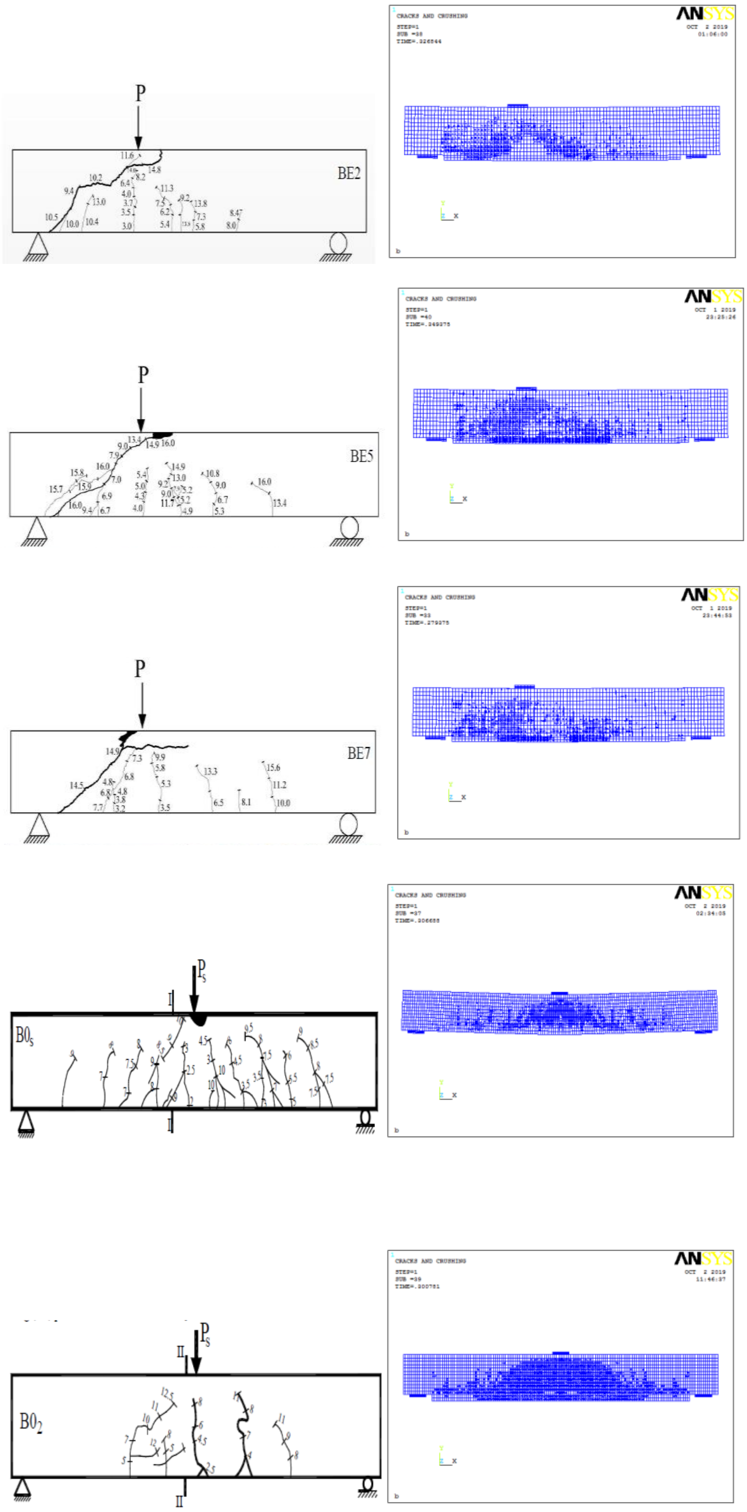
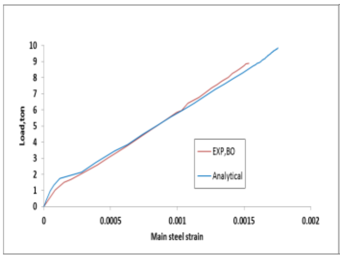
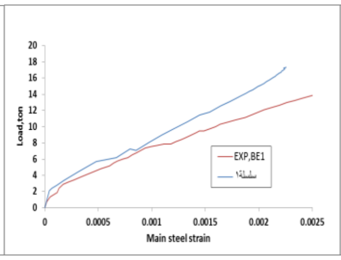


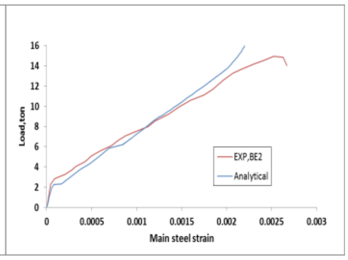
Fig. 3. Comparison between experimental results and FE results of the cracking pattern and mode of failure for the beams presented in [2, 3].



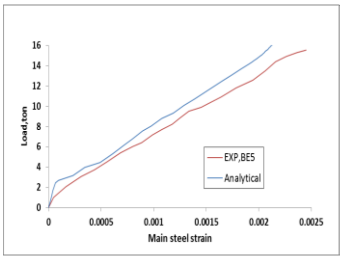
BO control beam [2].



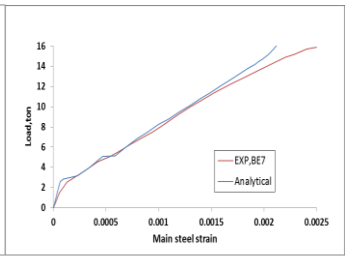
BE1 beam without shear connector [2].



BE2 beam with S.L shear connectors [2].



BE5 beam with D.L shear connectors [2].



BE7 beam with S.t shear connectors [2].

Figure 4 Comparison between the experimental and numerical results presented in [2].

It is clear from figure 2 that there is a quite good agreement between the predicted FE numerical simulations and experimental records at all stages of loading up to failure, the maximum deviation between the experimental and predicted numerical results for the ultimate loading is less than 8% for the entire specimens whereas for the ultimate deflection it does not exceed 10%. Also, figure 3 shows crack pattern at the ultimate load for series 1 and series 2. There is a quite good agreement between the predicted FE numerical simulations and experimental records. Finally, it is clear from figure 4 that there is good agreement between the predicted FE numerical simulations and experimental records in load-longitudinal steel strain curves for the beam specimens presented in series 1. The maximum deviation between the experimental and predicted numerical results for the longitudinal steel strain at the same load is similar.

5. CONCLUSION

In this numerical study of RC beams strengthening by SFRSCC jacket analytically are made to evaluate the effect of the fibers on the flexural, ductility, cracking, and shear behavior of the beams. The developed model was validated by comparing FE numerical simulations with their corresponding experimental measurements available in the literature. According to the results, the following observations can be made:

- The developed and validated finite element model presented in this study is suitable for modeling and analyzing RC beams strengthened with SFRSCC jackets and quite accurately and efficiently performing parametric studies on different retrofitting configurations.
- There is a quite good agreement between the experimental results and numerical simulations in terms of the load–deformation response at all stages of loading up to failure of the specimen.
- The mode of failure and pattern of cracks are the same for experimental results and numerical simulations.

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NOTATIONS

ε	Strain of the concrete.
ε_o	Strain at the ultimate compressive strength.
F_{cu}	Ultimate compressive strength of concrete, (MPa).
f_t	Tension strength for concrete, (MPa).
F_y	Yield strength for steel, (MPa).
ε_y	Yield strain at the yield strength.
V_f	Volume ratio of fiber content, %.
S	Spacing between shear connector, mm.
A_s	Area of bottom reinforcement, mm2.
F_{cuj}	Compressive strength of strengthened jacket, MPa.
t_j	Bottom thickness of jacket, mm.
S.C	Distribution of shear connector.
SFRC	Steel fiber- reinforced concrete.
FRP	Fiber reinforced polymer.
SFRSCC	Steel fiber-reinforced self-consolidating concrete
τ_f	The roughened interface shear stress in MPa.
τ_{fud}	The ultimate value of the shear stress in MPa.
S	The sliding in mm at the interface,
S_{fu}	The maximum value of sliding at the interface taken 2.0 mm
σ_c	The normal stress at the interface in MPa.