Numerical Modeling Approach for Progressive Collapse Analysis of Infilled RC Frames

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Abstract

Most design codes treat masonry infill walls as non-structural elements, and in the analysis of steel or reinforced concrete (RC) frame structures, their presence is often neglected. However, numerous studies have shown that infill walls significantly influence the strength, stiffness and ductility of the structure. Therefore, it is crucial to consider the contribution of infill walls to the building's behavior. Accurate modeling of the infill walls requires knowledge of the mechanical properties of masonry, various interacting parameters, and the contact conditions along the interface between the infill and the surrounding frame. Two primary techniques used in the analysis of infilled frame structures are macro-modeling and micro-modeling. Micro-modeling utilizes Finite Element (FE) software to represent the masonry panel as composed of many elements that simulate the bricks and mortar. In contrast, macro-modeling treats the masonry panel as a few elements, typically modeled as one or more compressive diagonal struts. Both techniques aim to capture the nonlinear behavior of the materials. The objective of this study is to assess the influence of the infill walls on the progressive collapse resistance of RC framed structures. To achieve this, an RC frame experimentally tested by Li et al. (2016) is modeled in Abaqus/Explicit software in two configurations: bare frame (without infill walls) and infilled frame (with infill walls). The frame consists of four bays and two stories, and progressive collapse is triggered by the failure of the middle column from the first story. This failure is simulated through a step-by-step unloading process in a displacement-controlled manner. To evaluate the progressive collapse behavior of the two numerical models, nonlinear explicit dynamic analysis is adopted to simulate the quasi-static loading scheme. The Concrete Damaged Plasticity (CDP) model is used for both concrete and masonry, with the compressive stress-strain relationship following the Kent-Scott-Park constitutive model. The steel reinforcement bars material properties are specified based on a bilinear stress-strain relationship. Tie-type connections are employed to model the interaction between the RC frame and masonry elements, while general contact is used to simulate the interaction between masonry elements. The resistance force (applied vertical load) versus the vertical displacement of the middle column is monitored up to a displacement of 500 mm. The progressive collapse behavior of the frame is divided into four stages. The numerical results obtained for both models show good agreement with the experimental ones. It was found that during the first deformation phase (when adjacent and exterior columns moved outward), the infilled frame model resisted a vertical force approximately 4.8 times greater than the bare frame model. In conclusion, completely neglecting the infill walls in the progressive collapse analysis of RC framed structures leads to unrealistic results.

Keywords: *Abaqus, infill walls, nonlinear analysis, progressive collapse, RC frame.*

1. Introduction

The progressive collapse analysis of structures is a highly important and intensively studied topic in the field of structural engineering and building safety. Progressive collapse refers to a phenomenon where a local failure within a structure propagates, leading to partial or total collapse [1]. Interest in this type of analysis significantly increased after dramatic events (such as the collapse of the World Trade Center Towers in 2001), which highlighted the vulnerability of modern structures to such phenomena.

The study of progressive collapse is carried out to better understand the mechanisms that lead to structural failure and to develop effective prevention methods. Through these studies, researchers aim to identify the weak points in structures and to devise constructive solutions that minimize the risk of collapse in the event of local failures, whether due to natural disasters (wind gusts, floods, fires, major earthquakes), terrorist attacks, design or construction errors.

In current design practice, partition and infill walls are considered non-structural elements, and it is assumed that these elements do not contribute to the building's strength. Their weight is considered in the seismic or progressive collapse analysis, but their contribution to the overall resistance of building is neglected.

Infilled frame buildings represent one of the most common construction solutions used throughout the world. In the most design codes masonry infill walls are considered non-structural elements, but they significantly affect the strength, stiffness and ductility of the structure. There are many studies [2-7] that have shown the beneficial effect of the infill walls on the seismic behaviour of reinforced concrete (RC) structures. More recent studies [8-18] have also investigated the contribution of the infill walls on the progressive collapse resistance of RC buildings.

Numerical modeling of infill walls can be performed using two methods: micro-modeling and macro-modeling. Micro-modelling involves using Finite Element computer software to define the masonry panel as in reality composed of many elements (which represents the bricks and mortar). While macro-modelling considers as few elements as possible to represent the masonry panel, which is considered equivalent to one or more compressive diagonal struts. Both techniques try to consider the nonlinear behaviour of the materials.

Li et al. [8], Shan et al. [10], Brodsky and Yankelevsky [12] have tested several RC frames with and without infill walls to investigate the influence of infill walls on the progressive collapse performance of these structures. Experimental results have shown that the infill walls can significantly enhance the resistance force and initial stiffness of the frames.

Using Finite Element models, Li et al. [9], Shan et al. [11], Eren et al. [13], and Yu et al. [14] have numerically validated the experimental tests conducted by Li et al. [8] and Shan et al. [10]. Additionally, parametric studies and load transfer mechanisms under a middle column removal scenario have been investigated. Based on the Applied Element Method in the Extreme Loading for Structures (ELS) computer software, Helmy et al. [16], Lupoae et al. [17], Besoiu and Popa [18] have found that infill walls play a valuable role in mitigating the progressive collapse of multistorey RC framed structures subjected to the loss of a column.

In order, to consider the effect of masonry infill panels, Crisafulli [2], Chrysostomou [3], and El-Dakhakhni [5] proposed different macro-models, which are commonly used in the seismic analysis of infilled frame structures. Recent studies [9, 19] have attempted to adopt these models in the progressive collapse analysis of structures.

The objective of this study is to numerically assess the influence of the infill walls on the progressive collapse resistance of RC frame structures. To achieve this, the RC frame tested by Li et al. [8] is modeled in Abaqus software in two configurations: bare frame (without infill walls) and infilled frame (with infill walls). The progressive collapse caused by the failure of the ground floor middle column is simulated in the experimental test through a step-by-step unloading process in a displacement-controlled manner. In Abaqus/Explicit software, a downward vertical displacement is applied incrementally, and a nonlinear dynamic analysis is adopted to simulate the quasi-static loading scheme. Finally, the results regarding the resistance force-vertical displacement curve of the middle column are compared.

2. Summary of experimental tests performed by Li et al. [8]

To evaluate the influence of infill walls in RC frame structures subjected to progressive collapse, Li et al. [8] have experimentally tested a planar frame with and without masonry infill walls. The specimen is made at 1/3 scale and represents the lower two stories of the original 8-story building. The tested sub-structure consists of four 1.7 m bays, the first level hight is 1.4 m, and the second level hight is 1.1 m. The geometry of the frame, the dimensions and reinforcement details of the beams and columns are illustrated in Fig. 1 (all dimensions are in mm).

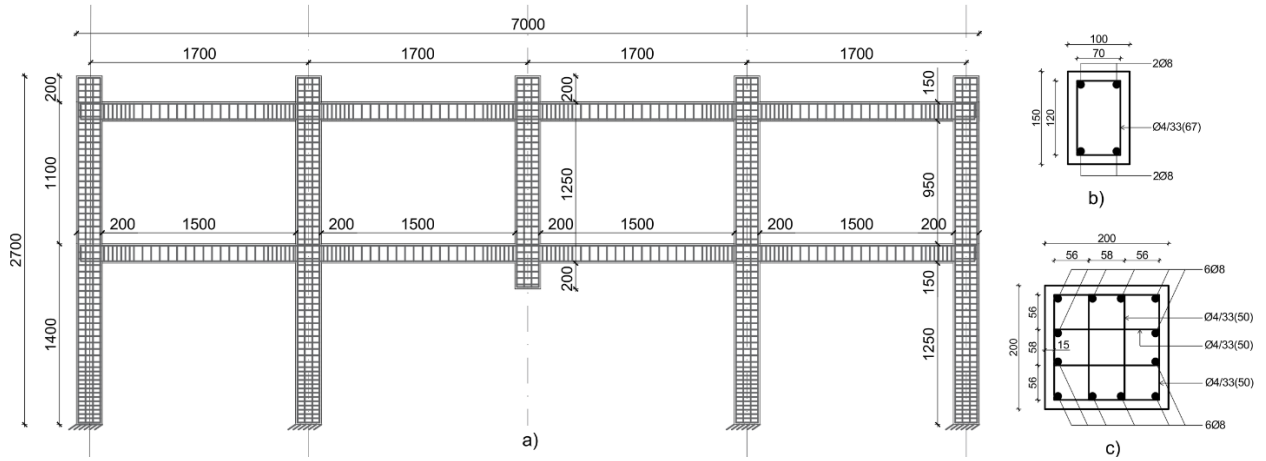


Fig. 1. Geometry of the RC frame (a) and reinforcement details of the beams (b) and columns (c).

The stirrups in the beams are arranged at 33 mm over a length of 250 mm from the face of the column (critical area) and at 67 mm in the rest of the areas. While, in columns only at the base of the first level the stirrups are arranged at 33 mm over a length of 450 mm, and in the rest of the areas they are arranged at 50 mm. The concrete cover is 15 mm. The infill walls are located on the second level of the specimen and are made of concrete blocks with dimensions of 130 mm x 63.5 mm x 63.5 mm. The thickness of the mortar on the horizontal joints is 8.9 mm, and on the vertical joints is 5.8 mm

The compressive strength of the concrete, determined on prisms of 150x150x300 mm, is 41.3 MPa in the elements on the first level and 31.8 MPa in those on the second level. For longitudinal reinforcement bars (ϕ=8mm) the measured yield strength was 415 MPa, the ultimate tensile strength was 588 MPa and the fracture strain was 18%. For stirrups (ϕ=4mm) the measured yield strength was 235 MPa, the ultimate tensile strength was 322 MPa and the fracture strain was 31%. The measured compressive strength for the masonry unit (combination brick and mortar) was 12.8 MPa and the shear strength was 1.08 MPa.

The progressive collapse of the frame was simulated in the experimental test by applying with a hydraulic jack on the top of the centre column a vertical load large enough to enforce the downward displacement of the column. Then the test was performed under displacement-controlled manner by lowering the hand jack (from the bottom of the column) step-by-step, thus simulating the loss of the first story middle column. Finally, the results regarding resistance force versus vertical displacement of the centre column for bare frame and infilled frame are compared.

For the bare frame, it was observed that the progressive collapse behaviour is divided into four stages: the initial stage - with nearly elastic behaviour, the compressive stage (before the vertical displacement reached 204.5 mm) - when cracks form near the beam ends, the catenary stage - when the cracks continually widened and at the vertical displacement of 317.1 mm the first rebar fractured in the top left end of the beam from the second level adjacent to the centre column, and the failure stage – when due to successive rebar fractures, the failure of the bare frame is observed at the vertical displacement of 417.7 mm.

For the infilled frame, the maximum resistance force of 152.1 kN is obtained at the vertical displacement of 13.5 mm (when the adjacent and exterior columns moved outward). After this, it was observed two sudden decreases in the resistance force at the vertical displacement of 20.3 mm and 90.6 mm, due to the appearance of cracks in the infill walls and the failures of the beams. The resistance force continues to gradually decrease due to rebar fractures and propagation of cracks in the infill walls up to a

vertical displacement of 367.5 mm, then increases again due to the catenary action. The test ended at a vertical displacement of 458.9 mm. [8]

3. Numerical models

The two experimental tests performed by Li et al. [8] are numerically modelled in the well-known Finite Element computer software Abaqus. To define the concrete and masonry solid elements, the eight-node linear brick element (C3D8) is used. To represent steel reinforcement bars, two-node truss element (T3D2) is used. The adhesion between concrete and reinforcement is considered an ideal interaction ("perfect bond"). The mesh size for all 3D solid elements (concrete and masonry bricks), and for linear elements (reinforcement bars) is set to 50 mm.

The material properties used in the numerical models developed in Abaqus computer software are given in Table 1.

Tab. 1. Material properties considered in Abaqus.

Material	Characteristics	Value
Concrete from the 1st level	Compressive strength	41.3 MPa
	Tensile strength	4 MPa
	Elastic modulus	32132 MPa
	Poisson ratio	0.2
Concrete from the 2nd level	Compressive strength	31.8 MPa
	Tensile strength	3.51 MPa
	Elastic modulus	28195 MPa
	Poisson ratio	0.2
Rebars (ϕ8)	Yield strength	415 MPa
	Ultimate strength	588 MPa
	Failure strain	0.18
	Elastic modulus	$2x10^5$ MPa
	Poisson ratio	0.3
Stirrups (ϕ4)	Yield strength	235 MPa
	Ultimate strength	322 MPa
	Failure strain	0.31
	Elastic modulus	$1.5x10^5$ MPa
	Poisson ratio	0.3
Masonry	Compressive strength	12.8 MPa
	Tensile strength	2.26 MPa
	Elastic modulus	11520 MPa
	Poisson ratio	0.15

To define in Abaqus the compressive stress-strain curves (σ - ε) for both concrete and masonry, the Kent-Scott-Park model (described in [20]) is used. These are presented in Fig. 2: a) for concrete and c) for masonry. The best results were obtained for the elastic modulus of concrete calculated with the equation (1), recommended by Mander et al. [21] for confined and unconfined concrete.

$$E_c = 5000\sqrt{f_c'} \tag{1}$$

where, $\mathbf{f_c'}$ is the compressive strength of concrete.

According to Abaqus documentation [22], for concrete the total strain values are processed using equation (2), resulting inelastic strain (values that are introduced in Abaqus).

$$\varepsilon_i^{pl} = \varepsilon_i - \frac{\sigma_i}{E_0} \tag{2}$$

where: $\boldsymbol{\varepsilon_i^{pl}}$ – plastic strain; $\boldsymbol{\varepsilon_i}$ – total strain; $\boldsymbol{\sigma_i}$ – compression stress, $\mathbf{E_0}$ – elastic modulus.

According to Masonry Standards Joint Committee [23], the elastic modulus of masonry units is calculated with the relationship:

$$E_m = 900 f_{cm}' \tag{3}$$

where, $\mathbf{f_{cm}'}$ is the compressive strength of masonry.

The tensile strength of concrete is determined by the following relation (also used by [8, 11]:

$$f_t = 0.6228\sqrt{f_c'} \tag{4}$$

where, $\mathbf{f_c'}$ is the compressive strength of concrete.

According to other studies [8, 11], the tensile strength of the masonry unit is 10% of its compressive strength. The residual tensile for concrete and masonry unit is set at 10% of tensile strength. Thus, the constitutive relations in tension for concrete and masonry are illustrated in Fig. 2. b) and d).

To define in Abaqus the mechanical behavior of the reinforcing bars, bi-linear stress-strain relationships were used (Fig. 2e). To simulate their failure, the stress was suddenly reduced to 5 MPa, immediately after the failure (ultimate) strain was reached. According to the Abaqus documentation [22], the option that can be used to simulate the progressive failure of ductile materials is "Ductile Damage". Thus, a value of the steel ultimate strain can be specified, in this case for the longitudinal reinforcement bars (Ø8) it is 18%, and for the stirrups (Ø4) it is 31%.

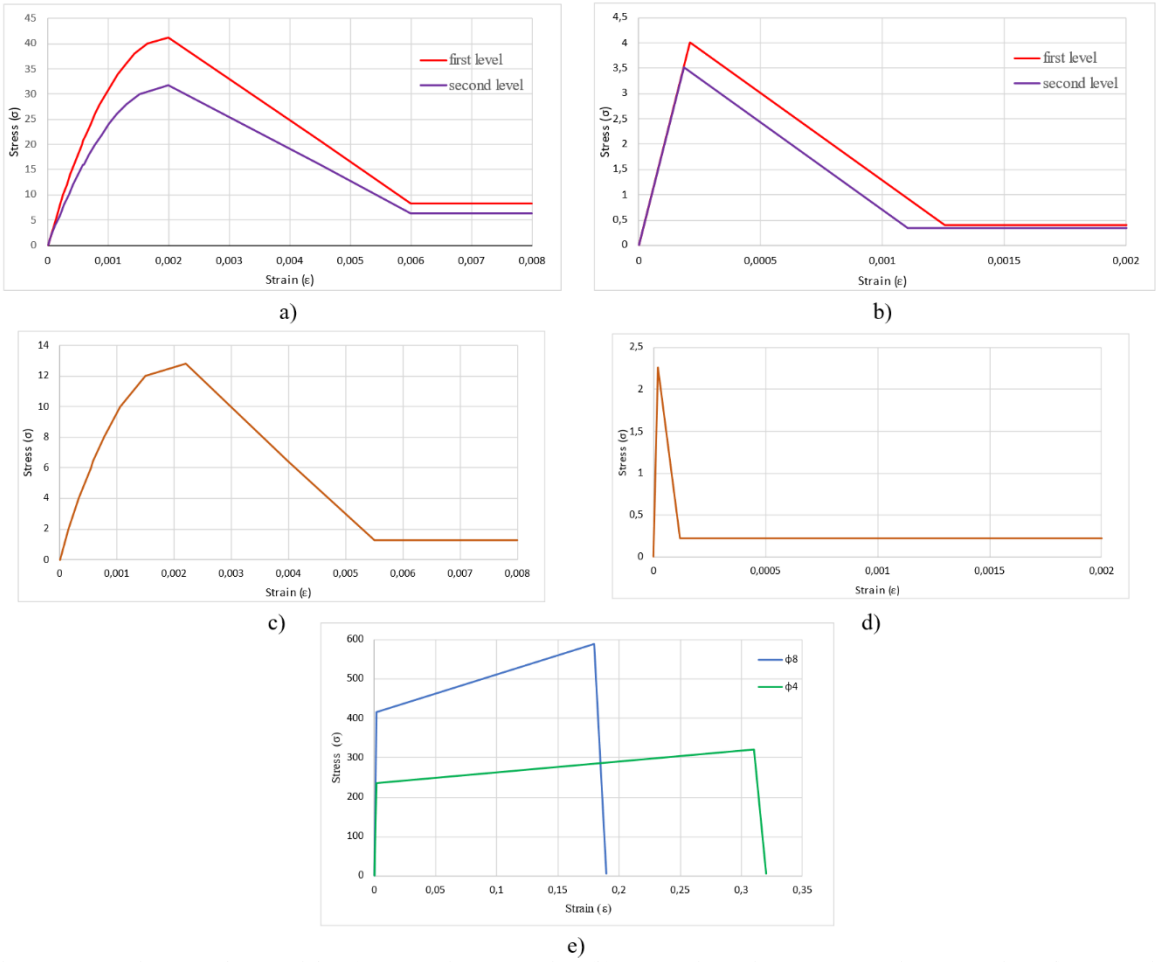


Fig. 2. Stress-strain curves for materials: (a) concrete in compression; (b) concrete in tension; (c) masonry in compression; (d) masonry in tension; (e) steel reinforcement bars.

In recent years, a significant increase in research on the nonlinear analysis of RC structures has been observed. Various analytical models have been developed to evaluate the behavior of concrete. The most common failure modes of concrete are compressive crushing and tensile cracking. In this study, the Concrete Damaged Plasticity (CDP) model was adopted for both concrete and masonry, to capture their behavior as accurately as possible. For this, it is necessary to define the following parameters in Abaqus computer software: dilatation angle (φ), eccentricity (ε), the ratio of the initial equibiaxial to uniaxial compressive strength (f_{b0}/f_{c0}), the shape factor of the yielding surface in the deviatory plane (K) and viscosity parameter (μ). The values of these parameters are given in Table 2, being established based on Abaqus documentation [22] and used also in other studies [8 ,11].

Tab. 2. Plasticity parameters used in Abaqus for CDP model.

φ	ε	f_{b0}/f_{c0}	K	μ
30	0.1	1.16	0.6667	0.001

According to Abdulla et al. [24], there are three techniques for modeling masonry elements in Abaqus: detailed micro-modeling, simplified micro-modeling and macro-modeling. For detailed micro-modeling, both masonry and mortar are modeled as continuum elements, and masonry-mortar interface is modeled as discontinuous elements. This type of approach can provide more accurate results but is computationally intensive and time-consuming. To reduce these shortcomings, simplified micro-modeling can be used, which is an alternative to detailed micro-modeling. In this case, the masonry units are expanded by adding half the mortar thickness for each brick. In the case of macro-modeling, masonry is considered as a homogeneous material, without distinguishing between masonry and mortar. The properties of the material used are obtained based on the average properties of the masonry components (brick and mortar). As a result, it can provide the global behavior of the structure but does not capture the detailed failure modes.

In Abaqus, "tie" type connections are considered to model the interaction between the reinforced concrete frame and the masonry elements, and general contact is used for modeling the interaction between the masonry elements. "Hard contact" is used for normal interaction and "penalty" for tangential interaction, with a friction coefficient of 0.7. For the "cohesive behavior", the following parameters k_{nn}, k_{ss} and k_{tt} (representing the elements of the stiffness matrix) must be defined. These parameters can be determined with the equation (5), used also by Abdulla et al. [24].

$$k_{nn} = \frac{E_u \cdot E_m}{h_m(E_u - E_m)}\ ;\ k_{ss} = k_{tt} = \frac{G_u \cdot G_m}{h_m(G_u - G_m)} \tag{5}$$

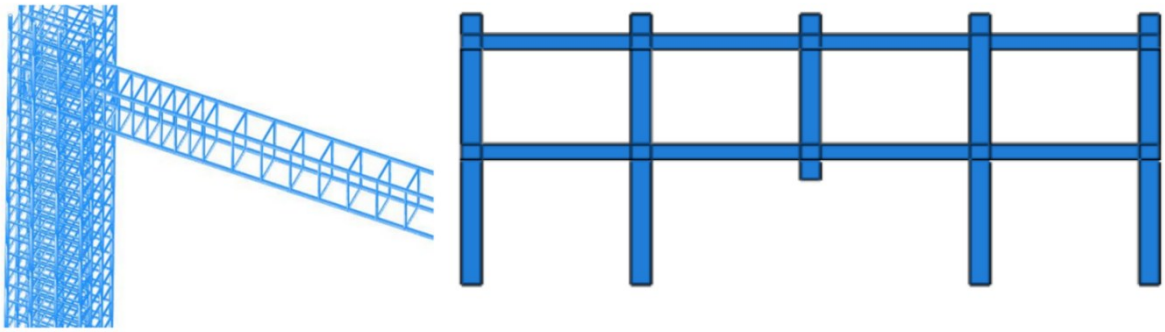
where: E_u, E_m, G_u and G_m are the elastic moduli and the shear moduli of the mortar and masonry units, and h_m is the thickness of the mortar.

In this study, the progressive collapse analysis is performed in Abaqus computer software, version 6.13. According to Abaqus documentation [22], using Abaqus/Explicit one can study dynamic and short-term phenomena, such as explosion, impact etc., which can also generate information about material discontinuities (crushing, cracking or rupture). Nonlinear dynamic analysis is adopted to simulate the step-by-step unloading of the middle column. All columns are fixed at the base, except for the middle column where only vertical displacement is allowed. Thus, horizontal displacements are blocked at the bottom of the central column, and a vertical displacement of 500 mm is applied in several steps. Removing the middle column from the ground floor gradually leads to the failure of the frame.

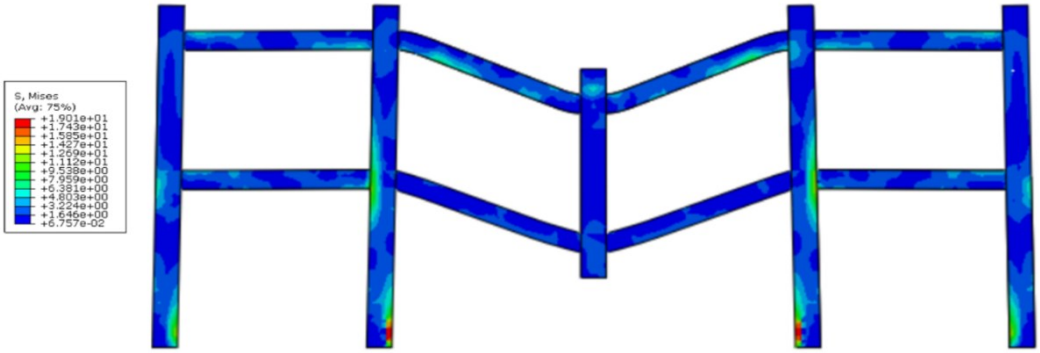
The resistance force (vertical load) and the vertical displacement of the removed middle column is monitored until the value of 500 mm resulting the capacity curve of the frame. The results obtained for the two numerical models developed in Abaqus based on experimental tests [8] are presented in detail below for: bare frame model (without infill walls) and infilled frame model (with infill walls). Finally, to evaluate the effect of the infill walls on the progressive collapse behaviour of the RC frame, these results are compared.

3.1 Bare frame model (without infill walls)

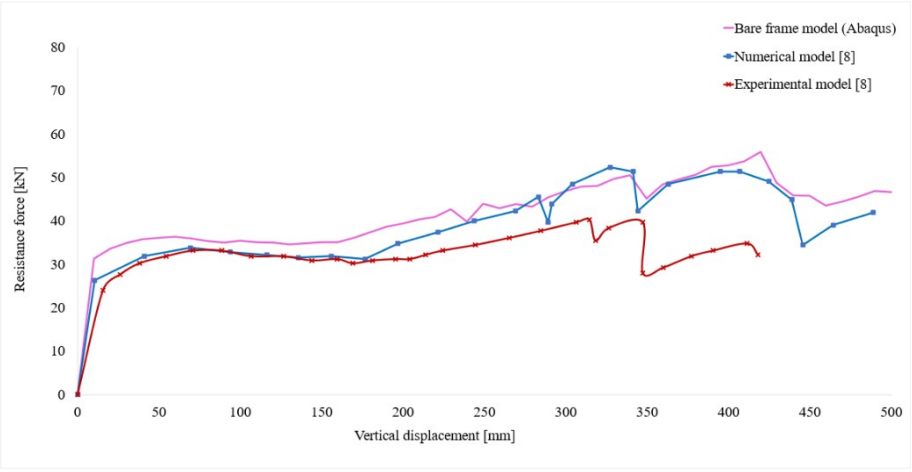
Considered as reference model, the bare frame illustrated in Fig. 3 (a) is firstly analysed in Abaqus. The numerical results are comparted with the experimental ones obtained by Li et al. [8]. In Fig. 3 (b) the frame deformation at the end of the analysis can be seen, and in Fig. 3 (c) the resistance force-vertical displacement curve of the middle column is presented.



a)



b)



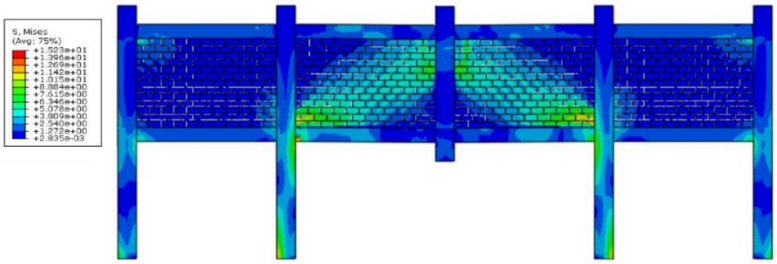
c)

Fig. 3. Bare frame model in Abaqus (a) and numerical results: frame deformation at the end of the analysis (b); resistance force-vertical displacement curve (c).

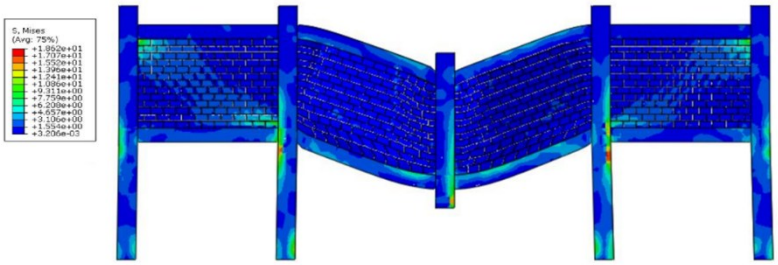
As in the experimental test, it is observed that the progressive collapse behavior of the numerical model developed in Abaqus follows the four stages (presented in section 2) as the vertical displacement of the middle column increases. In the initial stage the specimen behaves elastically with a constant initial stiffness, until the vertical displacement of 10 mm is reached. After this, the compressive arch action (stage II) develops in beams until the vertical displacement of approximately 160 mm. It is observed that the resistance force in the middle column gradually decreases after reaching the maximum values in this stage. Due to geometric nonlinearity, catenary action (stage III) develops in the beams adjacent to the middle column, leading to a gradual increase in the resistance force. Stage III is completed when the vertical displacement reaches the value of 339.46 mm. Subsequently, due to the successive ruptures of the reinforcing bars, sudden decreases in the resistance force occur until the final failure of the frame.

3.2 Infilled frame model (with infill walls)

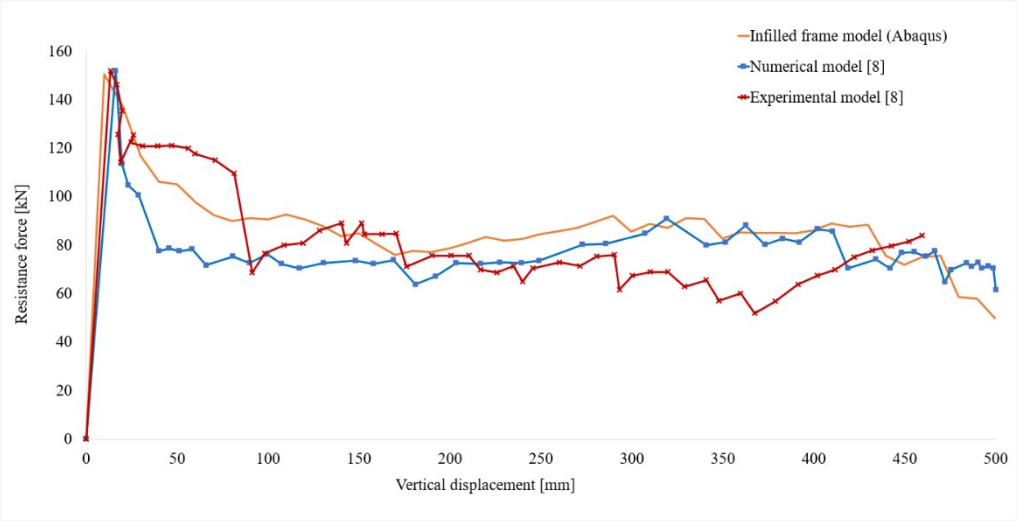
The infilled frame model is identical to the bare frame model, but with masonry walls along the entire height of the second level. For this, simplified micro-model is adopted in Abaqus, where the thickness of the mortar layer is reduced by half and attached to the adjacent bricks, and the masonry units are connected by cohesive elements. The numerical results are presented in Fig. 4 and are compared with those obtained experimentally and numerically by Li et al. [8].



a) Δ = 10 mm



b) Δ = 500 mm



c)

Fig. 4. Infilled frame numerical results: deformed shape at the initial stage (a); at the end of the analysis (b) and resistance force-vertical displacement curve (c).

As in the case of bare frame, the progressive collapse behavior of the infilled frame can be divided into four stages. In the initial stage, the infilled frame has an elastic behavior. A large increase in the resistance force is observed, reaching the value of 150.27 kN, at a vertical displacement of the middle column of 10 mm. This increase is due to the presence of the infill walls. In the second stage (compressive arch stage), stiffness decreases due to the failure of the beams, and a sudden decrease in the resistance force (at a vertical displacement of 40 mm) is also observed, caused by the deterioration of the cohesive layer between the bricks. Due to the yielding of the rebars, forming plastic hinges at the ends of the beams and due to the cracking of infill walls, the resistance force gradually decreases until the vertical displacement of 189.53 mm. After this, due to the catenary action (stage III) developed in beams, the resistance force increases until the vertical displacement of 429.56 mm, then it begins to decrease resulting in the failure of the frame (stage IV).

3.3 Comparative results

To evaluate the influence of infill walls on the progressive collapse resistance of an RC frame, the resistance force – vertical displacement curve of the middle column obtained for both the bare frame model and the infilled frame model is compared. As can be seen in Fig. 5, the infilled model has a much higher resistance capacity and stiffness than the bare frame model. The maximum value of the resistance force for the infilled model is 150.27 kN and it was recorded in the initial phase of frame deformation. For bare frame model, the maximum value of the resistance force is 55.95 kN, recorded at a vertical displacement of 419.63 mm (in stage IV). In the initial stage (at a vertical displacement of 10 mm), the resistance force for the bare frame model is only 31.3 kN, which means that for the infilled model it is approximately 4.8 times greater. This much higher resistance of the infilled frame model is also due to the diagonal compressive mechanism developed in the infill walls adjacent to the middle column, which can be observed in Fig. 4 (a).

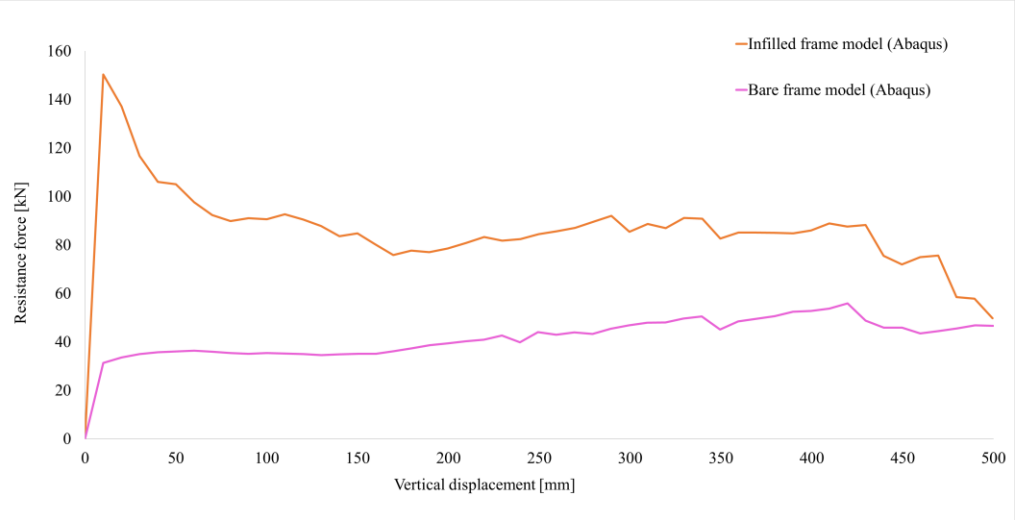


Fig. 5. Resistance force-vertical displacement curve for bare frame model and for infilled frame model.

Both models developed two deformation phases before the final collapse. In the first phase, the columns adjacent to the middle column move outwards, indicating compressive arch action in beams. In the second phase, the same columns move inwards, indicating the catenary action in beams. The horizontal displacement of beam-column joints (at the top of the 2nd level in the adjacent bays) versus vertical displacement of the middle column is illustrated in Fig. 6 for both models. It can be observed that, for the bare frame model the vertical displacement of the middle column corresponding to the maximum horizontal outward displacement of the adjacent columns is 110.06 mm, and for infilled frame model it is 119.95 mm.

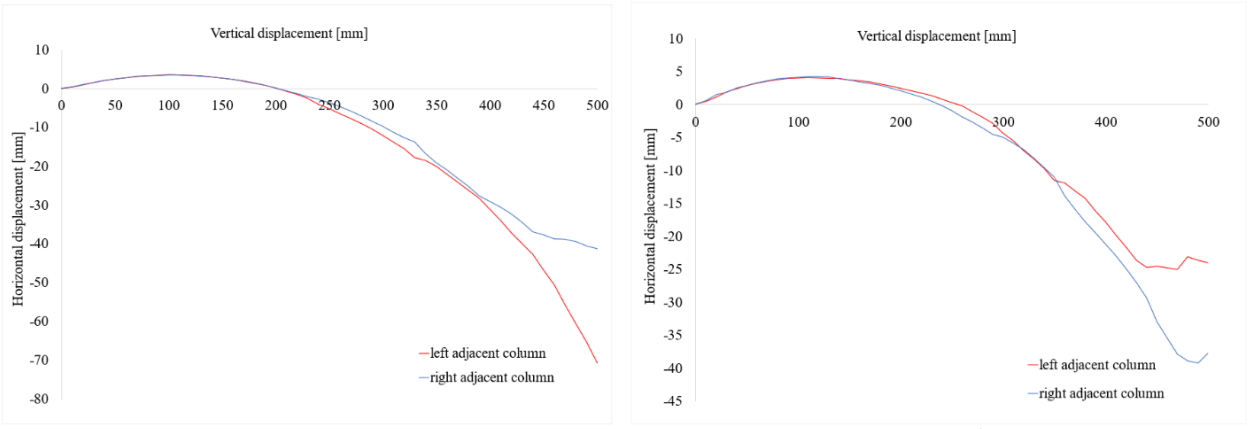


Fig. 6. Horizontal displacement of beam-column joints versus vertical displacement of the middle column: (a) for bare frame model; (b) for infilled frame model.

4. Conclusion

In this study, Finite Element modeling was used to evaluate the influence of infill walls on the progressive collapse resistance of RC frame structures. For this purpose, the calibration of an experimental test conducted by Li et al. [8] on a two-story, four-bays planar frame subjected to the loss of the ground floor middle column was performed in Abaqus computer software. Thus, two numerical models were developed: bare frame model (without infill walls) and infilled frame model (with infill walls).

To simulate the gradual loss of the middle column, a vertical displacement of 500 mm was applied incrementally. The nonlinear dynamic analysis was adopted in Abaqus/Explicit, and the total loading time was set at 2 seconds. In this study, for modeling the masonry infill walls a simplified micro-modeling technique was adopted. In this approach, the thickness of the mortar layer was reduced by half and attached to the adjacent bricks, and the masonry units were connected by cohesive elements. The analysis time for bare frame model was approximately 2 hours, and for infilled frame model about 12 hours.

The main objective of the study was to compare the capacity curves (resistance force versus vertical displacement of the middle column) obtained for the two numerical models. These were also compared with the experimental results obtained by Li et al. [8] resulting in a good agreement. It was observed that the two numerical models have the capacity to develop the failure mechanisms of the beams adjacent to the middle column: compressive arch action and catenary action.

Due to the presence of infill walls, the resistance force of the infilled frame model was 4.8 times grater than that obtained for the bare frame model (in the initial stage). Experimental results showed that the maximum resistance force when column moved outward was 4.58 times higher for the frame with infill walls than for the frame without infill walls. The horizontal displacement of the beam-column joins versus vertical displacement of the middle column was also compared for both models.

In conclusion, two numerical models capable to capturing the progressive collapse behaviour of both a bare frame and an infilled frame were developed in Abaqus. The presence of infill walls in the model led to a significant increase in the resistance force and stiffness of the frame. Also, it has been observed that the values of some parameters that describe the interaction between masonry units in Abaqus (like stiffness coefficients and fracture energy) have a major influence on the response of the structure. To evaluate the effect of these factors on the resistance force-vertical displacement curve for infilled frame model, a parametric study is in progress (as a future research direction).

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