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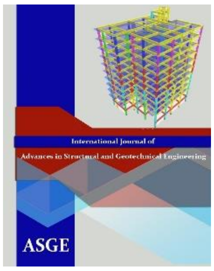

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ABSTRACT

Reinforced concrete (RC) jacketing is nowadays one of the most common techniques adopted for strengthening of existing RC columns. It is used to increase load-carrying capacity and ductility of weak existing members by means of a simple and cheap method. In this paper, a finite element model of RC columns strengthened with RC jacket under eccentric loading were developed using two-dimensional shell element model. To simulate the interaction between old and new concrete, the adhesive layer was modeled using cohesive surface-to-surface interaction model. Results of the finite element (FE) model are verified by comparing them with experimental work available in the literature. The result of the FE model showed good agreement with experimental results, as they were able to predict the behavior of the strengthened columns and the modes of failure with high accuracy. A parametric study was conducted to study the effect of assuming perfect bond at the interface between old and new concrete on the behavior of strengthened columns.

Keywords: Self-compacting concrete, Finite element model, Shear connectors, Cohesive interaction.

INTRODUCTION

In developed countries, rehabilitation and strengthening of existing structures surpasses construction of new structures. This has both economic and environmental advantages and is associated with many factors, including safety, serviceability and durability of the structure [1]. There are several methods for strengthening reinforced concrete (RC) columns, each one with different advantages, depending on the goals. RC jacketing is always more frequently adopted to strength existing RC columns with poor structural features. The critical aspect of this technique lies on the connection between the new concrete and the old one, so appropriate procedures should be taken to ensure a monolithic behavior [2]. Different researches were carried out in the last twenty years to evaluate experimentally the efficacy of the technique on the structural behavior of RC columns. Ersoy et al. [3] tested two series of jacketed columns under uniaxial compression or combined axial load and bending moment. They studied the effectiveness of repair and strengthening jackets and the differences between jackets made under load and after unloading. Julio et al. [4] carried out an experimental study to analyze the influence of the interface treatment on the structural behavior of columns strengthened by RC jacketing. After testing seven full scale models of column-footing, they concluded that for undamaged columns a monolithic behavior of the composite element can be achieved even without increasing their surface roughness, using bonding agents, or applying steel connectors before strengthening it by RC jacketing. Takeuti et al. [5] tested twelve RC-jacketed columns under uniaxial compression with and without preloading. The authors found that the entire core contributes to the axial capacity of the jacketed column, as long as adequate confinement is provided. Also, preloading does not adversely affect the capacity of the jacketed column, while it may increase its deformability. Nascimento [6] suggested improving the interface between old concrete and strengthening concrete by adding shear connectors in various amounts and positions. Experimental study was adopted in their study. The results of their study showed that choosing to use shear connectors and self-compacting concrete (SCC) can be considerably positive, nevertheless a greater number of shear connectors has been shown necessary to obtain a more ductile failure mode and avoid debonding failure mode. The current paper aims to provide a finite element model (FEM) for strengthening of RC columns with RC jacket using ABAQUS [7]. The details of the FEM (element types, constitutive models, and interaction between old and new concrete) are described. Based on the verified FE method, a parametric study is conducted.

1. Finite element analysis

In order to obtain an efficient and accurate finite element method, the analysis was conducted in ABAQUS/Standard module [7]. All parts of the model are presented in details as follows:

1.1 Element types, meshes and boundary condition

A 4-node bilinear plane stress quadrilateral element **(CPS4)** was adopted for the old and new concrete. And for the reinforcement steel a 2-node linear 2-D truss **(T2D2)** element was used. The relation between concrete and the RFT were perfect bonded modelled by embedded region constraint.

1.2 Material modeling

The Concrete damage plasticity (CDP) criterion is used to model the old and new concrete. The compressive crushing failure and tensile cracking failure are assumed [7]. The fracture energy method was used to specify the post-peak tension failure behavior of core and jacket concrete. For the uni-axial compression stress-strain curve of the concrete, the stress–strain relationship proposed by Saenz [8] was used as reported in [9]. The steel is modeled to be bilinear elastic-plastic material and definitions in tension and compression.

1.3 Contact modeling (Concrete-Concrete interface)

Abaqus/CAE allow for the modeling of adhesive layer using the traction-separation law in order to allow for the debonding failure mode. The available traction-separation model in Abaqus assumes initially linear elastic behavior followed by the initiation and evolution of damage, Fig. 1. Damage initiation refers to the beginning of degradation of the cohesive response at a contact point. The process of degradation begins when the contact stresses and/or contact separations satisfy certain damage initiation criteria. Maximum stress criterion was used which assumes that, the initiation of damage occurred when the maximum contact stress ratio (1) reaches a value of one. This criterion can be represented as:

$$max\left\{\frac{\sigma_n}{\sigma_n^o},\frac{\tau_s}{\tau_s^o},\frac{\tau_t}{\tau_t^o}\right\}=1 \qquad \text{Eq. (1)}$$

where represent the peak values of the contact stress when the separation is either purely normal to the interface or purely in the first or the second shear direction, respectively. And is the cohesive tensile stress and are the cohesive shear stress in the two perpendicular directions s and t. From Fig. 1, it is obvious that the relationship between the traction stress and effective opening displacement is defined by the elastic stiffness, *Knn*, *Kss*, and *Ktt*, the local strength of the material, , and the energy needed for opening the crack, *Gcr*, which is equal to the area under the traction–displacement curve.

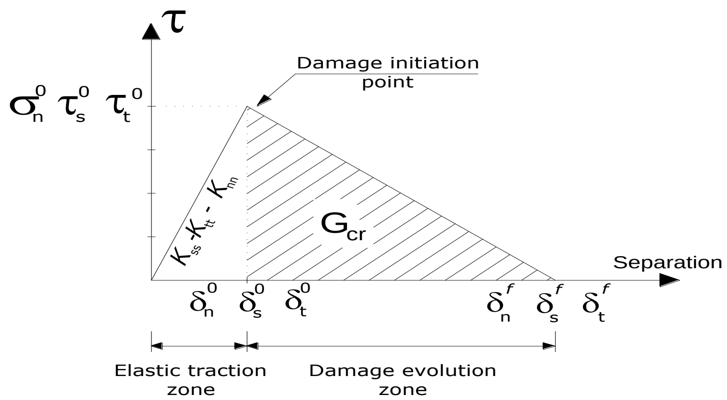


Fig. 1: Description of the traction-separation behavior [7]

To simulate the shear connectors, Abaqus/standard provides node-to-node interaction method using Cartesian connector element. Cartesian connector element provide a connector between two nodes that allows independent behavior in three local Cartesian directions.

2. Verification of the FE Model with Previous Work

To calibrate the FE model, a comparison was carried out between the finite element results and that reported in the experimental work of Nasimento et al. [6]. The experimental study focused on the behavior of self-compacting concrete jacket (SCC) –strengthened RC columns with different number of shear connectors. All columns were tested under eccentric loading. The experiment consists of nine columns. Two of columns were used as no strengthened reference columns. The remaining columns were strengthened with SCC on the compressed face. Reinforcement details and dimensions of the tested reference column is shown in Fig. 2. Also, Fig. 3 shows the location of shear connectors on the compressed face of each strengthened column.

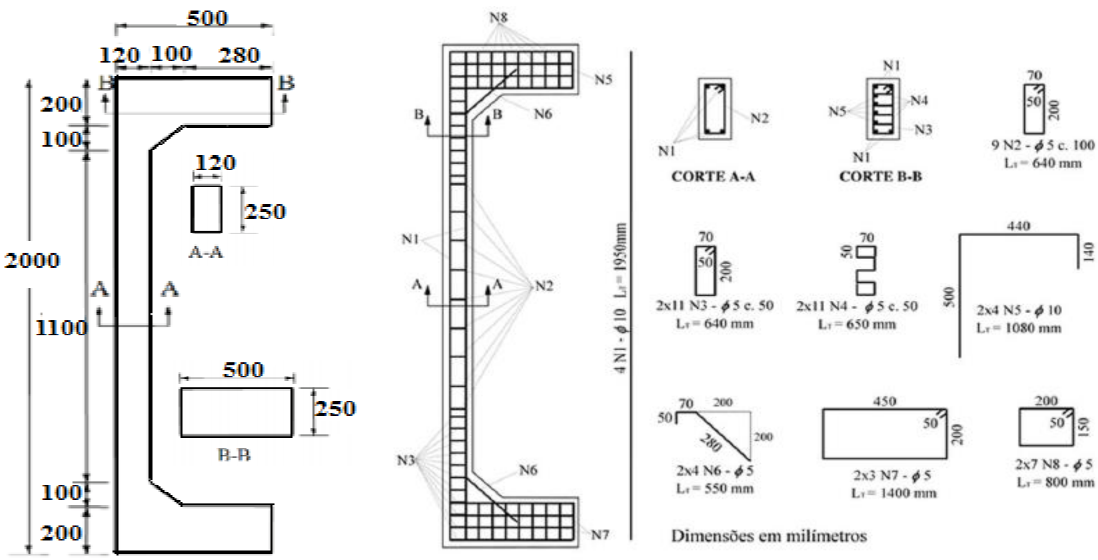
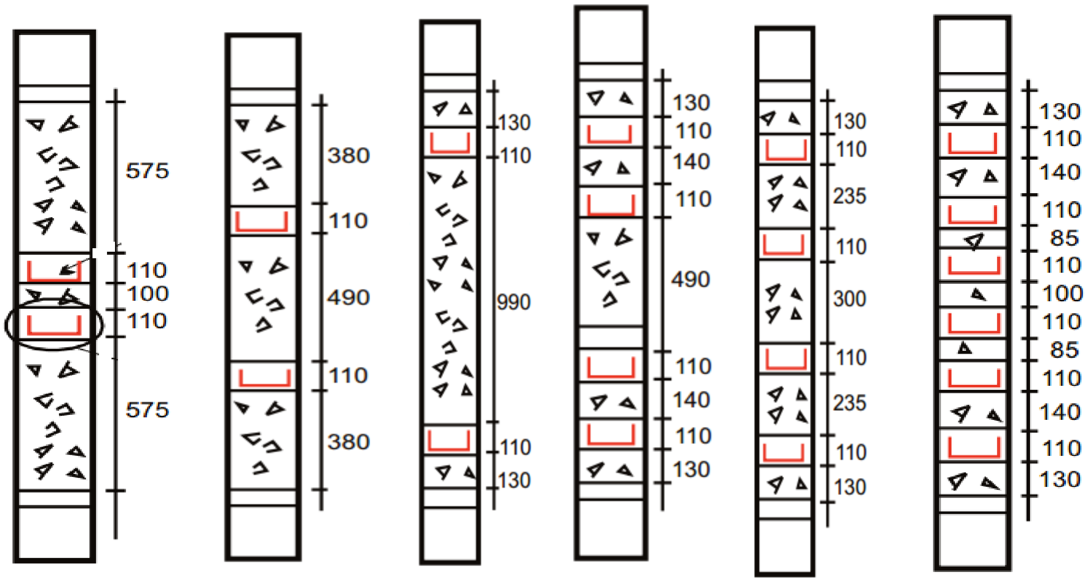


Fig. 2: The Dimensions and RFT details of the reference column [6]

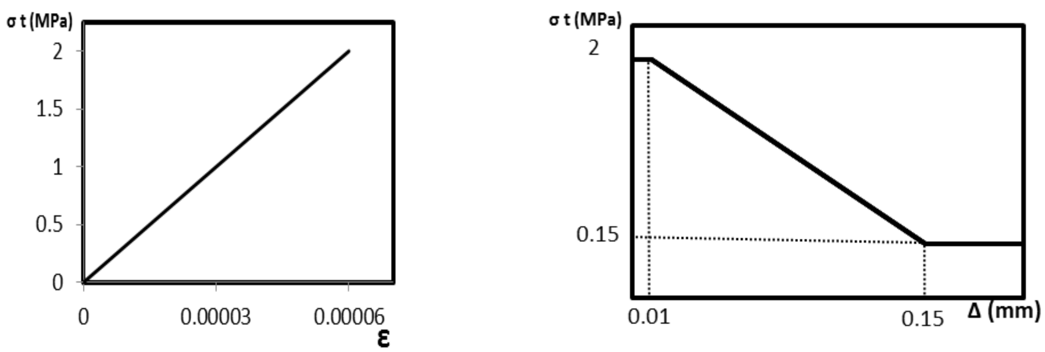


(a) P3 (b) P4 (c) P5 (d) P6 (e) P7 (f) P8

Fig. 3: Location of shear connectors at compressed face of strengthened columns [6]

2.1 Material properties

As described before, the Concrete damage plasticity criterion is used to model the concrete. The cylinder compressive strength, for the reference column (P1) was in the experimental work 42.00 Mpa. The elastic parameters required to establish the first part of the relation are elastic modulus, *Ec*, and tensile strength, *fct*, which can be calculated [10]. To specify the post-peak tension failure behavior of SCC the fracture energy method was used as shown in Fig. 4. The fracture energy for mode I, Gf, is the area under the softening curve. The stress–strain relationship in compression for concrete is represented in Fig. 5. Poisson's ratio was assumed to be 0.2. The elastic modulus, Es, and yield stress, σy, was in the experimental work for the steel reinforcement Es= 225.3 GPa and σy = 612 MPa. A Poisson's ratio of 0.3 was used for the steel reinforcement. A perfect bond between steel reinforcement and concrete was assumed.



(a) Stress-strain relationship under uni-axial tension. (b) Post-peak stress deformation relationship under uni-axial tension.

Fig. 4: Concrete stress strain relationship under uniaxial tension

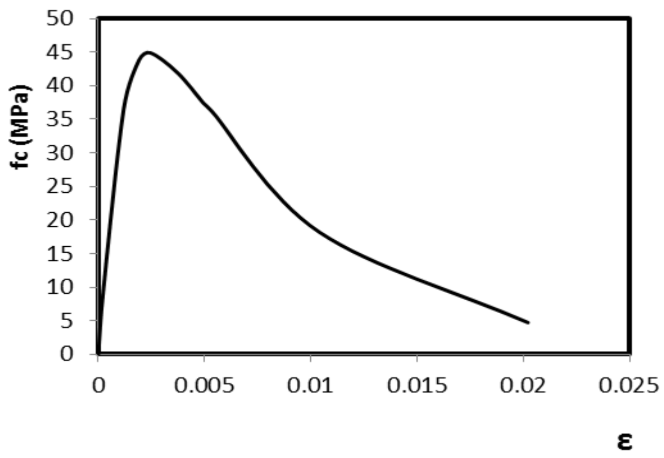


Fig. 5: Stress–strain relationship for concrete under uni-axial compression

2.2 Applied loads

In the experimental work, all the columns were tested under eccentric loading. The hinge support placed on upper and lower of the loading parts to put the load eccentricity and remained in placement by cap head screws [6]. All columns in the finite element model are simulated considering the advantage of symmetry (only one-half of columns) across their entire length to reduce computational time with the applied boundary conditions as shown in Fig. 6.

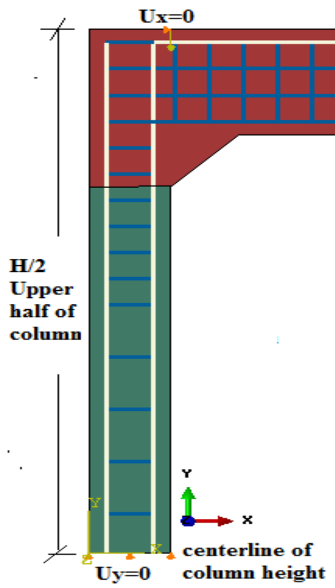


Fig. 6: Applied boundary conditions on the FE model

2.3 Traction-Separation Behavior

Using surface-based cohesive behavior which is primarily intended for situations in which the interface thickness is negligibly small ABAQUS [7]. The interface thickness is considered negligibly small, and the initial stiffness Knn, Kss, and Ktt, and in the normal and two shear directions respectively defined as [10-12]. The values used for this study were, the maximum shear stress, τmax was taken 2.5 MPa [10]. For the maximum normal stress, it was taken equal to the concrete tensile strength 3.00 MPa. Interface damage evolution was expressed in terms of energy release. The description of this model is available in the Abaqus material library [7]. The dependence of the fracture energy on the mode mix was defined based on the Benzaggah–Kenane fracture criterion [7]. For the fracture energy, Gcr in the two shear directions, previous researches have indicated values from 300J/m2 up to 1500 J/m2 [23]. The value used for the fracture energy, Gcr in the normal direction equals 800 J/m2 [13].

2.3.1 Shear connectors

To define the shear connectors' property in the present model, the normal and tangential mechanical behavior must be defined. For the normal behavior, one can define spring-like elasticity behavior for the available components of relative motion. The values used for this study were, the spring stiffness (shear connectors), D11 was taken 225.3 GPa [6]. For the tangential behavior, the U2 direction was defined as the slip direction (perpendicular to U1 direction), and defined the tangential behavior using the penalty friction formulation with a friction coefficient equal to 1.01 as specified in [14].

4. Comparison of experimental and finite elements results

4.1 Results of the reference column

The load vs. mid height lateral displacement obtained from the reference column (P1) from experimental and FEM analysis are shown in Fig. 7. Also Fig. 8 shows the yielding zones of the longitudinal RFT of the reference column (P1). It shows a good agreement between FEM and experimental results for the reference column (P1). The good agreement indicates that the constitutive models used for concrete and reinforcement can reasonably capture the mechanical behavior.

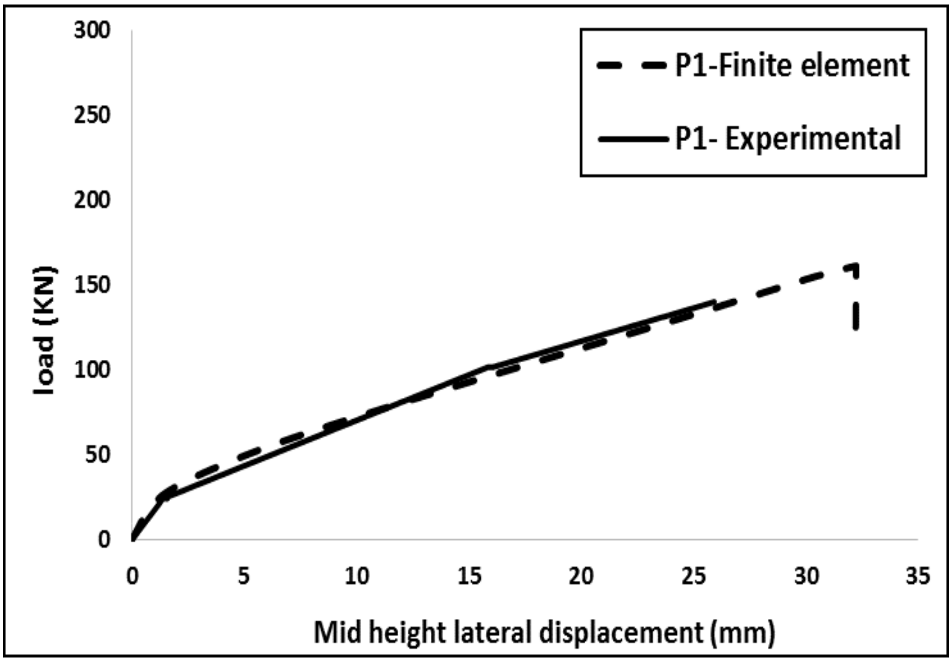


Fig. 7: Load vs. mid height lateral displacement for reference column (P1)

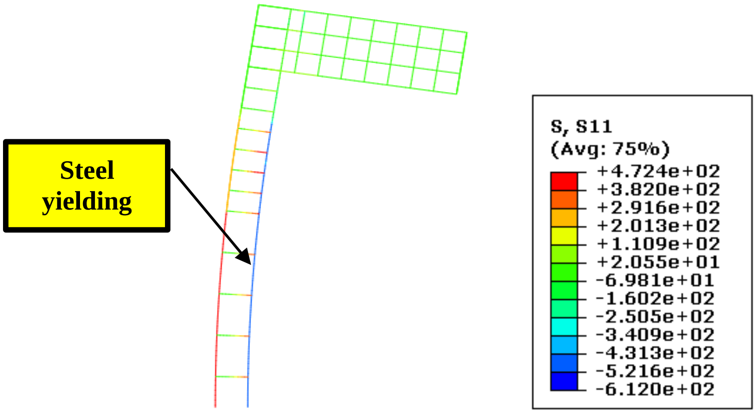


Fig. 8: Yielding stress zones in the RFT steel of reference column (P1)

4.2 Results of the strengthened columns

Fig. 9 illustrates the force displacement diagram relationship of the strengthened columns predicted by the adopted numerical model and the experimental results. It is shown that the FE analysis predicted the peak load quite accurately. Fig. 10 illustrates a simulated failure mode of the strengthened columns. It is clear from the figure, that the comparison is very satisfactory indicating significant matching between the experimental and numerical models. It is clear from the above comparisons that the FE model **(2D plane stress element)** could simulate, with acceptable accuracy, the eccentric behavior of RC columns strengthened with RC jacket.

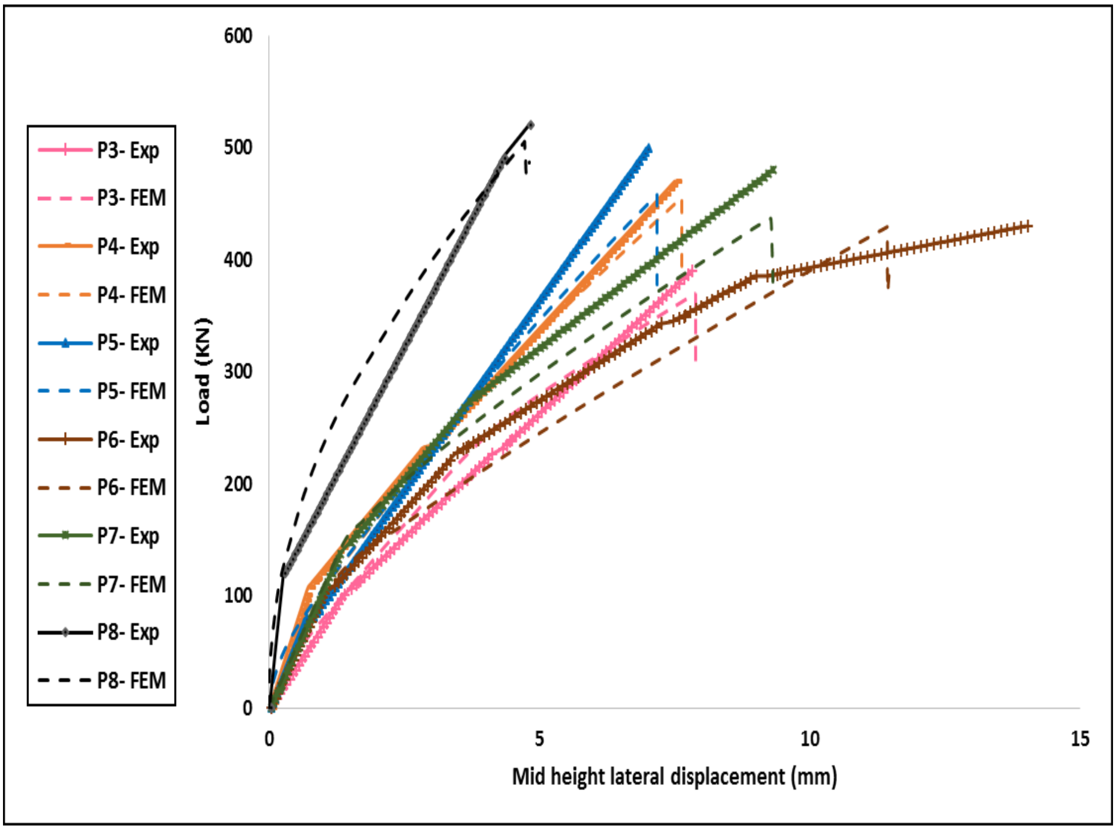
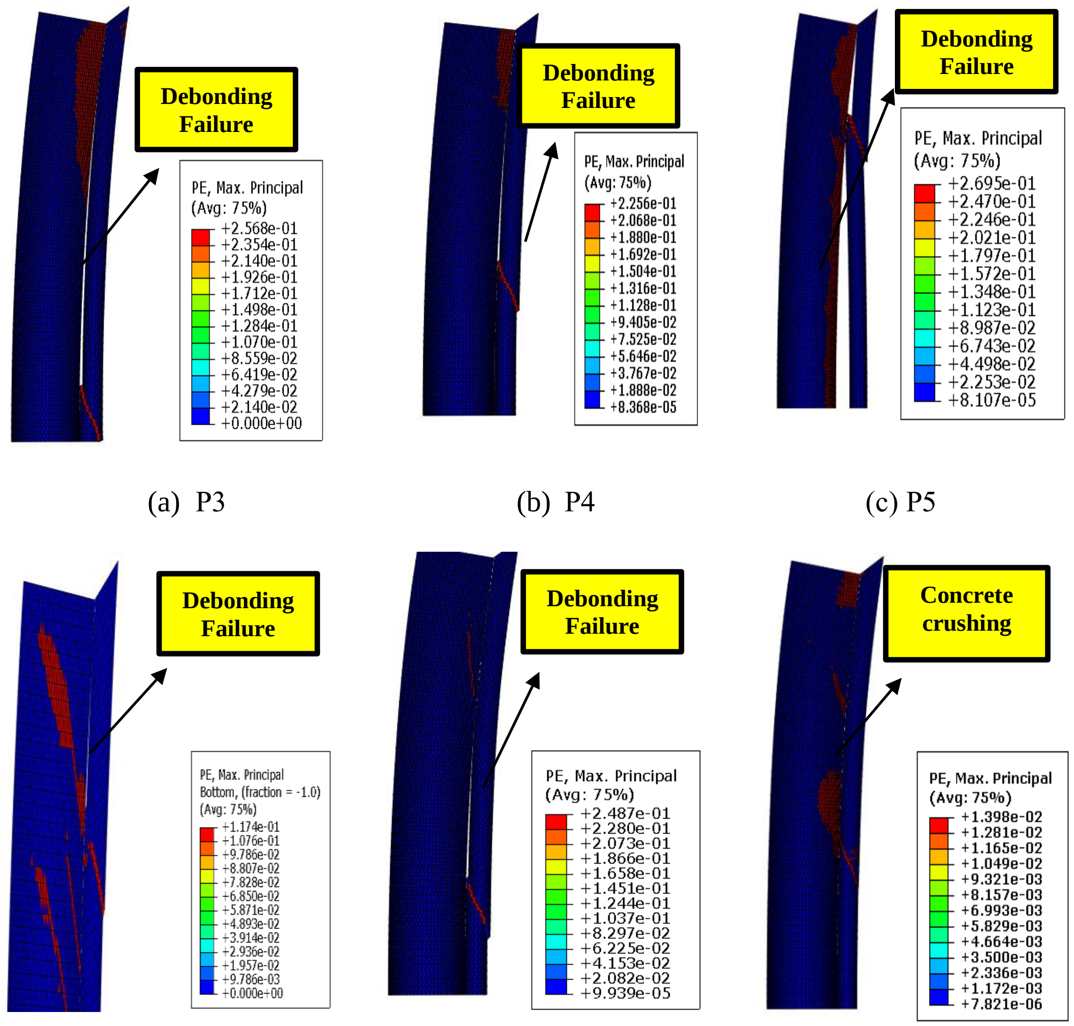


Fig. 9: Load vs. mid height lateral displacement for strengthened columns



(a) P3 (b) P4 (c) P5

(d) P6 (e) P7 (f) P8

Fig. 10: Plastic strain pattern represents cracks (2D Plane stress model) for strengthened columns

5. PARAMETRIC ANALYSIS

After the calibration of the numerical model, it was decided to make a parametric analysis to evaluate the effect of assuming perfect bond at the interface between old and new concrete on the behavior of strengthened columns and compare the results with the experimental and FE model for strengthened column (P8). The results show that the perfect bond model overestimates the stiffness at the intermediate loading stage and the ultimate load and also another mode of failure occurs, where the cohesive model proved able to represent more accurately the bond behavior at the interface between old and new concrete.

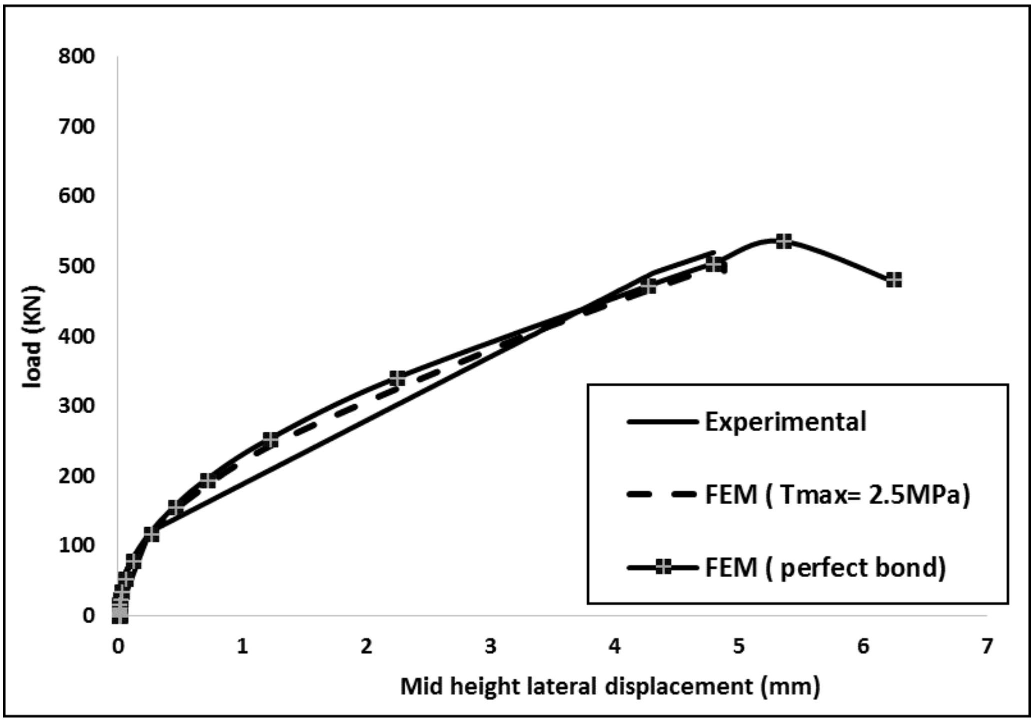


Fig. 11: load vs mid height lateral displacement for strengthened column (P8)

6. Conclusions

In this study, the behavior of RC columns strengthened with RC jacket was studied numerically using a finite element 2D shell model. The interaction at the interface between old and new concrete was modelled. To allow the occurrence of debonding mode of failure, a cohesive surface-to-surface interaction model was used to model the adhesive layer. Based on the previous results, the following conclusions can be drawn:

1. The results of the 2D finite element model in the current study showed good agreement with the experimental results available in literature to predict accurately the behavior of strengthened columns and capture the failure mechanisms of the specimens. Also, the failure load of the specimens has estimated fairly well.
2. The perfect bond model did not succeed in the representation of the behavior of the strengthened RC columns. While the cohesive models have the same pattern of collapse as in the experimental work.

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