American Journal of Engineering Research (AJER)
e-ISSN: 2320-0847 p-ISSN : 2320-0936
Volume-10, Issue-01, pp-70-78
www.ajer.org
Research Paper Open Access

Numerical analysis of RC beams strengthened in shear using externally bonded FRP sheets

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ABSTRACT : *Finite Element (FE) simulation of Fiber Reinforced Polymer (FRP) of strengthened Reinforced Concrete (RC) beams is a complicated problem due to the inability to accurately represent concrete materials. Recently, the FE ABAQUS software Concrete Damaged Plasticity (CDP) tool has been used in a broad range to represent concrete materials. The present study provides non-liner three-dimensional (3D) finite element analysis (FEA) of RC beams strengthened in shear using externally bonded FRP sheets. Eight strengthened beams with FRP collected from Literature to examine the calibration of detailed numerical model in a particular focus on ultimate shear capacity verses displacement. The parameters of CDP model dilation angle (ψ) and viscosity (μ) are adopted using the experimental dataset. Numerical results had presented that the FRP sheets increased both the shear capacity of the strengthened beam and the beam ductility over about 50%. The comparison clarified that the coefficient of variation between experimental dataset ultimate load-displacement versus numerical results is about 3.5%.*

KEYWORDS: *RC Beams, FRP Shear-Strengthened, FEA, ABAQUS*

Date of Submission: 29-12-2020 Date of acceptance: 10-01-2021

I. INTRODUCTION

It has become very popular today to use FRP sheets in the rehabilitation and strengthening of RC beams due to their low weight-to-strength ratio, non-magnetic properties, and resistance to corrosion. RC beams are needed to strengthened or repair when the beam is deficient in shear due to an increase of applied loads or an incidence of damage arising from defects in design or construction. In the last few years, many researches with FRP composites have been performed on shear strengthening (e.g. [1,2,3,4,5,6,7]) this has contributed to a deeper perception of the FRP shear strengthened behavior but this laboratory researches wasted time and effort plus money Therefore, numerical model which is more economical than experimental researches are needed to deep understanding of RC beams strengthened in shear.

Chen et al. [8] studied non-liner two-dimensional (2D) Finite Element (FE) simulations of shear-reinforced RC beams using externally bound FRP sheets using ABAQUS based on interfacial behavior and phenomenon of debonding. Steel bars and was modelled as truss elements (T3D2). FRP sheets was modelled tie elements perfectly bonded compared with Cohesive Interaction technique that available in ABAQUS software which provides bond–slip relationship between FRP and concrete interface. The FEA unified interaction model was validated against the test findings, showing a strong compatibility with the experimental results with a maximum variance of not more than 10 percent, but the numerical perfectly bonded model contributed to a broad shear effect.

Mostofinejad et al. [9] research in NSM technique in two approach experimental and numerical. In experimental approach, he tested four rectangular beams with cross section dimensions 200 mm in wide × 300 mm in height × 2000 mm in length. In numerical approach, he modeled non-liner three-dimensional (3D) Finite Element Analysis model in ABAQUS software. The behavior of the FRP/concrete interface is simulated as cohesive tensile strength interaction 2.9 Mpa, with Knn 25700 Mpa and Kss = Ktt 130 Mpa. The load-displacement curves conform to experimental findings with a maximum difference less than 10%.

Manos et al. [10] examined the effects of the FRP sheet anchorage system. Using ABAQUS to accurately simulate the used anchorage method, a 3-D Finite Element model was developed, and results compared to experimental findings, the FE findings showed a very strong correlation with experimental dataset. The anchorage system used improved the performance of FRP sheets and thereby increased the ability of shear capacity.

Godat [11] developed numerical analyses with the ADINA finite element tool. A comparison for simulation of FRP sheets, between truss elements and shell elements was done The FE results using shell elements for FRP sheets showed that were more effective to predict the load–deflection behavior than using truss elements.

The current study presents a three-dimensional RC beams strengthened in shear using externally bonded FRP sheets. The CDP model was used to represent the concrete behavior. The key parameters of CDP model were calibrated with 6 experimental datasets.

II. FINITE ELEMENT MODEL

A. CONCRETE

The representation of concrete materials is one of the biggest challenges in FE simulation. Recently the concrete damage plasticity CDP is available in ABAQUS software to simulation the non-linear behavior of concrete [12]. CDP can easily simulate the non-linearity of concrete that is based on compressive crushing and tensile cracking of concrete. Also, CDP model in briefly considers the non-associated Drucker-Prager hyperbolic flow potential function is based on the research by Lubliner et al. [13] and Lee, and Fenves [14]. Under uni-axial compressive loading, the CDP model acts as linear until the initial yield value is reached, followed by a hardening of stress and softening of strain overriding the final stress. The following non-linear plastic forward rule is utilized in the CDP model:

$$\dot{\varepsilon_p} = \lambda \frac{\partial G}{\partial \sigma} \tag{1}$$

$$G(\sigma) = \sqrt{(\epsilon.\sigma_{t0}.\tan\psi)^2 + \bar{q}^2} - \bar{p}.\tan\psi \tag{2}$$

where (σ) and (ε̇p) refers to the stress and plastic strain rate tensors, respectively, (λ) is a plastic multiplier, and (G) is the Drucker–Prager function, (p) is the hydrostatic stress and (q) the von Mises equivalent stress. and (ψ) is the dilation angle measured in the (p – q) plane at high confining pressure. and (ϵ) is the eccentricity parameter and (σt0) is the uniaxial tensile stress at failure.

In presented paper complete stress-strain curve for concrete under compression proposed by Hsu et al. [15]. This stress-strain curve used for maximum compressive strength up to 60 MPa. For other concrete grades, modifications should be referring to the original paper by Hsu et al. [15]. Figure .1 clarifying the ultimate compressive stress (σcu); (ε0) is strain at σcu and (εd) is strain corresponds to the stress at 0.3 σcu in the descending portion.

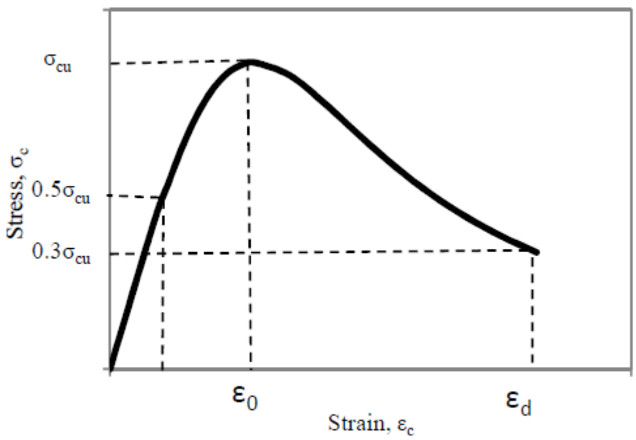


Fig.1. Concrete Compressive behavior [16]

The model by Hsu et al. [15] is used only to calculate the compressive stress values (σc) (kip/in2) between the yield point (at 0.5 σc) and the (0.3 σc) in the descending portion using:

$$\sigma c = \frac{\beta(\varepsilon c/\varepsilon o)}{\beta - 1 + (\varepsilon c/\varepsilon o)^{\beta}} \sigma c \tag{3}$$

$$\beta = \frac{1}{1 - [\sigma c(E o \varepsilon o)]} \sigma c \tag{4}$$

$$\varepsilon_o = 8.9 \times 10^{-5} \sigma c + 2.114 \times 10^{-3} \tag{5}$$

$$E_o = 1.2431 \times 10^{2} \sigma c + 3.283 \times 10^{3} \tag{6}$$

Concrete tensile behavior in this research simulated by the exponential curve as showed Figure 2. Exponential curve is the most accurate representation of concrete behavior in tension by using represent the stress crack opening displacement according to Cornelissen et al [17]:

$$\frac{\sigma_t}{f_{ct}} = \left[1 + \left(c_1 \frac{wt}{w_{cr}}\right)^3\right] e^{\left(-c_2 \frac{wt}{w_{cr}}\right)} - \frac{wt}{w_{cr}}(1 + c_1^3)e^{(-c_2)} \quad (7)$$

Where (σt) is the normal tensile stress to the crack direction. (fct) =0.33√fc [18] is the concrete uniaxial tensile strength; (wt) is the crack–opening displacement; (wcr) is the crack–opening displacement at the complete release of stress or fracture energy; c1 and c2 are material constants is taken as 3.00 and 6.93, respectively [20]; (Gf) is the total fracture energy of concrete required to create a stress–free crack over unit surface.

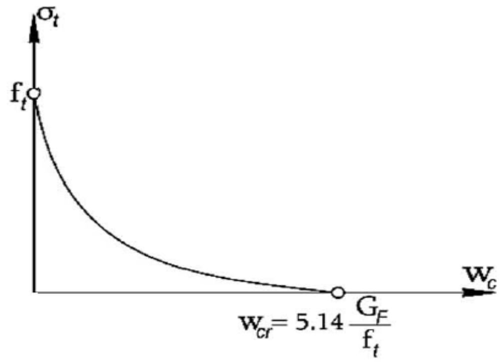


Fig. 2. The exponential curve [20]

B. STEEL

The steel reinforcement is assumed to be elastic perfectly plastic. The elastic behavior was defined by the longitudinal elastic modulus and Poison's ratio of 0.3. Truss element (T3D2) is the element used to model steel bars in ABAQUS software [12]. The bond between steel and concrete is represented as embedded interface methodology [10,21]. Figure .3 showed steel bar behavior.

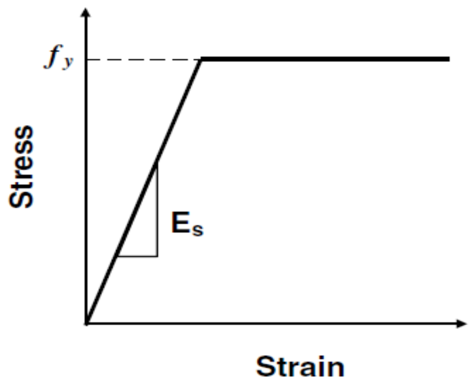


Fig. 3. steel bars behavior

C. FRP BEHAVIOR

The tensile rupture failure of FRP being established using the tensile strength obtained from flat coupon measurements, FRP reinforcement is presumed to be linear-elastic-brittle. The Shell element (S4R) was picked to represent the FRP reinforcement in the ABAQUS software. A relationship between the local shear stress, (τ) and the relative displacement (s) is simulated by the action of the FRP and concrete interface [22]; three separate bond-slip relationships have been proposed to be categorized according to their degree of evolution. [20] simplified and bilinear. The bilinear gives an accurate representation of debonding behavior of FRP reinforcement [8].

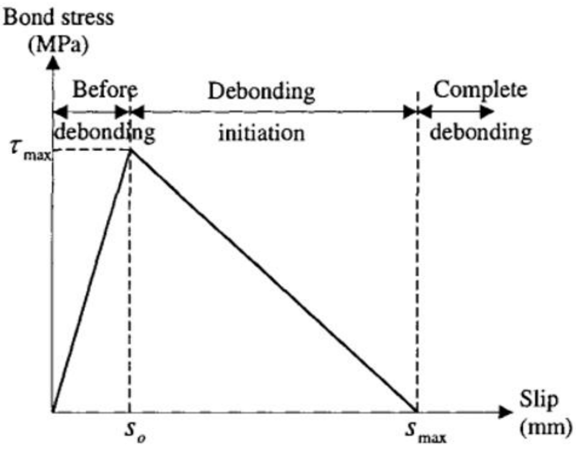


Fig. 4. Bilinear bond-slip model [11]

As seen in Figure. 4, the bilinear model was used in the current analysis. The bilinear model is based on simplicity. The fracture energy values are entirely similar for two models. Although the debonding happens within the concrete cover at a few centimeters [11], it is visible. There is no criterion taken into consideration that takes into account the adhesive properties or thickness. The tensile strength of the concrete (ft) was one of the parameters that regulated the FRP-concrete interfacial behavior. Considering that the overall bond stress (MPa) and the related slip (mm) are the maximum bond stress [22], the following coefficients are:

$$\tau = (\tau_{max}/so)\, s \tag{8}$$

$$\tau_{max} = 1.5 f_{ct} \sqrt{(2.25 - \frac{bf}{sf})/(1.25 + \frac{bf}{sf})} \tag{9}$$

$$k_{nn} = \frac{1}{\frac{tc}{Ec} + \frac{tepoxy}{Eepoxy}} \tag{10}$$

$$k_{ss} = k_{tt} = \frac{1}{\frac{tc}{Gc} + \frac{tepoxy}{Gepoxy}} \tag{11}$$

where (So) the corresponding slip (mm), bf and sf are width of FRP composites and the central spacing between the strips, respectively (in the case of continuous sheets bf/sf = 1); (Gepoxy) is Shear modulus of or adhesive in MPa; (tc) is Concrete thickness, mm; (tepoxy) is thickness of adhesive; (Eepoxy) is Young's modulus of adhesive in MPa.

III. NUMERICAL MODEL VERIFICATION

To verify the accuracy of the numerical model, only quarter of strengthened RC beam is molded to save the analysis time as shown in Figure .5 with mesh size 15×15 mm. Finite element results are compared with experimental dataset of Adhikary and Mutsuyoshi [1]. The authors Adhikary and Mutsuyoshi tested beams named from B1 to B6. One beam that was considered as a control beam named B1 while the other five were strengthened using FRP sheets. In different ways to the top edge of the beam or to limited height in this Specimens making true simulations of reality without neglected slab thickness in strengthen beams.

The dimensions of the tested beams were 150 mm in width and 200 mm in height and 2,600 mm in length for all beams There was no internal stirrups in the shear span to ensure Shear failure, even though FRP sheets are added. There is also to be understanding Shear strengthening effects of FRP sheets when there are no steel stirrups given. The Geometrical and mechanical data of strengthened RC beams given in Table 1.

To adopt the accuracy of CDP model, high number of trails were performed with two main CDP parameters viscosity (μ) and dilation angle (ψ). Table 2 presents the values of the CDP model parameters needed to be adopted according to Literature [21,23] and recommended values of ABAQUS user manual Guide [12]

Viscosity parameter (μ) were taken range to 0.00001,0.0001, and 0.001 while concrete dilation angle (ψ) was taken range to 20,25, and 30 in permutations and combinations for two specimen B2 and B4 as shown in Figure 6 and Figure7, respectively. Then the numerical Model had been developed by using the proposed CDP model with viscosity (μ) parameter were taken equal to 0.0001 while concrete dilation angle (ψ) was taken equal to 25 in order to simulate the ultimate load- displacement relationship as shown in Figure 9.

CDP model adopts different criteria to show the crack direction. It does not show the crack direction itself [12]. However, it assumes that the direction of the vector normal to the crack plane is parallel to the direction of the maximum principal plastic strain. For Specimen B4 a noticeable large crack occurs in the shear area of strengthened beam at angle of 45o as shown in Figure .8 In ABAQUS FRP debonding observed from

Scalar stiffness degradation for cohesive surface ABAQUS Output, (CSDMG). diagonal shear crack and Shear crushing clearly observed from compression damage parameter and FRP rupture observed by FRP sheet stress, s11 (main local direction FRP sheet stress). FRP rupture failure mode occurs when stress in FRP sheet reaches the ultimate sheet stress (fu,FRP). Specimen B4 fail with debonding as mentioned in literature dataset. Figure 8. Show ABAQUS Output Failure modes and crack pattern for specimen B4.

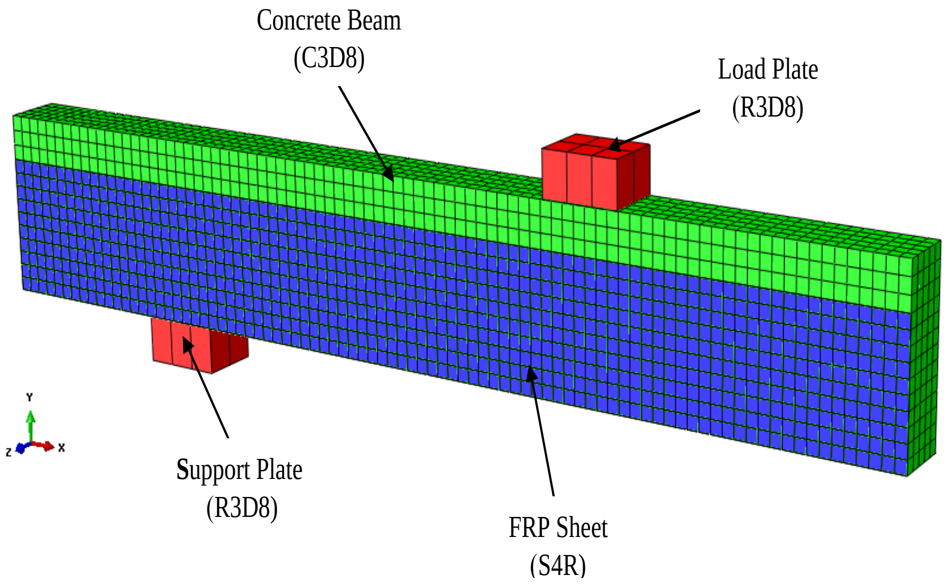


Fig. 5. Quarter of strengthened RC beam.

Table 1. The Geometrical and mechanical data of strengthened RC beams.

Specimen name	Beams Geometrical characteristics						Concrete	Tensile steel			Compression Steel			FRP characteristics					Failure mode
	b(mm)	*h(mm)*	*c(mm)*	*L(m)*	*Lc (m)*	*a(mm)*	*fc,(Mpa)*	*N*	*Ø*	*fy*	*n*	*Ø*	*fy*	*(S,U)*	*tf*	*df*	E_{FRP}	f_{FRP}	
B1	150	200	30	2.6	1.94	510	30.5	2	22	582	2	13	576	-	-		-	-	Diagonal crack
B2	150	200	30	2.6	1.94	510	35.4	2	22	582	2	13	576	S	0.17	100	230	3400	Rupture
B3	150	200	30	2.6	1.94	510	33.5	2	22	582	2	13	576	S	0.33	150	230	3400	Rupture
B4	150	200	30	2.6	1.94	510	31.5	2	22	582	2	13	576	S	0.17	150	230	3400	Debonding
B5	150	200	30	2.6	1.94	510	31	2	22	582	2	13	576	S	0.33	150	230	3400	Debonding
B6	150	200	30	2.6	1.94	510	33.7	2	22	582	2	13	576	S	0.33	200	230	3400	Debonding
B7	150	200	30	2.6	1.94	510	34.4	2	22	582	2	13	576	U	0.17	150	230	3400	Shear crushing
B8	150	200	30	2.6	1.94	510	35.4	2	22	582	2	13	576	U	0.17	200	230	3400	Shear crushing

where specimen refer to experimental beam as named according [1], L is the total length of the beam in m; b and h are width and height of the beam cross-section in mm, respectively. c and are the concrete cover in mm and Lc clear length from supports in m, a is shear span in mm, *fc* is the concrete compressive strength in Mpa. n, ø and *fy* is the number, diameter in mm and yield strengths in Mpa of the bar respectively and S or U refers to configurations of frp sheet S for 2 side wrapped and u for u-wrap *tf*, refers to thickness of frp layers in mm. *ffrp* and *Efrp* are the thickness, ultimate strength and elastic modulus of frp sheets in Mpa and Gpa,

Table 2: Recommended values of CDP Model Parameters.

Parameter Item	ψ	E	σ_{bo}/σ_{co}	Kc	μ
ABAQUS user manual Guide [12]	37	0.1	1.16	0.667	0
Parameter range in literature [21,23]	20:35	0.1	1.16	0.667	0.001:0.00001

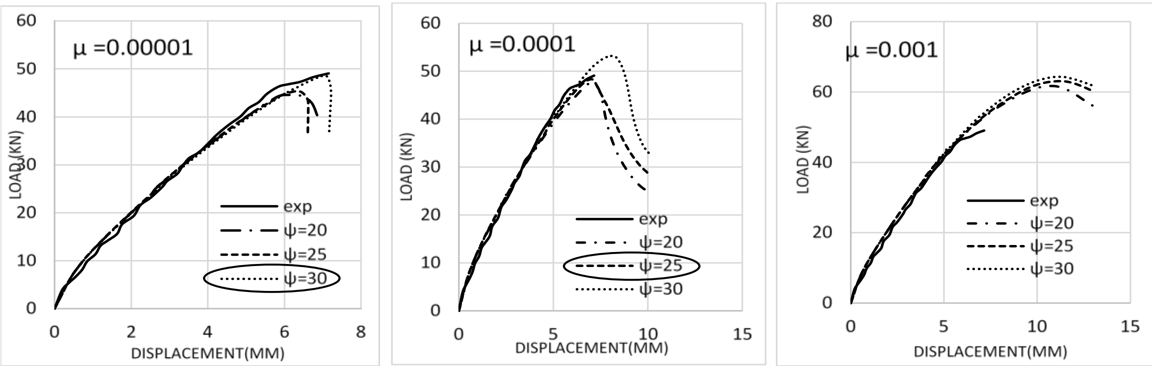


Fig. 6. Ultimate load-displacement Attempts for Specimen b2

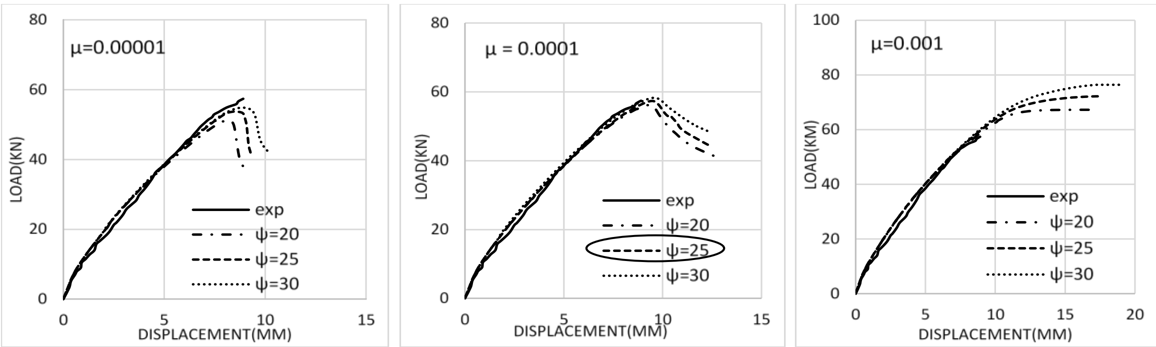
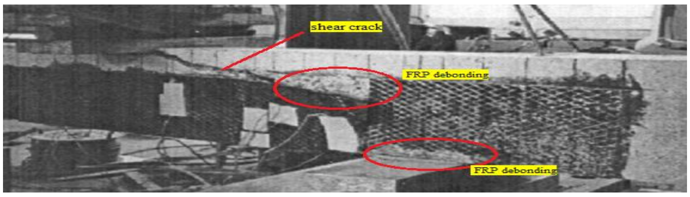
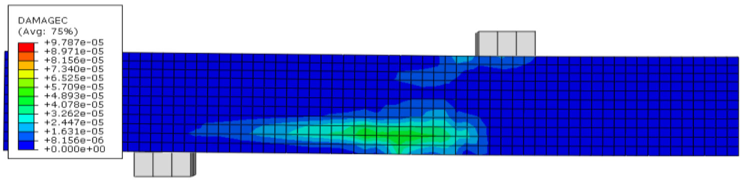


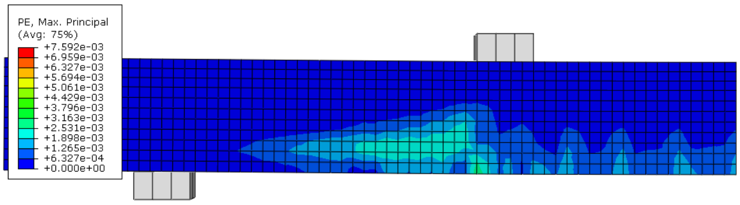
Fig. 7. Ultimate load-displacement Attempts for Specimen b4



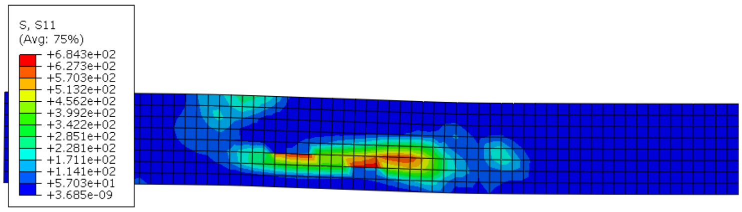
(a) Experimental



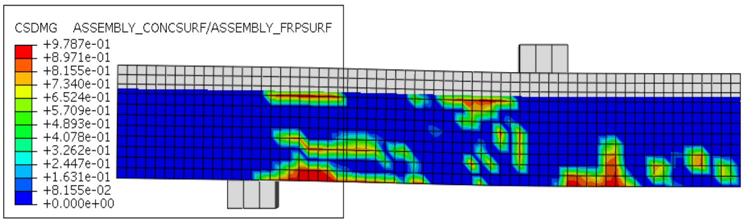
(b) ABAQUS Output, Compression damage of RC beam



(c) ABAQUS Output, Crack pattern of RC beam



(d) ABAQUS Output, Stress in FRP sheet



(f) ABAQUS Output, FRP debonding failure

Fig. 8. ABAQUS Output Failure modes and crack pattern for specimen B4

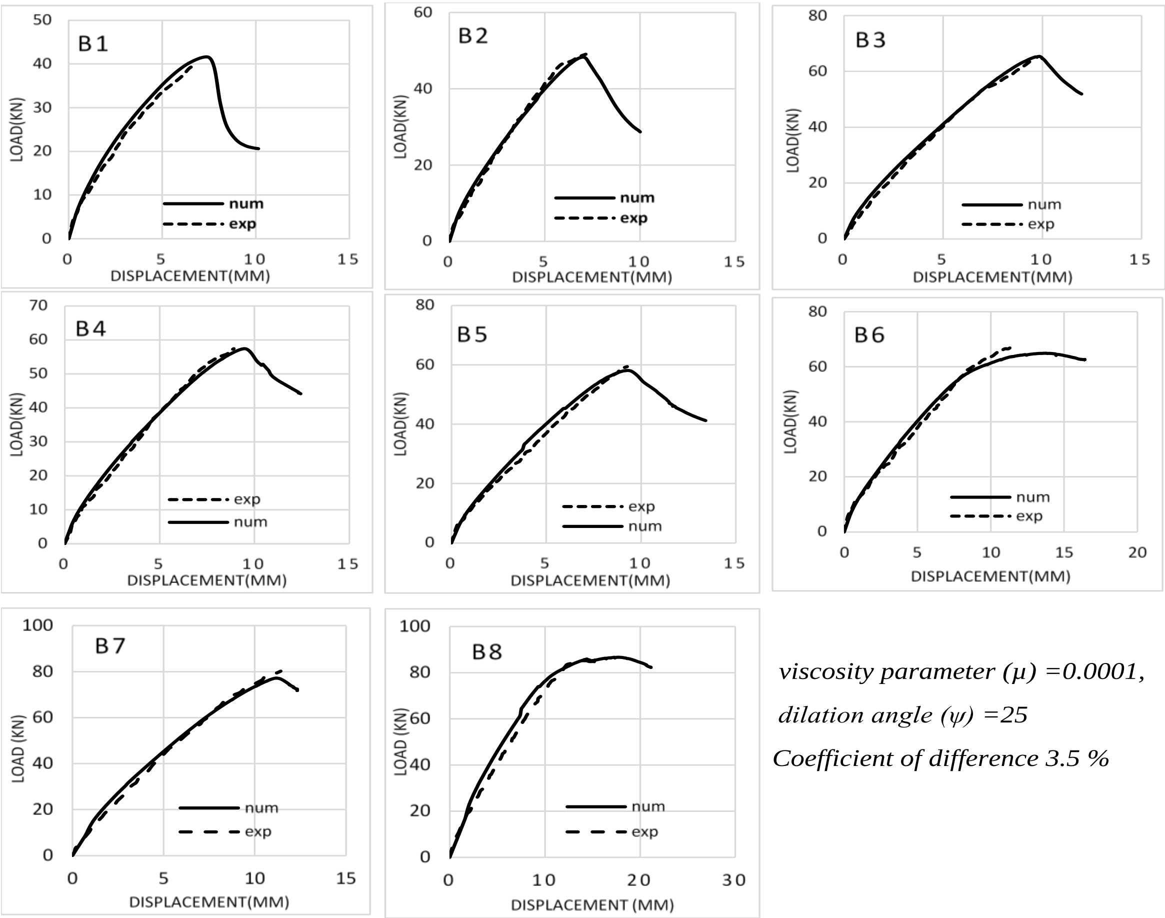


Fig. 8. Experimental versus Numerical ultimate load-displacement curves.

IV. CONCLUSION

The numerical model is developed in this paper to predict the reaction of shear-strengthened reinforced concrete beams with externally bonded FRP composites with a specific emphasis on the creation of a numerical model that can capture the actual behavior of externally FRP shear- strengthened beams. The numerical model is easy and essential to replicate actual specimens and research many of the various effects without wasting time and money Compared to experimental approach.

The findings indicate that the shear capacity of the strengthened beam was increased by the FRP sheets and the beam ductility over about 50 percent relative to the control beam. The difference between the ultimate load-displacement relationship of experimental and numerical specimens are roughly less than 5 percent from literature dataset with coefficient of difference 3.5%.

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