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Numerical investigation of the hysteretic response analysis and damage assessment of RC column

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Abstract. The Finite Element (FE) modeling of Reinforced Concrete (RC) under seismic loading has a sensitive impact in terms of getting good contribution compared to experimental results. Several idealized model types for simulating the nonlinear response have been developed based on the plasticity distribution alone the model. The Continuum Models are the most used category of modeling, to understand the seismic behavior of structural elements in terms of their components, cracking patterns, hysteretic response, and failure mechanisms. However, the material modeling, contact and nonlinear analysis strategy are highly complex due to the joint operation of concrete and steel. This paper presents a numerical simulation of a chosen RC column under monotonic and cyclic loading using the FE Abaqus, to assess the hysteretic response and failure mechanisms in the RC columns, where the perfect bonding option is used for the contact between concrete and steel. While results of the numerical study under cyclic loading compared to experimental tests might be unsuccessful due to the lack of bond-slip modeling. The monotonic loading shows a good estimation of the envelope response and deformation components. In addition, this work further demonstrates the advantage and efficiency of the damage distributions since the obtained damage distributions fit the expected results.

Keywords: abaqus; finite element analysis; micro-modeling; nonlinear response; reinforced concrete

1. Introduction

Under strong earthquakes, the elastic range stays limited for the seismic assessment of the new or existing structures, since the nonlinear analysis provides us a good understanding of the structure behavior including the strength and stiffness deterioration associated with the inelastic material behavior. The accuracy of the numerical results compared to those of experimental tests can be extremely sensitive to small variations in the model and input parameters. Several approaches for simulating the nonlinear response have been developed based on the plasticity

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distribution alone the model. Misra and Poorsolhjouy have developed and implemented a micromechanical model for granular material, like concrete (Misra and Poorsolhjouy, Ould Ouali *et al*. 2017). La Borderie *et al*. (2007) were interested in the microscopic approach to concrete by highlighting the contribution of geometric representation. The Concentrated Plasticity Models are the simplest formulation that is based on concentrating the inelastic deformation at a specific location where the damage is expected to be located generally, at the ends of columns and beams. The inelastic concentrated plasticity model first proposed by Clough *et al*. (1965). Since then, different improvements have been carried out over the last 50 years (Gilberson 1967, Takeda *et al*. 1970, Banon *et al*. 1981, Lai *et al* 1984, De Llera and Chopra 1995, Kim and Engelhardt 2000, Ibarra *et al*. 2005). The Distributed Plasticity Models are considered a more advanced type of model since the damage in the reinforced concrete structure is not lumped in a specific zone. Rather, the inelastic deformation is spread along with the element or specific length which will give more accuracy for the estimation of the nonlinear behavior of RC structure. The first models that have been developed for the distribution were by Bazant (1977) and Soleimani *et al*. (1979). Soleimani's model was further improved by Meyer *et al*. (1983) and Roufaiel and Meyer (1987).

A new approach for the Distributed Plasticity Models has been proposed by Filippou and Issa (1988), where the element is divided into a finite number of short sub-elements, and each sub-element describes an inelastic behavior. Kaba and Mahin (1984) had introduced a new concept so-called the Fiber Section Model. This model distributes plasticity by numerical integrations through the member cross-sections and along the member length by several fibers. To capture the nonlinear hysteretic axial stress-strain in the cross sections of each fiber, the uniaxial material models have been used. The Opensees platform is considered one of the most programs base on the fiber section.

The most complex models are the Continuum Models. These models could have a continuum nonlinear hysteretic constitutive property along the member length and through the cross-sections into small (micro) finite elements. By way of example, but not all-inclusive, ABAQUS is one example of a software that implements these continuum models for RC elements. However, the numerical modeling under seismic loading should be done with careful consideration of element types and the formulation used to describe the materials and their contact, as well as the nonlinear analysis strategy.

In this study, the dynamic analysis using ABAQUS/Implicit is adopted to model RC column under Quasi-Static loading (monotonic and cyclic) in order to understand the seismic behavior of the structural element. This concept has been chosen due to the impact and the appropriate strategy for the RC elements. A three dimensional non-linear finite element model for the chosen RC column was modeled. The numerical results were verified than with the experimental test results in terms of load-displacement relationship, damage distributions and failure modes (Bechtoula *et al*. 2005). The results of the experimental hysteretic force-displacement response test have been compared with three different model types for simulating the nonlinear response (the Concentrated Plasticity Model, the Fiber Section Model, and the Continuum Models). In addition, this work demonstrates further the advantages and efficiency of the Continuum Models in terms of damage distributions since the obtained damage distributions should fit the expected experimental results.

2. Finite element modeling

2.1 Constitutive models for concrete

The concrete is modeled with 3D solid elements, and the model available in ABAQUS for this

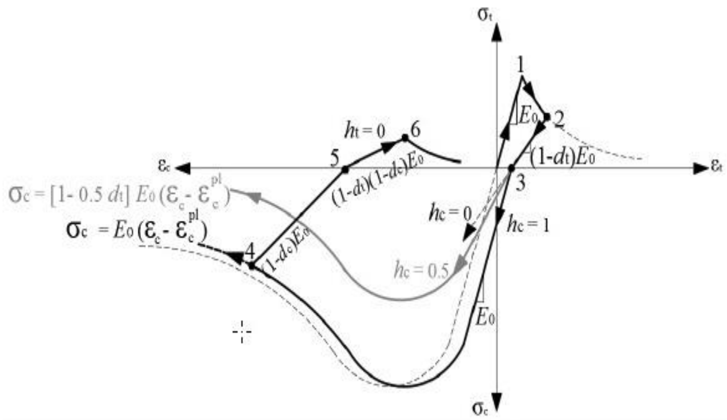


Fig. 1 Uniaxial loading-unloading for the concrete

kind of analysis so-called Concrete Damaged Plasticity Model CDPM, which is based on the formulations proposed by Lubliner *et al*. (1989) and Lee and Fenves (1998). Different researchers have developed and proposed different damage and stress formulations based on splitting damage and stress into compression and tensile parts, and each one determined separately (Hognestad *et al*. 1955, Kent *et al*. 1969-1971, Lubliner *et al*. 1989, Hu and Schnobrich 1989, Carreira and chu 1985, Lee and Fenves 1998, Nayal and Rasheed 2006, Bashar Alfarah *et al*. 2017).

The Lubliner/Lee/Fenves (1989-1998) approach was used in this study. In most of the formulations, the calibration of the damage and strain evaluation needs to be done experimentally. Lee and Fenves (1998) assumed that the stress can be determined by the plastic strain and it takes an exponential form. The relation between the uniaxial stress and plastic strain in both compression and tensile is assumed as:

$$\sigma_x = f_{x0}[(1 + a_x)\,exp(-b_x\varepsilon^p) - a_x\,exp(-2b_x\varepsilon^p)] \tag{1}$$

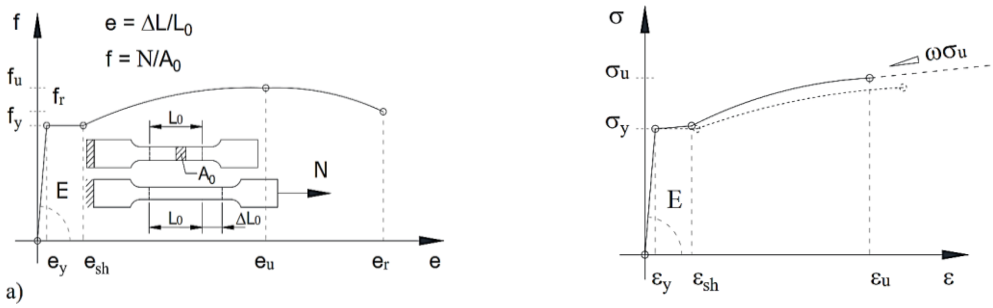
where f_{x0} the initial yields stress, defined as the maximum stress without damage; a_x and b_x are two parameters determined so that this curve reproduces the response of the material. In this approach, the evolution law of the stress in steady softening depends on the size of the finite element. The energy density of cracking g_{fx} is represented by Bažant and Oh (1983), and Rots (1998), which is related to the energy of cracking G_{fx}. Where it is presented as the following equation:

$$g_{fx} = G_{fx}/l_c \tag{2}$$

Assuming that the degradation damage also takes an exponential form as follows (Lee and Fenves 1998):

$$1 - D_x = exp(-d_x\varepsilon^p) \tag{3}$$

Under a cyclic loading the behavior of the concrete becomes more complex and additional parameters should be defined due to the opening and closing of the cracks, as well as the concrete crashing. During the unloading and reloading, two damage variables are defined - h_t and h_c (h_t and h_c ranging between 0 and 1). Factor h_c accounts for reclosing of cracks after tension-compression reversal; ht represents the recovery of crushed concrete after the compression-tension reversal (see



(b) engineering stress-strain curve (b) true stress-strain curve

Fig. 2 Schematic representation of the response of steel under monotonic tensile loading (Zub and Dubina 2019)

Fig. 1). According to the Abaqus Analysis User's Manual a constitutive parameters are needed: dilatation angle, eccentricity, and 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 (ψ , ϵ , fb0 / fc0, K_c).

2.2 Constitutive models for steel

The 2D truss finite element will be used in this study to model the reinforcement bars. For simulations under monotonic loading a rate- independent elasto-plastic model with isotropic hardening can be used. However, when it comes to simulations under cyclic loading, more complex material models are necessary. One of the options is using the combined isotropic-kinematic hardening. Lemaitre & Chaboche (1990) have shown the modeling of metal plasticity under cyclic loading using the combined isotropic-kinematic hardening material. In addition, the Abaqus analysis user's manual required input parameters for both hardening components, these parameters require calibration by the experimental test. Researchers have evaluated different procedures to obtain the input parameters. On the other hand, Abaqus allows also for advanced users to define their own material models using UMAT/VUMAT subroutine. Zub and Dubina (2019) developed a calibration procedure that uses only tensile test data might be used (see Fig. 2), this procedure is available in both Abaqus/Standard and Abaqus/Explicit packages. Another possibility to simulate the nonlinear behavior of steel bars under cyclic loading is to use a linear-kinematic hardening model instead of combined hardening "isotropic and kinematic"; such model requires the hardening parameter to be defined (Bashar Alfarah *et al*. 2017).

2.3 Nonlinear analysis strategy

The dynamic analysis using ABAQUS/Implicit was used in this paper as an incremental-iterative solution strategy to perform nonlinear quasi-static (monotonic or cyclic) analysis of the analyzed model. Where the quasi-static response is generated, and a cyclic or monotonic displacement are applied with a sufficiently slow loading rate that they do not induce dynamic effects (for the implicit option there is the quasi-static option). For this case the nonlinear equation can be linearized by the conventional Newton-Raphson method.

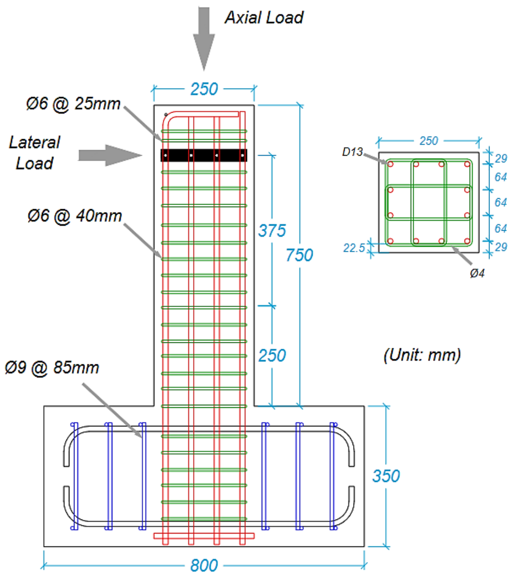


Fig. 3 Specimen dimension of (Bechtoula *et al*. 2005) RC column experiment

Table 1 Present the material characteristics (Bechtoula *et al*. 2005)

Specimen	Specimen Configuration				
	Column width D (mm)	Shear span L (mm)	Concrete Strength (mm)	Longitudinal Rebar (ratio) [Fy] MPa	Shear Rebar (ratio) [Fy] MPa
D1N3 (0.3 f'c/Ag)	250	625	37.6	12-D13 (2.44%) 461	Φ4 @40 (0.5%) 485
D1N6 (0.6f'c/Ag)					

3. Study case: Reinforced concrete columns under varying transverse and axial loads

As the columns are the primary member of the structure that faces earthquakes for Beam-column systems and most the structures collapse due to the columns failures. The response of the column during an earthquake has become a sensitive topic for researchers (Lynn *et al*. 1996, Saatcioglu and Razvi 1998, Lam *et al*. 2003, Bechtoula *et al*. 2005, Lou and Xiang 2008, Lu and Chen 2008, Acun and Sucuoglu 2010, Hadi and Zhao 2011, Wang *et al* 2017, Bechtoula *et al*. 2015, Hidayat *et al* 2020, Abedini and Zhang 2021, Ping *et al* 2021). To evaluates the parameters that have a significant impact on the seismic performance of the plastic hinges in reinforced concrete columns Bechtoula *et al*. (2005) have made an experimental investigation of the seismic performance of cantilever reinforced concrete columns under varying transverse and axial loads (see Fig. 3).

3.1 Details of the chosen column

A reinforced concrete column under varying transverse and axial loads will be studied (Bechtoula *et al*. 2005). Table 1 presents the material characteristics and test variables. The

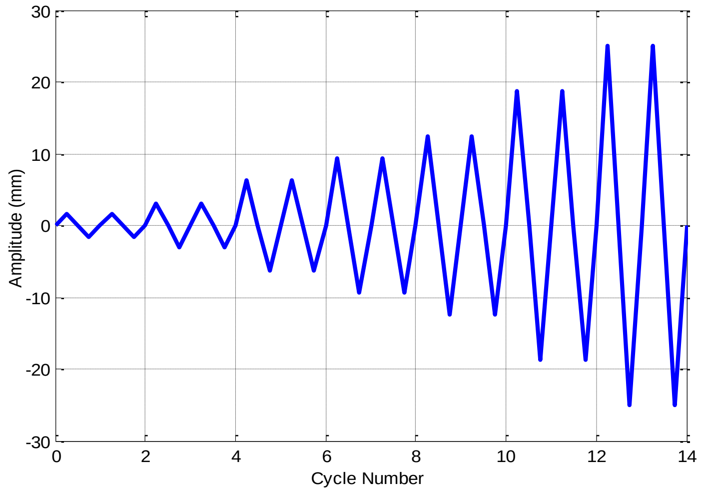


Fig. 4 Loading Protocol

Table 2 The selected values of the concrete parameter

Parameters		Denotation
Uniaxail loading		
f'c(MPa)	37.6	Compression Strength
E (MPa)	28819.854	Young Modulus
ν	0.18	Poisson's Ratio
fc0(MPa)	15.04	Yield stress on compression
ft0(MPa)	2.87	Yield stress on tensile
ψ	39	Dilatation angle
Є	0.1	Eccentricity
σb0/σc0	1.16	Ratio of biaxial to uniaxial
Kc	0.667	The ratio between the magnitudes of deviatoric stress in uniaxial tension and compression

column was loaded with a constant axial load of (0.3 or 0.6) f'c/Ag and a cyclic lateral displacement protocol was imposed by hydraulic jacks. The protocol is shown in Fig. 4.

3.2 Numerical modeling of the analyzed column

FE software ABAQUS/implicit is adopted for this analysis. The CDPM is used to simulate the material behavior of concrete. Table 2 displays the selected values of the concrete parameter. The time integration follows an implicit formulation, in this study, analyses are conducted for large displacement. An isotropic hardening has been used to define the steel material for both longitudinal and transverse bars. Fig. 5 shows the concrete discretization with 3D 8-node hexahedron solid elements (C3D8R) and the steel discretization with 2 node truss elements (T3D2). The Embedded element option has been used to connect reinforcing steel bars with the surrounding concrete assuming the perfect bond conditions. Fig. 6 present the concrete inputs properties of the

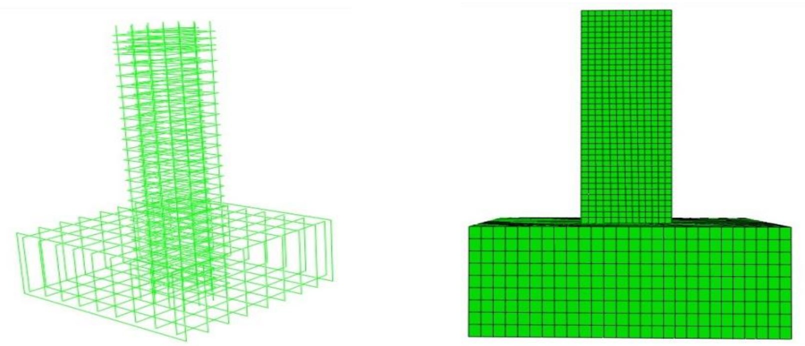


Fig. 5 Finite element discretization of RC column experiment (Bechtoula *et al*. 2005)

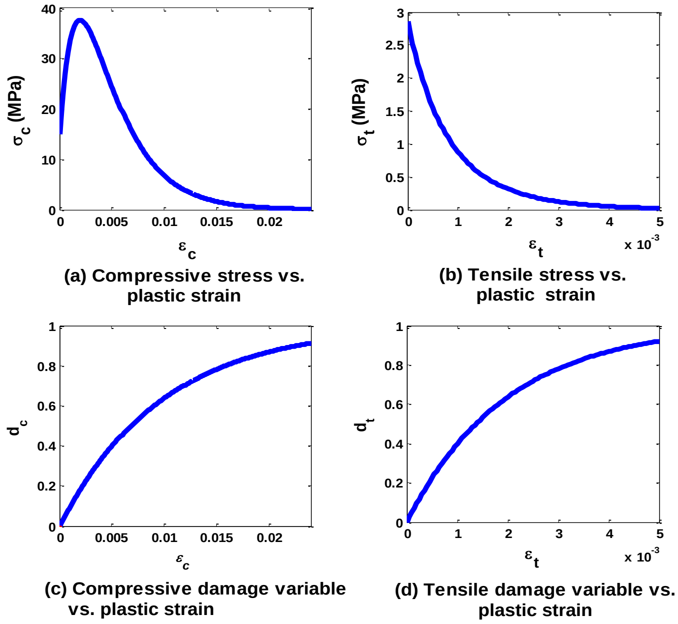
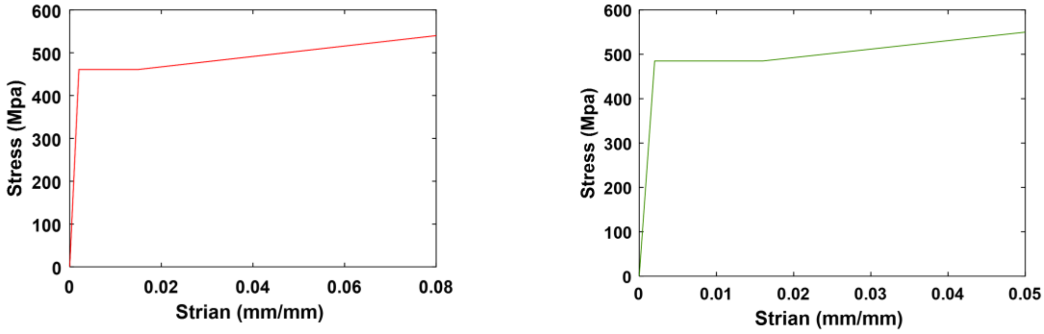


Fig. 6 Concrete inputs properties of the RC column experiment using the Lubliner/Lee/Fenves (1989-1998) approach

RC column experiment using the Lubliner/Lee/Fenves (1989-1998) approach. Where Fig. 6(a) and Fig. 6(b) present the compressive and tensile stress vs. plastic strain curves, respectively. Fig. 6(c) and Fig. 6(d) present the compressive and tensile damage variable vs. plastic strain curves. Fig. 7 presents the steel input properties of the longitudinal rebar D13 (Fig.7(a)) and shear rebar Φ4 (Fig.7(b)).

The CDM is known to be sensitive to the mesh size and the softening branches as well. For this simulation the chosen mesh for the column is 20 mm. We started with mesh size of 50 mm, 25 mm and after reduced that to 15 mm until it stabilized. However, there was no difference in the results between the 15 and 20 mm unless we had extra analysis time (see Fig. 8). The mesh sensitivity study found that further reducing the mesh size (less than 20 mm) had no significant effect on the calculated column response but it did cause convergence problems within the analysis since we are



Longitudinal Rebar D13 (b) Shear Rebar Φ4

Fig.7 Steel inputs properties of the RC column (Bechtoula *et al*. 2005)

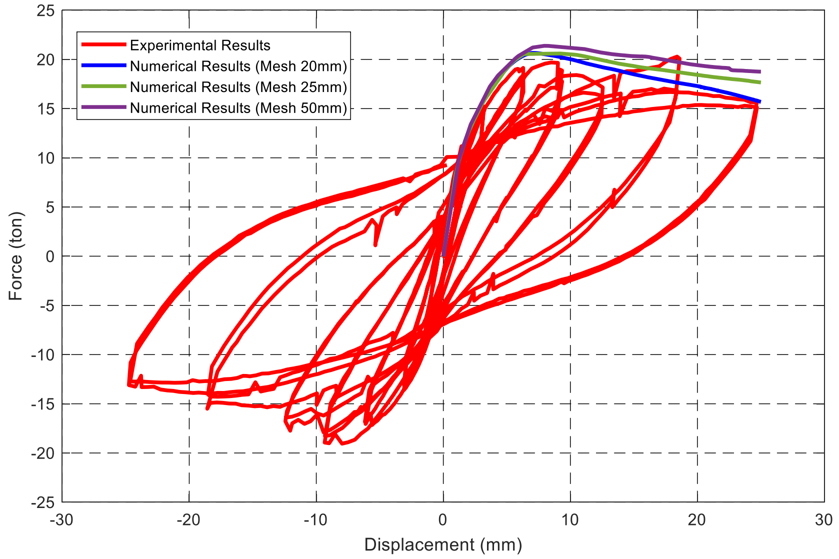


Fig. 8 Experimental and simulated capacity curves for D1N3 RC column (Bechtoula *et al*. 2005)

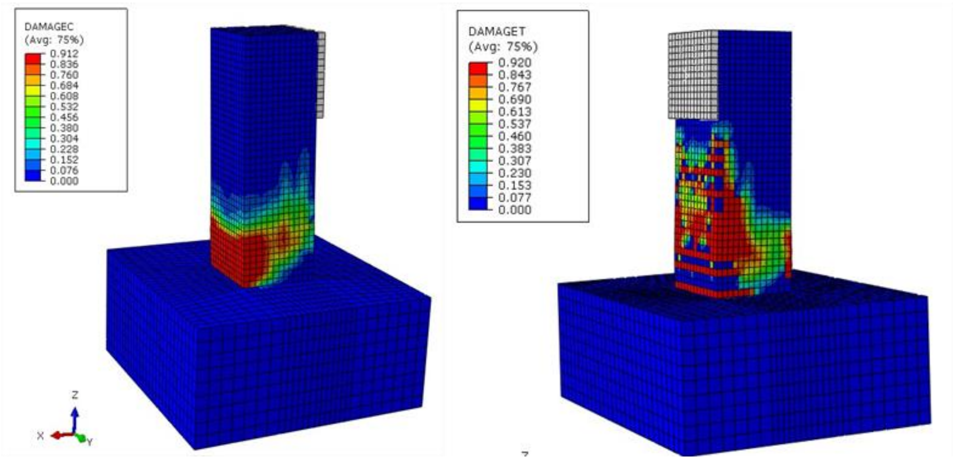
using implicit analysis. For the foundation part, a 50 mm mesh size was used. The damage result of the fine mesh (see Fig. 9(a)) is better than the medium (see Fig. 9(b)) and the large mesh (see Fig. 9(c)); certainly, further refinements would lead to higher accuracy (see Fig. 9).

3.3 Results and comparison between FE model and testes results:

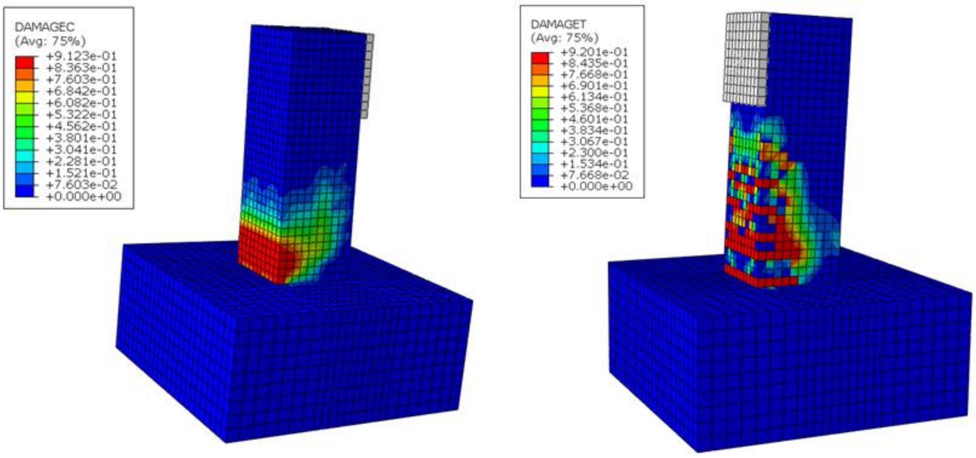
3.3.1 Monotonic loading

For the monotonic loading, the results of the numerical capacity curve are plotted together with the experimental results (see Fig. 8). Plots from Fig. 8 show that the numerical model captures the initial stiffness, the inception of overall yielding, maximum strength capacity, and ductility phase.

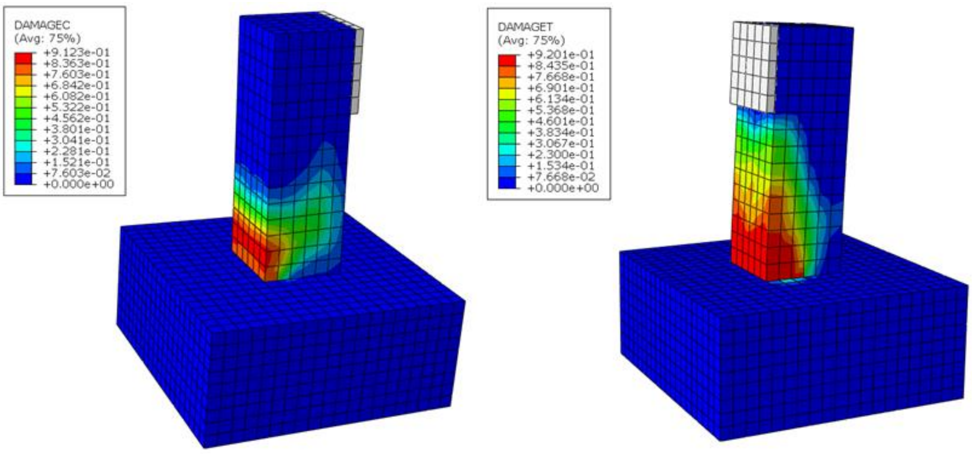
Fig. 9 (a) represents the final damage state of the compressive and tensile damage variables at the base of the chosen column, respectively. Where both the compressive and tensile damage variables took values close to 1. On the other hand Fig. 10 represents the results of the plastic



(a) Mesh Size 20 mm

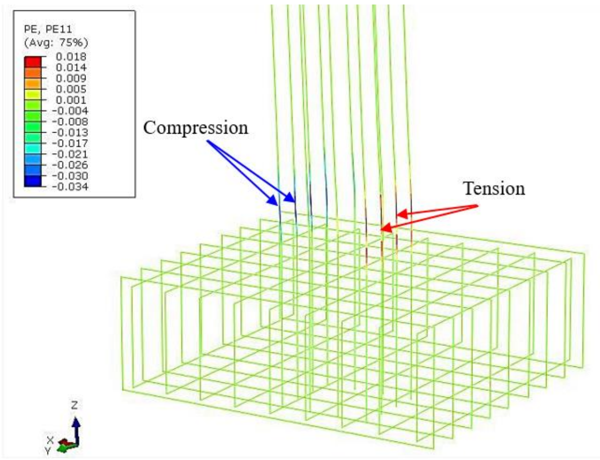


(b) Mesh Size 25 mm

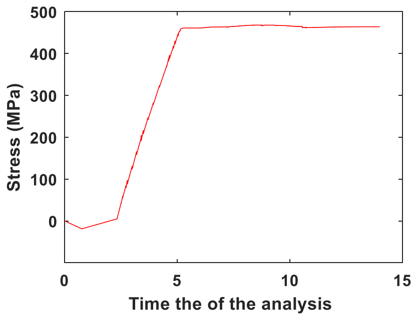


(c) Mesh Size 50 mm

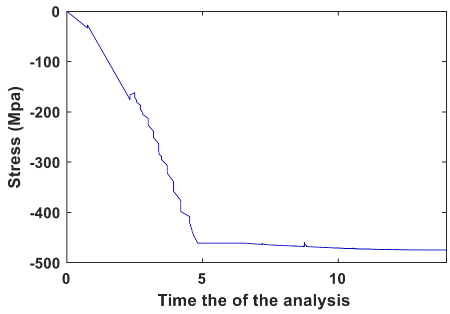
Fig. 9 The final damage state of the compressive damage variables at the base for D1N3 RC column experiment (Bechtoula *et al*. 2005) with deferent Mesh Size



(a) The plastic strain in reinforcing bars at the base for D1N3 RC column



(b) Tension stress(N/mm2) vs. Time of the simulation



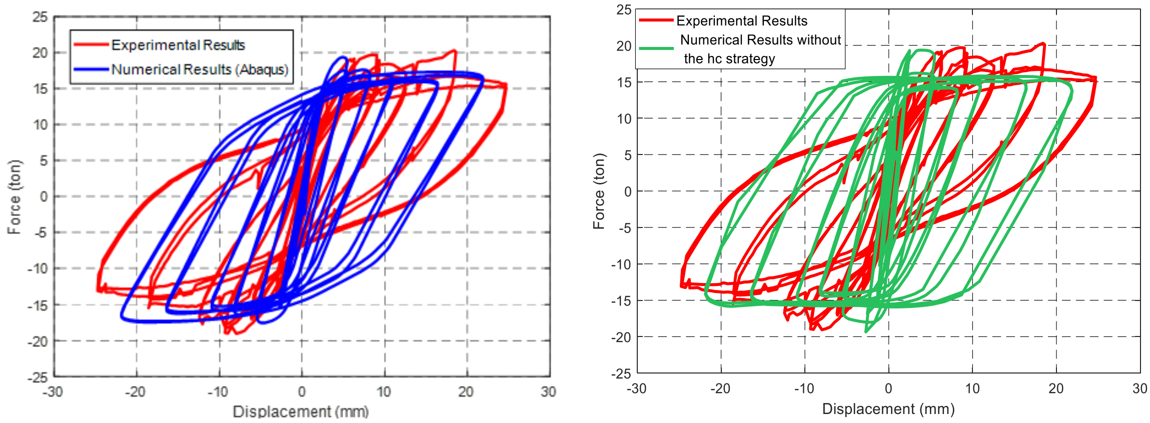
(c) Compressive stress (N/mm2) vs. Time of the simulation

Fig. 10 The plastic strain and the stress state in the reinforcing bars for D1N3 RC column

strain distribution for the longitudinal bars (Fig. 10(a)) and the plastic strain and the stress evolutions in the reinforcing bars in compression (Fig. 10(b)) and tension (Fig. 10(c)) for D1N3 RC column. Fig. 10 shows the maximum value of plastic strain of 0.018 and the minimum value of plastic strain of 0.034. The plastic strain and the stress evolutions in the reinforcing bars for D1N3 RC column.

3.3.2 Cyclic loading

For the cyclic loading, the results of the numerical hysteretic curve are plotted together with the experimental results (see Fig. 11(a)). Plots from Fig. 11 show unsuccessful in capturing energy, hysteretic response due to the lack of convergence since the RC column would experience significant sliding (opening and closing the cracks), and the option used takes perfect bonding between the concrete and the steel. The same conclusion has been concluded by Gulec, (2009) for the RC shear wall studied (see Fig. 11(b)). However, in this study the smeared strategy to simulate the reclosing of cracks after tension-compression reversals (with different values of hc along the length of the column) was used in this part based on Bashar Alfarah *et al*. (2017) approach to have more accurate results (see Fig 11(a)). For the Bond-slip effect modeling, an approach have been developed by Juan Murcia-Delso *et al* (2013) and evaluated after by Alfarah *et al*. (2017). This latter gives good results in terms of capturing energy and hysteretic response.



(a) with different cracks opening-closing behavior (b) with constant cracks opening-closing behavior

Fig. 11 Experimental and numerical (ABAQUS) force-displacement response of the D1N3 RC column (Bechtoula *et al*. 2005)

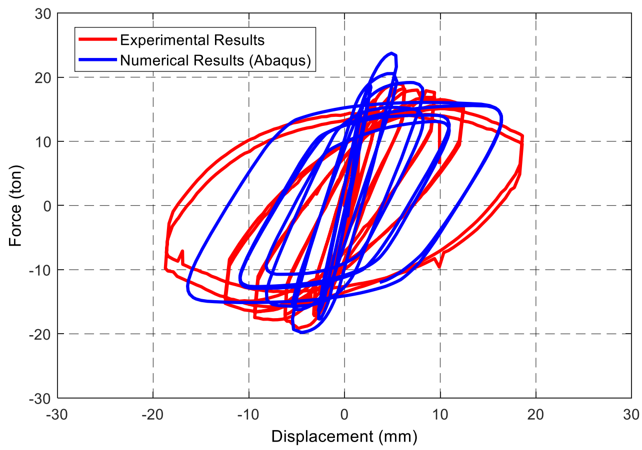


Fig. 12 Experimental and numerical (ABAQUS) force-displacement response of the D1N6 RC column (Bechtoula *et al*. 2005)

The reclosing of cracks after tension-compression reversals is governed by the parameter hc, representing the percentage of compression stiffness recovery in the reclosed cracks. Murcia-Delso, J. (2013) introduced discrete cracks at the member ends. In a current study, Bashar Alfarah *et al*. (2017) proposed an alternative solution by using different values of hc near the member ends.

Fig. 12 shows the experimental and numerical force-displacement response of the D1N6 RC column (Bechtoula *et al*. 2005). The results for D1N6 better capture the energy, hysteretic response, and deformation components compared to the D1N3 since the experimented column D1N6 did not experience significant sliding, and the perfect bonding option was enough to capture experimental behavior.

Fig. 13 shows the experimental and numerical force-displacement response of the D1N3 RC column (Bechtoula *et al*. 2005) using the Quasi-Static Cyclic Analysis with the IDARC program (Reinhorn *et al*. 2006) which is based on the concentrated plasticity modeling . In the case of the

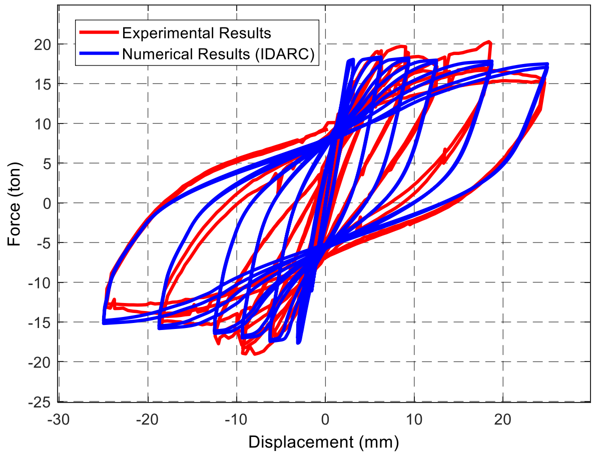


Fig. 13 Experimental and numerical (IDARC) force-displacement response of the D1N6 RC column

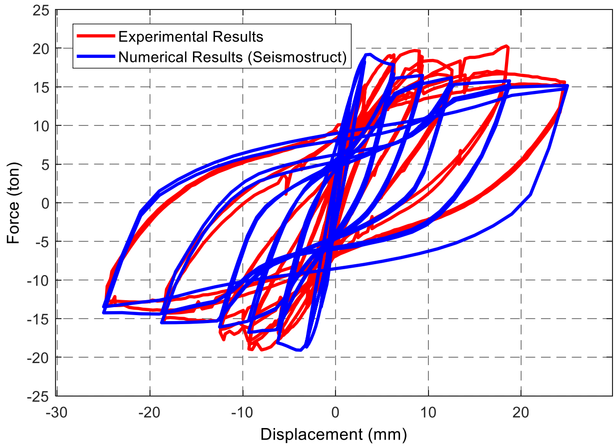
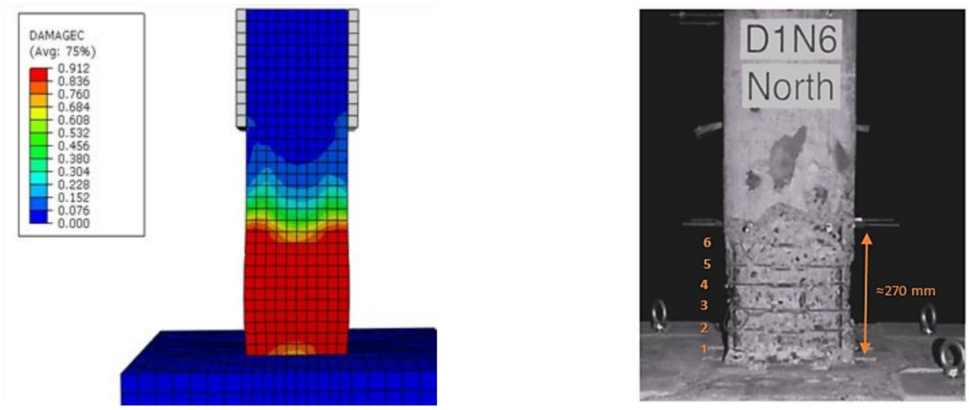


Fig.14 Experimental and numerical (Seismostruct) force-displacement response of the D1N6 RC column



(a) Numerical result (b) Experimental result

Fig. 15 The final damage state of the compressive damage variables D1N6 RC column and the experiment test (Bechtoula *et al.* 2005) numerical model used the bond-slip effect has been taken into consideration where the plots provide a satisfactory agreement with the test in terms of capturing energy, hysteretic response, and deformation components, however, It shows unsuccessful in capturing the stiffness degradation.

For the fiber section modeling Fig. 14 shows the results of the experimental and numerical hysteretic force-displacement response of the D1N3 RC column using the Seismostruct software where the behavior of the reinforcing steel is modeled using the cyclic material model developed by Menegotto and pinto (1973) and Filippou *et al*. (1983), and for the uniaxial hysteretic behavior of concrete is represented with the material model proposed by Chang and Mander (1994) that modeling the cyclic stress-strain behavior of the concrete.

In the case of the numerical results provide a satisfactory agreement with the test in terms of capturing energy, hysteretic response, and deformation components. The final state D1N6 RC column (Bechtoula *et al*. 2005) has been used to be the reference model. Fig. 14 presents a comparison between the numerical (Fig. 15(a)) and Experimental (Fig. 15(b)) final damage states where the obtained damage distributions fit the expected results.

The lengths of the crushed zone at the bottom part of the column obtained from the simulations and the experiment when the compression damage variable Dc >0.9 are shown in Fig. 15. In Fig. 15(b) six stirrups are clearly visible (clear distance is 20 mm) and the length is approximately equal 270 mm. For Fig. 15(a) the length measured is 275 mm. The numerical results lead to enough estimation of the crushing length of the tested column.

4. Conclusions

Numerical simulations of the chosen RC column under monotonic and cyclic loading using the FE Abaqus was presented in this paper in order to evaluate the hysteretic response and failure mechanisms in the RC elements. The main conclusions according to the obtained results of this investigation are summarized as follows:

• The numerical study by applying the cyclic behavior compared to experimental tests might be unsuccessful due to the lack of bond-slip modeling. On the other hand, the numerical model of the monotonic loading shows a good estimation of the envelope response and deformation components (captures the initial stiffness, the inception of overall yielding, maximum strength capacity, and ductility phase).

• The comparison between the Experimental and numerical final damage state fit the expected results found by Bechtoula *et al*. (2005) and the lengths of the crushed zone at the bottom part of the column obtained from the simulations (when the compression damage variable Dc >0.9) provide a close enough estimation of the crushing length of the tested column.

• In the FE Abaqus and for the reinforced concrete structures the embedded option is usually used for the contact between concrete and steel, which is considered as perfect bonding in this case; the slipping won't be taken into consideration. While modeling the bond-slip behavior is very important in terms of energy, hysteretic response, and deformation components. It is also to simulate the cracks spacing and widths in reinforced concrete, and the stiffness and deformation capability. For the RC element using advanced users (defined new subroutine) it will be one of the solutions to define the bond-slip effect in the case of the cyclic loading.

• The comparison between the experimental and the different numerical simulations showed a good estimation in terms of the hysteretic force-displacement response.

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