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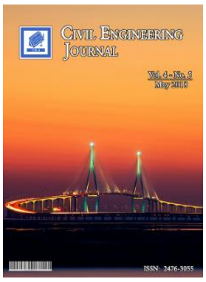
Civil Engineering Journal

Vol. 4, No. 5, May, 2018



Finite Element Modeling of a Reinforced Concrete Column Strengthened with Steel Jacket

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Received 30 March 2018; Accepted 20 May 2018

Abstract

The reinforced concrete column is designed to have a nominal axial resistance. Under different conditions like errors in design, and changing the use of the building from residential to public or storage (extra live loads), the reinforced concrete column will not be able to sustain the desired applied load, and the strengthening is required. This paper presents a finite element model to simulate and investigate the behavior of adding steel jacket to a preloaded and non-damaged reinforced concrete column. Depending on the loading state of the non-strengthened reinforced concrete column and the purpose of adding the steel jacket, two possible cases have been studied. In the first case, which is suitable to investigate the reinforced concrete column with design errors, the steel jacket has been added to the unloaded reinforced concrete column; while the second case is suitable for adding steel jacket to the pre-loaded non-damaged reinforced concrete column. The finite element model was carried out using the ABAQUS/standard v. 6.13 software. The results obtained by the proposed finite element model showed fairly good agreement with the existing experimental and analytical results.

Keywords: Reinforced Concrete Column; Nominal Axial Resistance; Steel Jacket; Finite Element Model.

1. Introduction

At present, rehabilitation considers as one of the most important and widespread aspects of civil engineering. Rehabilitation is a process, which is used to bring the deficient structure or any structural component to the pre-established performance level. Two main categories can be noticed in rehabilitation: repairing and strengthening. This study will focus on the strengthening category. Strengthening is defined as the increase in the current capacity of the non-damaged structural component to another specified level. For reinforced concrete (RC) columns, strengthening with steel jacket consists of four longitudinal angles and horizontal strips is the most effective and used technique which has been used in this study.

Most of the experimental and analytical studies investigated the behavior of RC columns strengthened with steel jacket under concentric and eccentric axial load, but all these studies assumed the loading of the RC column and steel jacket at the same time, i.e. the load applied on the RC column before adding of steel jacket is zero. This paper presents a non-linear finite element model to simulate and investigate the behavior of adding steel jacket to a preloaded and non-damaged RC column, by using the deactivated and reactivated techniques and stepped loading stages, to simulate the strengthening process of an existing RC column. The presented model will be verified with the experimental and analytical results.

2. Literature Review

Garzón et al. 2012 [1] presented a finite element model of a RC column strengthened with steel caging subjected to

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http://dx.doi.org/10.28991/cej-0309144

bending moments and axial loads. The model is used to obtain the N-M diagrams, studying the difference between fitting and not capitals at the end of the strengthened RC column, next to the beam-column joint. In addition, the model is used to perform a parametric study in which it is investigated the influence of several parameters.

Campione 2013 [2] proposed simple analytical equations on the basis of constitutive laws of confined concrete and steel angles to compute the moment-axial forced domain of a R.C. column externally strengthened with steel angles at the four corners and strips. The Comparison with experimental results showed good agreement.

Tarabia and Albakry 2014 [3] conducted a study on the effect of some parameters related to the strengthening steel cage on seismically deficient RC columns. Size of steel angles, size and spacing of batten plates, type of bonding grout between the RC concrete column and the steel angles, and the connection between the head of column and steel angles are the parameters that have been studied. Tested results showed that the strengthening system improved the load carrying capacity of tested specimens and using of battens increases the ductility of strengthened specimens due to confinement effects.

Khalifa and Al-Tersawy 2014 [4] presented an experimental investigation on RC columns strengthened with steel jacket. During the test, it was observed that the failure type in control specimen was a compression failure and for strengthened specimens the failure was occurred when the steel cage did not have the ability to confine the concrete. Test results showed that strengthening techniques have been increased the axial load resistance and increased the ductility compared with the control specimen.

Hoque et al. 2015 [5] evaluated the axial load capacity of RC columns made with brick aggregate concrete strengthened by steel jackets. The test results indicated that the load capacity increases with decreasing strip spacing and increasing the area of the jacket.

Cavaleri et al. 2016 [6] presented a selected review of literature on models of confinement for concrete specimens with steel jacketed. They presented a parametric study in which the main confinement parameters predictable by each of models were compared.

Ezz-Eldeen 2016 [7] studied the performance of rectangular RC columns strengthened with steel angles and battens. Tested specimens were subjected to an eccentric axial load until failure. Different sizes of steel angles have been used in this study. Tested results showed that the load carrying capacity of strengthened columns increases when the cross-sectional of angles used increased as well as increasing the coverage area of the strengthening system.

A study intends to investigate the performance and behavior of RC columns strengthened with steel jackets under concentric and eccentric axial loads is presented by Debasish 2017 [8]. An experimental program has been designed to identify the behavior of the strengthened RC columns under various levels of load eccentricity. The experimental model has been followed by a finite element model using ABAQUS software. In finite element model, both material and geometric nonlinearities were considered. The finite element models show fair agreement with the observed experimental results in terms of ultimate capacity and failure mode. Tested results showed that adding of a steel jacket will improve the behavior of RC columns under of concentric and eccentric axial loads.

Al-Sherrawi and Salman 2017 [9] presented two analytical models to construct the axial load-bending moment interaction diagram of an RC column strengthened with a steel jacket. The derivation of expressions was made by assuming equivalent stress block parameters for confined concrete. The proposed models show good agreements with available experimental data and design proposals.

Al-Sherrawi and Salman 2017 [10] presented an analytical model for the hand computation to construct the load-bending moment interaction diagram for a RC column strengthened with steel jacket using the plastic stress distribution method, by assuming the strengthened column behaving as a composite column. The results obtained by the analytical model showed fairly good agreement with the experimental results.

3. Methodology

RC column is designed to have a nominal axial resistance (N_n) depends on column cross-section, compressive strength of concrete, reinforcement bars size, and yield stress of reinforcement. This resistance must be adequate to sustain the applied load. For many reasons such as errors in design, and changing the use of the building from residential to public or storage (extra live loads), N_n will not be able to sustain the applied load and the strengthening is required. When the RC column is loaded with a certain value of a load, which equals to $P\%\ N_n$. $P\%$ is between 0 to 80%. After that the steel jacket will be added and the behavior of strengthened RC column will depend on the value of $P\%$. Two possible cases of adding the steel jacket will be discussed in this study:

- Case I: P% = 0%

When a RC column was designed to have a nominal axial resistance (N_{n1}) and due to design errors, N_{n1}decreases to N_{n2}, the RC column will not be able to sustain the desired applied load. In this case, the RC column is not loaded with any amount of loads (the stresses in the concrete and reinforcement bars are equal to zero). The steel jacket will be added to improve the behavior of the RC. The load will be applied to the RC column and transmitted to the steel jacket due to the direct contact between concrete and steel.

- Case II: P% > 0%

When a RC column was designed to have a nominal axial resistance (N_n) and loaded with N_1 ($P\% \, N_n$). Due to changing the use of the building from residential to public or storage (extra live loads), N_1 will increase to N_2. The RC column will not be able to sustain the additional load (N_a). In this case, the steel jacket will be added to improve the behavior of the RC. The RC column will be unloaded from N_1, then the steel jacket will be added and the strengthened RC column will be loaded by N_2 from the beginning.

4. Finite Element Model

The finite element method (FEM) model was carried out using the ABAQUS v. 6.13 [11] software. In order to accurately simulate the behavior of a RC column strengthened with a steel jacket, the FEM model took into consideration the second-order geometric effects, the non-linear behavior of the concrete and steel (in both steel jacket and reinforcing bars) and the existence of an RC-steel jacket interface.

4.1. Geometry

The concrete part, the steel section part, the batten part and the reinforcements part (longitudinal part and ties part) were done separately. The concrete part, the steel section part and the batten part were done as three dimensional (3D) deformable solid elements, and the elements type was hexahedral element with eight nodes and three degrees of freedom per node, with reduced integration and hourglass control, C3D8R. The reinforcements part done as 3D deformable wire elements, and the elements type was two-node truss element linear displacement, which can transmit only axial force, T2D3. These parts have been merged together in the assembly module, as shown in Figure 1.

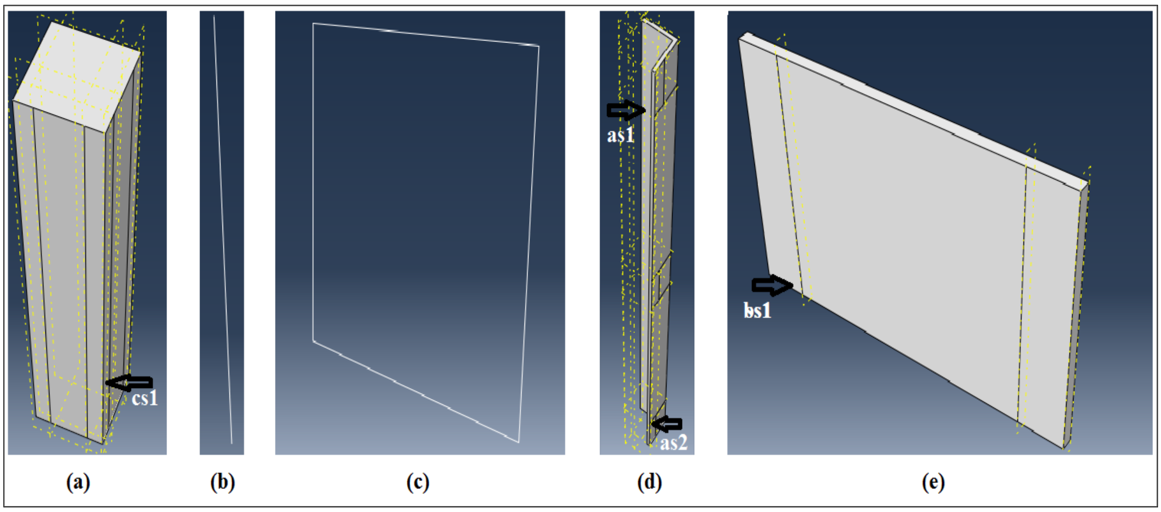


Figure 1. FEM parts: (a) Concrete part, (b) Longitudinal bars part, (c) Tie part, (d) Steel section part, (e) Batten part

4.2. Materials

The concrete was modeled with the Concrete Damage Plasticity (CDP) model. The CDP model assumes that the two main concrete failure mechanisms are cracking and crushing. Crack propagation is modeled by using continuum damage mechanics, stiffness degradation. The CDP model requires the values of elastic modulus, Poisson's ratio, the plastic damage parameters and description of compressive and tensile behavior. The five plastic damage parameters are the dilation angle, the flow potential eccentricity, the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress, the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian and the viscosity parameter that defines viscoplastic regularization. The values of these parameters were assumed to be 36o, 0.1, 1.16, 0.667, and 0, respectively.

The stress–strain relationship proposed by Saenz (1964) [12] was used to construct the uni-axial compressive stress–strain curve for the concrete. Initially, the linear elastic portion is defined using the modulus of elasticity in compression. According to Eurocode No.2 (2004) [13], the proportional limit or elastic limit for a normal strength concrete is assumed to be 40% of its compressive strength. The stress–strain relationship is:

$$\sigma_c = \frac{E_c \varepsilon_c}{1 + (R + R_E - 2)\left(\frac{\varepsilon_c}{\varepsilon_0}\right) - (2R - 1)\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2 + R\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^3} \qquad (1)$$

$$R = \frac{R_E(R_\sigma - 1)}{(R_\varepsilon - 1)^2} - \frac{1}{R_\varepsilon}$$

$$R_E = \frac{E_c}{E_0}$$

$$E_0 = \frac{f_c}{\varepsilon_0}$$

Where R_σ and R_ε can be assumed as 4, E_c is the concrete elastic modulus:

$$E_c = 4700\sqrt{f_c} \tag{2}$$

And ε_0 is the concrete strain corresponding to f_c and obtained from presented model by Almusallam and Alsayed (1995) [14]:

$$\varepsilon_0 = (0.2f_{cu} + 13.06) \times 10^{-4} \tag{3}$$

Where f_{cu} is the cubic concrete compressive strength.

The ultimate compressive strain in the concrete was assumed to be 0.003 and Poisson's ratio was assumed to be 0.16. The model presented by Wang and Hsu (2001) [15] was used to construct the tensile stress–strain curve for concrete:

For $\varepsilon_t \leq \varepsilon_{cr}$

$$\sigma_t = E_c\varepsilon_t \tag{4}$$

For $\varepsilon_t > \varepsilon_{cr}$

$$\sigma_t = f_c\left(\frac{\varepsilon_{cr}}{\varepsilon_t}\right)^{0.4} \tag{5}$$

Where ε_{cr} is the cracking strain and can be calculated by using the relation below:

$$\varepsilon_{cr} = \frac{f_{ct}}{E_c}$$

Where f_{ct} is the tensile strength for concrete, Yasmeen (2011) [16]:

$$f_{ct} = 0.33\sqrt{f_c} \tag{6}$$

The constitutive model used to simulate the steel was the classical metal elastic-plastic model with strain hardening. The input for the steel model includes elastic modulus, Poisson's ratio, yield stress and plastic strain. The model proposed by Eurocode No.2 (2004) [13] is used to construct the stress-strain relationship for steel in both tension and compression. The elastic modulus was assumed to be 200000 MPa and Poisson's ratio as 0.3.

4.3. Interfaces

The "embedded region" function of ABAQUS was used to simulate the interface between the concrete and reinforcement bars, while the "Tie" function of ABAQUS was used to simulate the interface between the vertical steel angles and horizontal strips, and also the interface between the steel jacket and the concrete.

4.4. Applying of Loads

For case I, one loading step was used. In this loading step, the load has been applied on the concrete and transmitted to the steel jacket until failure occurs. For case II, when the steel jacket (angles and strips) has been added to the preloaded column, three loading steps have been used. In the first loading step, the steel jacket part has been deactivated and the load N_1 has been applied to the concrete only, then a restart model has been used. The restart model has the same information of the original model. The restart model will read the results obtained by applying the first loading step in the original model. In the restart model, a second loading step will be applied. The purpose of this loading step is to unload the load applied in the first loading step. The steel jacket part will remain deactivated at the second loading step. The third loading step follows the second load step and the steel jacket part will be reactivated at this step and the new amount of load (N_2) will be applied to the concrete and this load will be transmitted to the steel jacket until failure.

5. Validation of FEM

A set of experimental investigations presented by Tarabia and Albakry 2014 [3] named (SCN1), and Ezz-Eldeen 2016 [7] named (CS22e0) were used to validate the presented FEM model. The Details of these two specimens are illustrated in Table 1 and the FEM models are shown in Figure 2. The FEM model results for both of case I and case II were discussed in this section. These results will be compared with the experimental results if exist and also compared with the results obtained by the analytical models presented by Al-Sherrawi and Salman 2017 [9] and Al-Sherrawi and Salman 2017 [10].

Table 1. Details of specimens used

Specimen	Cross-section (mm)	Length (mm)	Longitudinal bars	Tie	Steel section size (mm)	Steel strip size (mm)	Clear strip Spacing (mm)	f_c (MPa)	f_{yr} (MPa)	f_{yan} (MPa)	f_{yst} (MPa)
SCN1	150 × 150	1000	4 ϕ10 mm	ϕ6 mm @ 100 mm	4 L 50 ×4.5	150 × 50 × 5	120	46.25	420	415	415
CS22e0	120 × 160	1000	4 ϕ8 mm	ϕ6 mm @ 120 mm	4 L 20 × 2	120 × 20 × 2	220	28	260	380	380

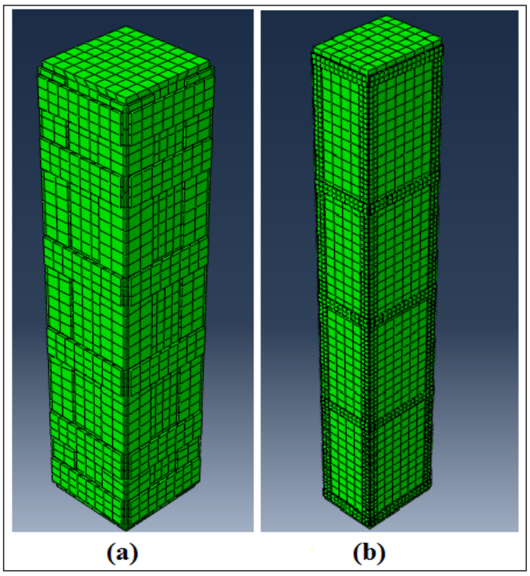


Figure 2. FEM specimen geometry: (a) SCN1 specimen, (b) CS22e0 specimen

- Case I

As shown in Figure 3, the axial load resistance obtained from the FEM gives good agreement when compared with the experimental and analytical results for both of SCN1 specimen and CS22e0 specimen. The analytical results came from applying of the two analytical models presented by [9] (Strain M1 and Strain M2) and the analytical model presented by [10] (Plastic model).

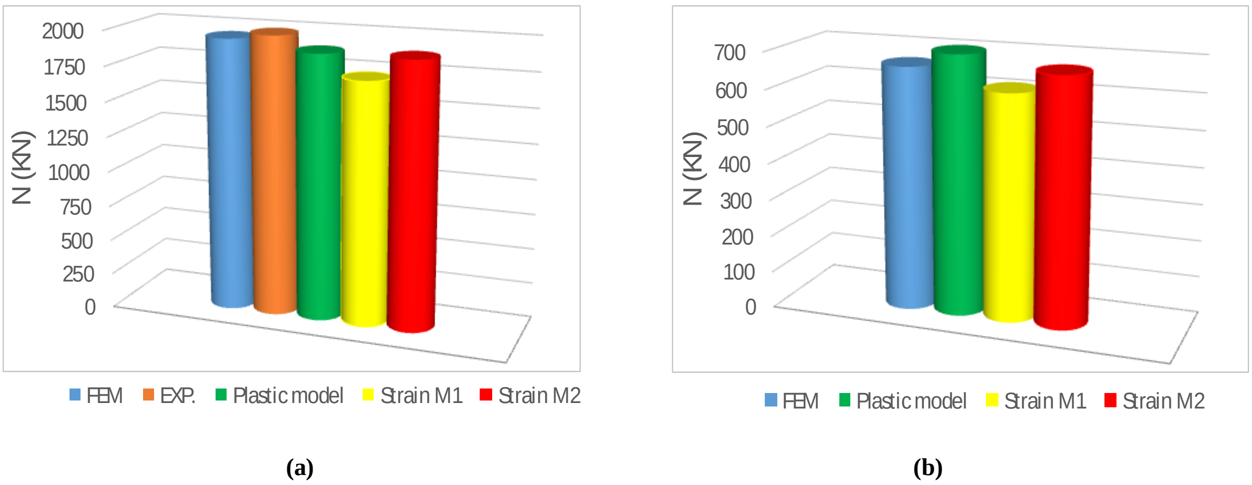


Figure 3. FEM results for case I: (a) SCN1 specimen, (b) CS22e0 specimen

Figures 4 and 5 show the stresses in the concrete, steel jacket, and reinforcement when the failure occurred. It can be noticed that and due to confinement effects, the compressive strength increases from 46.25 to 60.88 MPa for SCN1 specimen and from 28 to 30.23 MPa for CS22e0 specimen, and these values give good agreement when compared with the analytical confined compressive strength of concrete obtained by Al-Sherrawi and Salman 2017 [9] and [10].

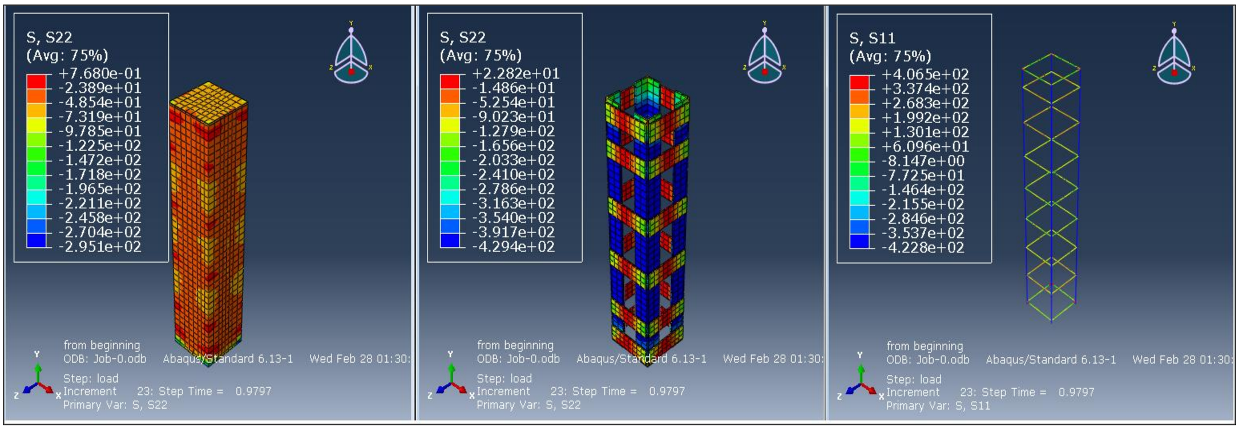


Figure 4. Stresses in the concrete, steel jacket, and reinforcement when the failure occurred for SCN1 specimen

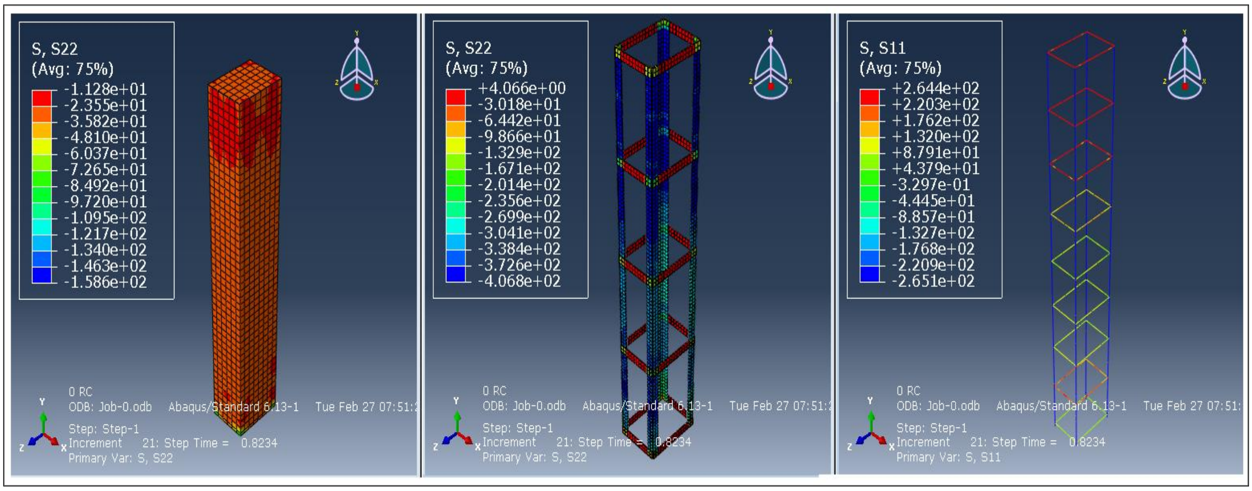
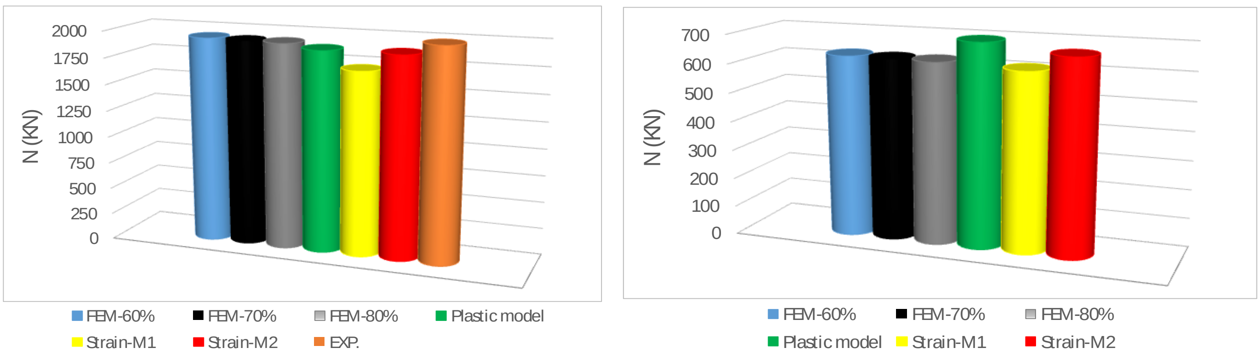


Figure 5. Stresses in the concrete, steel jacket, and reinforcement when the failure occurred for CS22e0 specimen

For SCN1 specimen, and due to the little difference between the yield stress of the reinforcing bars and the steel angles, both of the reinforcing bars and the steel angles reach its yield stress at the same applied load, which is 1920.6 kN. After this point, the reinforcing bars, and steel angles enter the plastic zone. The increase in the axial resistance for the strengthened RC column through the plastic zone until the failure occurs was 1.5%. For CS22e0 specimen, the reinforcing bars reach its yield stress at the applied load 616.9 kN, while the steel angles reach its yield stress at the applied load 662 kN. So, the reinforcing bars enter the plastic zone before the steel angles, and the increase in axial resistance for the strengthened RC column after yielding of reinforcing bars is 7.3%, and after yielding of steel angles until the failure occurs is 0.3%.

- Case II

As shown in Figure 6, the axial load resistance obtained from the FEM gives good agreement when compared with the experimental and analytical results for both of SCN1 specimen (Figure 6-a) and CS22e0 specimen (Figure 6-b).



(a) (b)

Figure 6. FEM results for case II: (a) SCN1 specimen, (b) CS22e0 specimen

Figure 7 shows the stresses in the concrete and the reinforcing bars at the end of the first loading step (before unloading) and at the end of the second loading step (after unloading) for the case of P% equals 60% for SCN1 specimen. After completing of unloading process, residual stresses will be remained in the concrete and the reinforcing bars, as shown in Figure 7. These residual stresses will be compressive stresses in some concrete regions and tensile stresses in other regions. The tensile stresses that will be generated in the concrete will follow the tensile stress-strain curve used in the definition of concrete material in ABAQUS. The stresses in concrete at first loading step for P% equals 60%, 70%, and 80% exceed the elastic stress in both of SCN1 specimen and CS22e0 specimen. In the unloading process, the stress and strain in the concrete will be decreased but due to the non-linear behavior of concrete, the stress and strain will not back to zero as shown in Figures 8 and 9. Increasing the applied load in the first loading step will increase the remaining strain in the concrete at the end of unloading step and that will reduce the final resistance produce by the concrete in the third loading step.

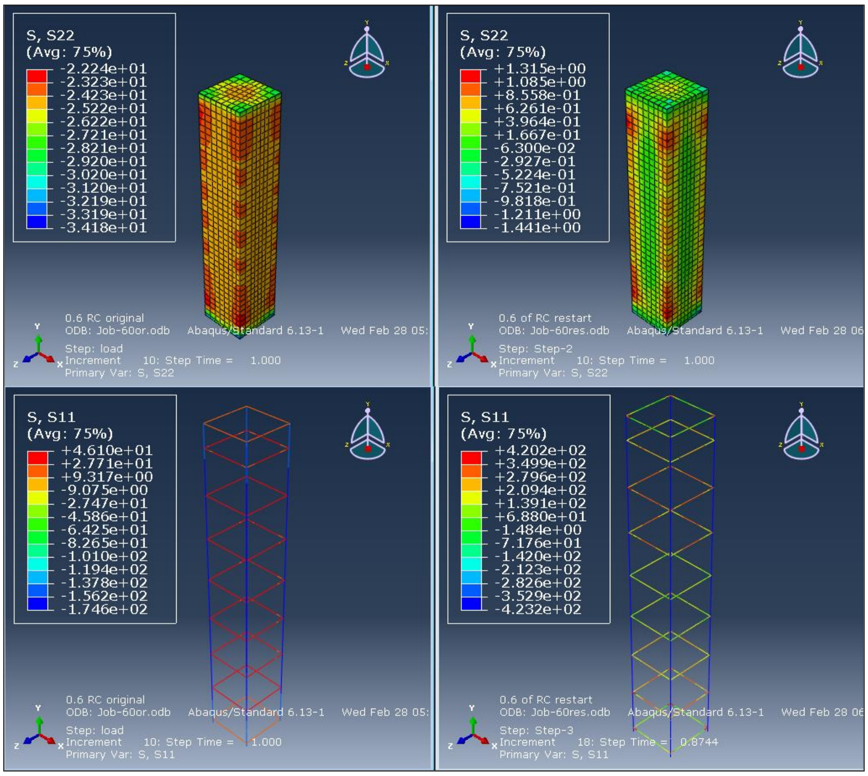


Figure 7. Stresses in concrete and reinforcing bars before and after the unloading process for the case of 60% for SCN1 specimen

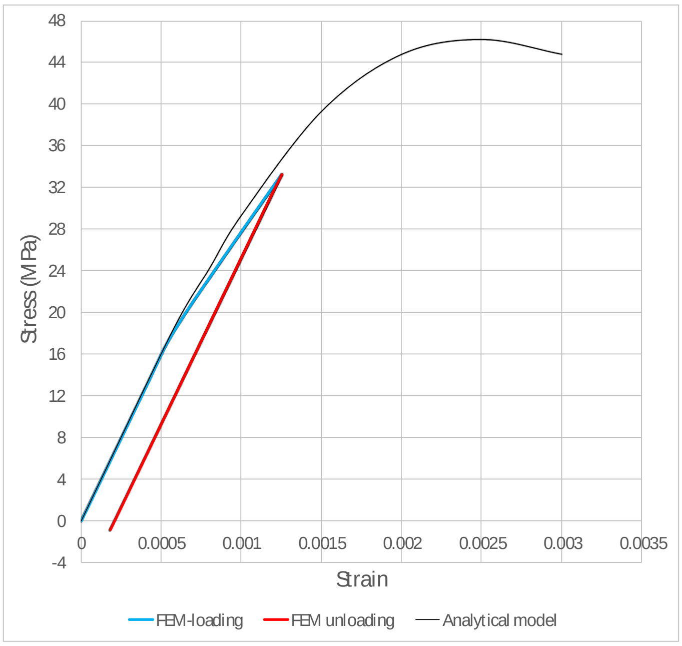


Figure 8. FEM loading, unloading, and analytical concrete stress-strain curves for SCN1 specimen for P% = 80%

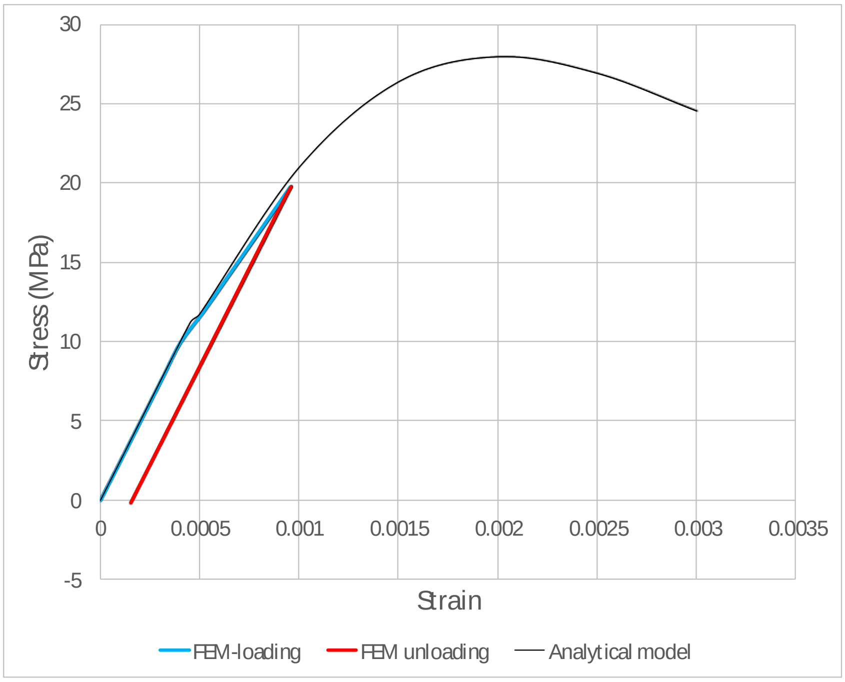


Figure 9. FEM loading, unloading, and analytical concrete stress-strain curves for CS22e0 specimen for P% = 80%

For the reinforcing bars (Figures 10 and 11) and at the first loading step, the stresses and strains are within the elastic range and did not reach its yield stress and strain. In the unloading process, these stresses and strains will be decreased, but even though it were on the elastic range theses stresses and strains will not back to zero because of the interaction "embedded region" between the concrete and the reinforcing bars. Residual stresses will be remained in the reinforcing bars, as shown in Figure 7. These residual stresses will be compressive stresses in some bars and tensile stresses in other bars. Increasing the applied load in the first loading step will increase the remaining residual stresses in the reinforcing bars at the end of unloading process, and that will reduce the resistance produced by the reinforcing bars.

Finally, in the third loading step (applying of N_2), the Pre-existing stresses (residual stresses in the concrete and the reinforcing bars) and strains will cause a reduction in axial resistance of the strengthened RC column.

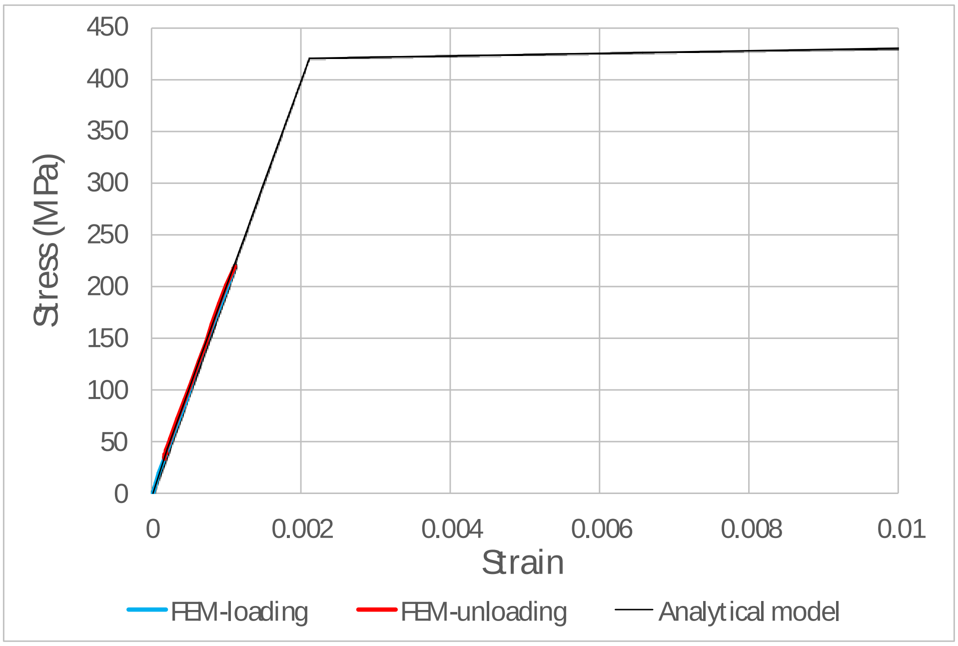


Figure 10. FEM loading, unloading, and analytical steel stress-strain curves for SCN1 specimen for P% = 80%

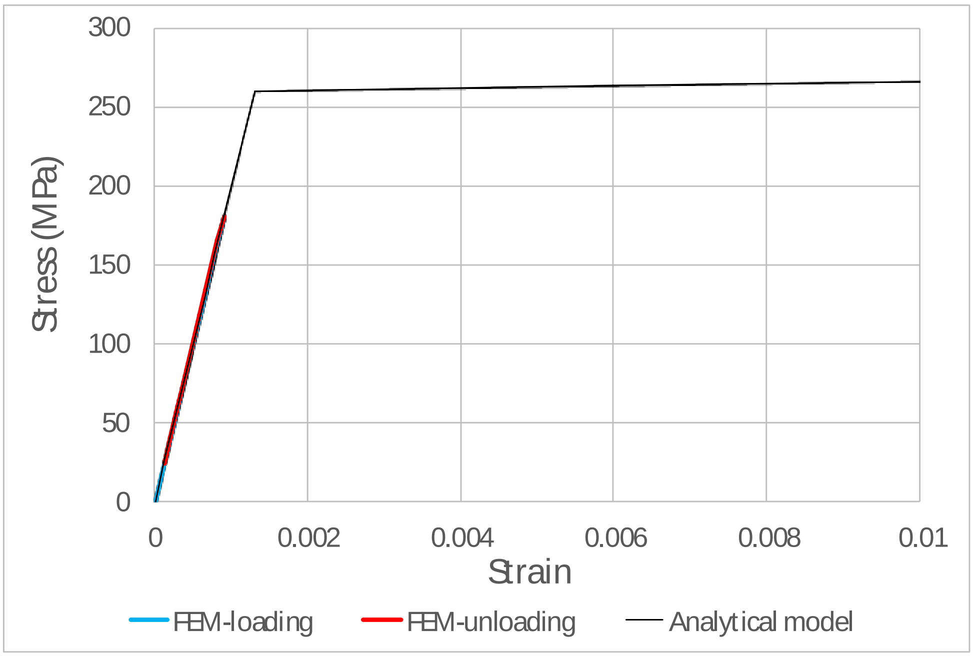


Figure 1. FEM loading, unloading, and analytical steel stress-strain curves for CS22e0 specimen for P% = 80%

Table 2 illustrates the compressive strength of the concrete at the end of third loading step, and the values of the load in which the reinforcing bars and the steel angles reach its yield stress for both of SCN1 specimen and CS22e0 specimen and for all different loading stages.

Table 2. Results for Case II

Specimen	60%			70%			80%		
	f_c (MPa)	N_{yr} (kN)	N_{yan} (kN)	f_c (MPa)	N_{yr} (kN)	N_{yan} (kN)	f_c (MPa)	N_{yr} (kN)	N_{yan} (kN)
SCN1	60.47	1800	1800	59.7	1800	1800	59.4	1800	1800
CS22e0	29.2	614.8	629	28.8	596.7	627.3	28.7	584.6	626.5

From Table 2, it noticed that the reinforcing bars and the steel angles in Case II reach its yield stress faster than Case I, because of pre-existing stresses and strains at the end of second loading step (end of unloading process) and the beginning of the third loading step (applying of N_2).

6. Conclusion

In the present work, a finite element model to simulate and investigate the behavior of adding a steel jacket to a preloaded and non-damaged RC column has been introduced. Depending on the state of the non-strengthened RC column and the purpose of adding the steel jacket, two possible cases have been studied. The results obtained by the proposed FEM model showed fairly good agreement with the existing experimental and analytical results. Adding the steel jacket improves the axial resistance of the RC column by increasing the concrete compressive strength due to the confinement effects and sharing the applied loads with the RC column. For case I, the strengthened RC column gives higher axial resistance than in Case II, due to the pre-existing stress and strain in the concrete and the reinforcing bars produced by the unloading process. It noticed that the pre-existing stresses increase with increasing the percentage of loading the RC column before adding the steel jacket. The pre-existing stress and strain were considered as a loss in the original stress and strain of the component (concrete and reinforcing bars) and that causes a reduction in the resistance produced by the component itself, thus a reduction yields in the resistance of the strengthened RC column.

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