Behavior of Reinforced Concrete Frame with Masonry Infill Wall Subjected to Vertical Load

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Abstract:

The effectiveness of masonry infill wall on behavior of a Reinforced Concrete (RC) frame subjected to a column failure is studied experimentally. For this reason, one full scale RC frame designed according to Eurocode is statically tested to investigate the behavior of the frame with and without masonry infill wall. The obtained results show that infill wall can significantly increase the load carrying capacity of RC frame and thus serve as an important robustness reserve in the case of unpredictable extreme events (i.e. local impact, blast or earthquake). A photogrammetry analysis is carried out to study the behavior of the structure. Results give valuable information about the alternative load path, transfer of the applied load to the column and beams, and interaction forces between RC frame and infill wall. At the end, the experimental program is simulated by the OpenSees software to study the behavior of the frame. After having demonstrated that this model can predict the load deflection with good accuracy, a parametric study is conducted to evaluate the effect of the percentage of longitudinal reinforcement ratio of beams and columns on the load carrying capacity of the infilled RC frame.

Keywords: Robustness, Reinforced Concrete Frame, Infill walls, Photogrammetry, OpenSees.

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Introduction:

Progressive collapse means failure of a primary structural element that can resulting in the failure of adjoining structural elements, which in turn causes further structural failure [1]. Progressive collapse of multistory buildings can occur after local failure of a key structural member, typically originated by extreme unforeseen events, such as: earthquake, different types of natural disasters, man-caused accidents, and terrorist attacks. In Ronan Point, London [2], a gas explosion on the 18th floor of a residential building blew out a wall element causing a progressive collapse of the building. In the Bad Reichnhall arena, Germany [2], due to a design error and undetected deterioration, a progressive collapse occurred under snow load, and led to the total collapse of the roof of the structure. These are clearly some examples of non-robust structures and the observed types of failure that may be seen as the result of the incapacity of the structures to develop alternative load paths after local damage of a member [3].

To avoid disproportionate failure, robustness must be ensured, i.e. the structure must develop alternative load paths after loss of a key-member. For instance, after sudden failure of a column, the connecting beams and infill panels must be able to transfer the redistributed loads to the adjacent columns [4]. This scenario highlights the importance of robustness. Robustness has been recognized as a desirable property of structural systems which mitigates their susceptibility of progressive disproportionate collapse [5]. In general, robustness is defined as the insensitivity of a structure to local failure. In a robust structure, no damage disproportionate to the initial failure will occur. Thus, an appropriate assessment of the structural behavior requires accounting for alternative load bearing scenarios that contribute to the overall resistance. Among the fundamental mechanisms of arrest, shear deformation of infill panels can provide significant enhancement of the resistance against collapse in frame RC structures [4].

The influence of the masonry infill panels is not generally considered in the design process of RC frames subjected to lateral loads, due to early brittle failure and consequently formation of soft-story mechanism and column shear failure. In reality, masonry walls are often arranged non-uniformly in different floors for functional reasons that cause the RC buildings have vertical irregularity, such as stiffness irregularity (soft story), strength irregularity (weak story), mass irregularity, and short-column effects. However, it is generally accepted that these elements have a significant influence on the structural behavior. They increase initial stiffness and decrease the natural period of the frame, which might be beneficial depending on the frequency of earthquake motion [6].

Shan et al. [7] studied experimentally the progressive collapse of a two-story four-bay RC frame with and without infill wall. The test results showed that the infill walls can provide alternative load paths for transferring the loads originally only supported by the beams, and thus, improve the collapse resistance capacity of the RC frame.

Tsai and Huang [8, 9] studied the progressive collapse of RC frames numerically and showed the effect of infill walls on the structure's resistance capacities against progressive collapse. The results showed that the progressive collapse depends on the dimensions as well as the locations of infill wall. However, infill walls have slightly influence on the structure's collapse resistance capacity because, infill walls have a brittle behavior and with a small deformation whereas collapse of structures is involving with a large deformation [7].

Tiago and Julio [10] described a case-study of a 12 stories residential building that experienced a landslide during the rainy season which destroyed three perimeter columns at the basement level and that, nonetheless, did not collapse because masonry infill walls created an alternative load path and transferred load failed columns to adjacent columns. Cachado et al. [2] performed a numerical simulation on the mentioned building. Authors concluded that regardless of the low mechanical resistance of masonry infill wall elements, compared with other structural elements, their contribution for the global behavior of the damaged structure is essential.

In a research conducted by Pujol et al. [11], the influence of masonry infill walls in the mitigation of progressive collapse of a RC structure was investigated by conducting an experimental program composed of a full-scale three-story flat-plate structure that was strengthened with infill brick walls. Results have shown that continuous masonry infill walls can contribute positively to reduce the vulnerability of the RC structures. These walls were effective in terms of increasing the strength (by 100%) and stiffness (by 500%) compared to the reference frame without masonry infill wall.

The behavior of Hotel San Diego, a six-story reinforced concrete infilled-frame structure that two adjacent columns simultaneously were removed using explosives, was studied by Sasani [12]. The structure resisted progressive collapse with a measured maximum vertical displacement of 6.4 mm above the removed columns. Reaction of transverse and longitudinal frames with contribution of infill walls were identified as the principal mechanism for redistribution of the loads in the structure.

The purpose of this study is to experimentally evaluate the behavior of a full-scale RC frame designed according to Eurocode 2 (EC2) [13] and Eurocode 8 (EC 8) [14], featured without and with the infill walls in the case of loss of a supporting column and to study the role that the infill wall played in the stiffness of the RC frame. The frame is filled by typical double ceramic brick wall. In this study the dynamic aspects of the column failure and the resulting frame response are not studied, and attention is just given to the quasi-static loading behavior of the RC frame. The experimental program consisted of one full scale RC frame subjected to vertical load as column failure and tested in three phases: i) isolated RC frame tested in elastic regime; ii) RC frame with masonry infill wall tested up to failure of the wall; and iii) isolated RC frame after infill wall removal tested up to failure. The experimental program is detailed, and the obtained results are presented and discussed.

Following the experimental research, a numerical simulation was carried out using the OpenSees software [15]. The values of the parameters that define constitutive models used in numerical simulation were calibrated by simulating the tested frame, considering the properties obtained in the experimental programs for the characterization of the relevant properties of the used materials, and the suggestion of EC2 [13]. After having been demonstrated that the model is capable of simulate the behavior of the tested frame with high accuracy, a parametric study was carried out to study the influence of the percentage of longitudinal reinforcement ratio in beams and columns on the load carrying capacity of the infilled frame.

Experimental Program:

3D frame is used to address three dimensional effects such as contribution of slabs, while 2D frame may give conservative results if these results are not accounted for [7]. However, 2D frame is recognize accurate to study the behavior of infill walls in term of progressive collapse. The experimental program was composed of one real scale one-bay, one-story reinforced concrete frame (5000 mm×2550 mm^2, center to center), designed according to EC2 and EC8 [13, 14]. According to EC8 [14], the frame was designed to have strong columns and weak beams. Al-Chaar and Lamb [16] model was used to estimate the influence of the infill wall on the RC frame and then reinforcement was overdesigned to allow a single frame to be tested in three different phases without experiencing significant damage. Dimensions and reinforcement details of the frame are presented in Fig. 1. To allow in-plane deformation, the out of plane deformation was restrained at the upper beam level. The right-hand column base was fixed in both directions (horizontal and vertical), to present the lower floor column and the horizontal elements at the adjacent span. A vertical constant load of 220 kN was applied to represent the upper floor column axial force. This load was applied by pre-stressing of two vertical steel bars as shown in Fig. 1. Longitudinal reinforcement ratio in columns and beams was 2.7% and 0.96%, respectively, whereas transverse reinforcement ratio was 0.88% and 0.44%. The left side column bottom was not presented in structural design to simulate the scenario of removal column from system.

The experimental program was divided in three main phases:

Phase 1: Elastic regime (F-Re) (Fig. 1b), to understand the behavior of the frame without masonry infill wall. As mentioned before, the frame was overdesigned, then no plastic strains were expected to be reached in the longitudinal reinforcement.

Phase 2: In this phase the frame was filled by typical double ceramic brick wall (T-Tr) (Fig. 2) and was pushed up to failure of the wall. The ceramic bricks used in the two wall panels had a dimension of 300×200×150 mm^3 and 300×200×110 mm^3. An air gap of 40 mm was left between the wall panels to improve thermal performance. Again, due to increased stiffness provided by the masonry wall, no plastic strains were expected to be reached in the longitudinal reinforcement of RC frame.

Phase 3: in this phase the infilled wall was removed, and the bare frame was pushed up to failure (F-B).

Material Properties

The shear strength of masonry wall units was assessed according to EN 1052-3 [17]. As shown in Fig. 3a, specimens composed of three bricks and two mortar interfaces were tested under a 3-point load test setup. Supports were placed under the lateral bricks, whereas the load was applied to the central brick. To measure vertical displacements, a set of three Linear Variable Differential Transducer (LVDT) was used, as shown in Fig. 3b. The relative displacements of lateral bricks to the central brick were measured using LVDTs 1 and 2, while LVDT 3 was used to measure the absolute displacement of the central brick. The shear strength of each specimen was determined according to Eq. (1):

$$f_v = \frac{F_{\max}}{2A} \tag{1}$$

where $F_{\max}$ is the maximum applied load and A is the effective normal area of the specimen. The results of shear strength are presented in Table 1. According to the results, the shear strength of masonry wall is highly dependent on the mortar interface strength, because all the three samples failed at mortar interface.

Masonry compressive strength was executed on prismatic specimens composed by three bricks and two mortar interfaces, as shown in Figs. 3c and 3d. Each specimen was monitored with one vertical LVDT to measure modulus of elasticity. A 300 kN load cell was used to measure testing force. In the first stage, five load-unload cycles were applied (Fig. 4). As shown in this figure, the first two cycles were driven up to a maximum load corresponding to $0.1 f_k$, and the remaining three cycles were driven up to $0.2 f_k$. For each cycle, the maximum load was kept constant during 30s. Then, the load was increased up to failure of the specimens. During this last stage, a vertical displacement was imposed at a rate of 0.01 mm/s. The compressive strength, f_c, of each specimen, was computed by the following equation [18]:

$$f_c = \frac{F_{\max}}{A} \quad (2)$$

where $F_{\max}$ is the maximum recorded force, and A is the effective loaded area of the specimen. The modulus of elasticity was computed based on the records of the vertical displacement transducers. The results of the compressive strength and modulus of elasticity are presented in Table 2.

Regarding the mechanical properties of masonry walls, it is obvious that the mechanical characteristics of the masonry walls directly depend on constituent materials. Besides that, the quality of the workmanship is effective. From the tests conducted by Pires [19] it can be concluded that the quality of the mortar and workmanship have a strong influence on the masonry shear strength, however, not a significant influence on the compressive strength.

The concrete compressive strength of the frame was evaluated at 28 days by direct compression tests on cubes of 150×150×150 mm^3. The average of cubes concrete compressive of 12 specimens was 44.23 MPa. The values of tensile properties of the steel bars were obtained from uniaxial tensile tests. The average value of the yield stress of the steel bars of 10, 16, and 20 mm diameter were 540, 533, and 618 MPa, respectively, while the average value of the tensile strength for these corresponding bars were 570, 640, and 720 MPa, respectively.

Test setup

In reality, if a column is removed from an RC structure the beam-column joins at top start to move downward. While, in this experimental program, a quasi-static load was applied by using a closed-loop servo controlled hydraulic actuator at the bottom side of column (Fig. 1) and the column pushed up due to the limitation of laboratory. The general arrangement of the test setup is shown in Fig. 1. All the three phases were run in a displacement-controlled mode at a rate equal to 0.01 mm/s. The vertical deflection of the frame was measured with one LVDT at the location of the applied load. The out of plane movement of the frame was recorded by two LVDTs. The strain in the longitudinal reinforcement of the columns and beams were measured by 40 strain gages (Fig. 1a). These strain gages helped to be sure the frame was in elastic regime and steel longitudinal reinforcement did not reach their yielding in first and second phase of the test. The right side of RC frame was fixed to rigid floor of laboratory by using two pre-stress steel bars. The right bottom part of RC frame is also fixed to the rigid wall as shown in Fig. 1a. Two LVDTs were installed to assure a rigid support in top and bottom parts of the frame as shown in Fig. 1a.

The testes were monitored by using one global (#5) and four local high-resolution cameras (#1 to #4) to capture the deformed shape of the frame and behavior of the beam-column connection, respectively. The positions of the cameras are presented in Figs. 1a and 2b. The surface of the frame and wall were painted white for better detection of the targets on the pictures. To capture the global behavior of the frame, 30 big targets were painted at a distance 500 mm (Figs. 1b and 2b). To the local analysis of the beam-column connection, a regular grid of circular target was painted in a rectangular area (1450 × 1100 mm^2) at 50 mm in both direction [20].

Results and discussion:

The load deflection diagram of the tested frame in three different phases is presented in Fig. 5a. The corresponding load at 30 mm deflection of each individual phase is also presented in Table 3. According to the results presented in Fig. 5a, the behavior of the tested frame with and without masonry infill wall is almost linear up to a certain limit load. While, the linear part of the load-deflection curve of the F-Tr frame is also more lengthened compared with the bare frame, due to higher lateral strength provided by masonry units.

The tested carried out for the F-Tr frame was stopped at a deflection of 32 mm due to the failure of the masonry wall (crushing of compressive strut). From the obtained results, the $\Delta F_{\max} / F^{F-\mathrm{Re}}_{\max-30} = (F_{\max-30} - F^{F-\mathrm{Re}}_{\max-30}) / F^{F-\mathrm{Re}}_{\max-30}$ ratio was evaluated, and the values are indicated in Table 4, where $F^{F-\mathrm{Re}}_{\max-30}$ and $F_{\max-30}$ are the maximum load capacity of the reference frame (F-Re) and of the other frames at deflection of 30 mm, respectively. It was calculated the $\Delta F / F^{F-\mathrm{Re}}$ ratio where ΔF is the increase in load provided by infilled masonry walls ($\Delta F = F - F^{F-\mathrm{Re}}$), being $F^{F-\mathrm{Re}}$ the load capacity of the reference frame, and F the corresponding (for the same deflection) load capacity of the other infilled frame. The $\Delta F / F^{F-\mathrm{Re}}$ (%) *vs.* corresponding deflection curves at loaded section are depicted in Fig. 5b, and their maximum values $(\Delta F / F^{F-\mathrm{Re}})_{\max}$ are presented in Table 3. According to the results presented in Fig. 5b and Table 3, the initial stiffness can increase approximately 500% for the infilled frame compared to the bare frame (F-Re). According to Fig. 5b, it can be concluded that the F-Re and F-B tests had a similar behavior in elastic regime. The negative results obtained for F-B test is because of the micro cracks formed at the top and bottom beams when the frame was tested in the first phase.

The increasing frame deflection at the point of the missing column support is restrained due to the structural resistance of the masonry infill wall and its composite action with the surrounding RC frame that cause developing interaction forces between the infill wall and the surrounding frame [20]. At a vertical displacement of around 7.5 mm in F-Tr frame, a horizontal crack was formed between the masonry brick and RC frame (red ellipse in Fig. 6a). By increasing the load, the crack propagated and gradually widened in the later stage. After the formation of the main crack a few cracks formed and propagated into the masonry bricks (Fig. 6b) due to compressive arch. Fig. 6b shows the crack pattern of the infilled frame at a vertical displacement of 30 mm. The failure of the bare frame was governed by formation of plastic hinges at the beams ends as expected due to design approach of the frame according to EC8 (strong columns and weak beams) [14].

Toughness indicator, as a measure of the energy absorption capacity, is obtained for the tested frame up to 10 mm and 30 mm by determining the area behind the force *vs.* deflection curve (Table 4). According to results presented in Table 4, the tested frame in the first phase (F-B) had a behavior like that the tested frame in the last phase (F-Re) in elastic regime since both had a same amount of toughness up to 10 mm and 30 mm. The toughness of F-Tr frame is approximately 4 times and 2.7 times higher than the bare frame (F-Re) up to a deflection of about 10 mm and 30 mm, respectively. That indicates the contribution of masonry infill wall to increase the strength and stiffness of RC frame.

Photogrammetry:

As mentioned in introduction, the behavior of the frame under vertical load was monitored using photogrammetry technique [21]. For this purpose, five high resolution cameras were used in 5 different stations. Four cameras (#1 to #4) used to monitor the local behavior of the joints and interaction between wall and surrounding RC frame and one camera (#5) just monitored the global behavior of the frame. To recognize the possibility of error in photogrammetry technique, the results obtained by LVDT and photogrammetry for each phase in different stage are presented in Table 5. According to the results, the average ratio of measured vertical displacement by LVDT to photogrammetry technique is 0.98 with a COV of 7.86%, that shows the accuracy of the photogrammetry technique.

Figure 7 shows the deformation of the frame in each load stage. The deformation was obtained by measuring the displacement in each painted global target in beams and columns. According to the results, it can be concluded that the columns did not show significant rotation in any of each test that can be explained by the fact that there was a complete fixed support in the other side.

As mentioned before, the loss of a column causes significant increase of the frame deflection that is restrained by the shear resistance of the masonry infill wall, thus developing interaction forces between the infill wall and the surrounding frame [20]. According to analysis of F-Tr test at station #3 and stage #5 (Fig. 8a), on the initial stage of loading, the load had equally distributed through the column and beam as well as the masonry wall, while this distribution was not more uniform after formation of the horizontal crack and separation of the infill wall from the beam (Figs. 8b)

Macro modeling:

Macro finite element model analysis was carried out by OpenSees [15] to investigate the behavior of the tested frame in different phases. Parametric studies were carried out to further study the effect of the longitudinal reinforcement ratio on the load carrying capacity. The column was pushed up by displacement to simulate loading strategy in the experimental program.

Constitutive model and its predictive performance

Beams and columns were modeled using force-based elements, with five integration points along each element length and Corotational Coordinate Transformation for geometric nonlinearity. Three layers of fibers in the cover region and twenty layers of fibers in the core region were assigned to model the beam and column cross sections. The values of the parameters that define constitutive models used in numerical simulation were calibrated by simulating the tested frame, considering the properties obtained in the experimental programs for the characterization of the relevant properties of the used materials, and the suggestion of the EC2 [13].

The material "Concrete02" available in OpenSees was used for the concrete frame. The constitutive model of this block is presented in Fig. 9a, where f_{pc} is concrete compressive strength at 28 days, ε_{psc0} is concrete strain at maximum strength, f_{pcu} is the crushing strength, ε_{psu} is the strain at crushing strength, λ is the ratio between unloading slope at ε_{psu} and initial slope, f_t is the tensile strength, and E_{ts} is the tension softening stiffness (absolute value). More information on this constitutive model can be found in Opensees [15]. The values of this diagram are presented in Table 6. As mentioned before, the concrete compressive strength of frame is $f_{pc} = 44.23$ MPa. Then other parameters can be found based on EC2 [13], ε_{psc0} and ε_{psu} are equal to 0.0027 and 0.0035, respectively. f_{pcu} is 30 MPa, and f_t = 2.7 MPa. λ and E_{ts} were calibrated by simulating the tested frame.

The material "Steel01" available in Opensees is used to define the reinforcement of columns and beams. This is a elasto-plastic with hardening model, where the stiffness of the post-yield branch is controlled by the strain-hardening ratio b, given by the ratio between the post-yield tangent and initial tangent (Fig. 9b). More information on this constitutive model can be found in Opensees [15].

The contribution of the masonry walls was implemented using the eccentric truss element as suggested by Al-Chaar and Lamb [16], the strut width a for a solid infill can be estimated as follows:

$$a = 0.175 \times D \times (\lambda L)^{-0.4} \tag{3}$$

where λ is:

$$\lambda L = L \times \sqrt[4]{\frac{E_m \times t \times \sin(2\theta_b)}{4 \times E_c \times I_{beam} \times l}} \tag{4}$$

The strut width (a) is dependent of the relative bending stiffness between the beams and the masonry panel (λL). The distance Lb represents the length of formation of plastic hinges and is determined geometrically (Figure 10):

$$Lb = \frac{a}{\sin(\theta_b)} \tag{6}$$

where:

$$\tan(\theta_b) = \frac{h}{l - 2Lb} \tag{8}$$

In Equation 4, L is the distance between the columns midlines, l is the masonry panel width, t is the panel thickness, E_m refers to the modulus of elasticity of masonry, E_c represents the modulus of elasticity of concrete, I_{beam} is the moment of inertia of the beams. In Equation 3, D is the diagonal length of the panel. The strut material is assumed Kent-Scott-Park model [15]. According to the test results of masonry bricks, maximum compressive strength for the model is assumed 1.2 MPa with corresponding strain of 0.0022, the crushing strength is 0.1 with corresponding strain of 0.005.

A model of infill wall (F-Tr) is presented in Fig. 11. In this model, it was assumed that the masonry infill had a compressive linear elastic behavior and do not resist tension stresses. By using the properties obtained from the mechanical properties of masonry and deriving from inverse analysis the data for the masonry infill model was found. The experimental and the numerical relationships between the applied load and the deflection at the loaded section for the tested frame in different phases are presented in Fig. 12. This figure also shows a comparison between experimental and numerical simulation in terms of strain-load relationship. This figure shows that the numerical model can capture with high accuracy the deformational response and strain in longitudinal bars of the tested frame in different phases.

Parametric Study:

Due to the good performance of the adopted model in simulating the behavior of the structure, confirmed in the previous section, the model was adopted to study the influence of percentage of longitudinal reinforcement ratio in beams and columns on the load carrying capacity of the frame. For this purpose, the area of the longitudinal reinforcement implemented in OpenSees [15] was changed to simulate the effect of the longitudinal reinforcement. The geometry of beams and columns, the material properties of concrete, the support and load conditions, and the length of the elements were those adopted in the previous section.

Influence of longitudinal reinforcement ratio of the columns

In this case, the influence of columns longitudinal reinforcement ratio on the load-deflection is investigated. For this purpose, two different percentage of column reinforcement ratio are assumed: 1% and 6%, the first one is lower and the last one is higher than the one corresponding to the percentage of the columns reinforcement of the tested frame.

The obtained results, depicted in Fig. 13, show that the high percentage of reinforcement does not have effect on the load carrying capacity of RC frame. Because, as mentioned before, the failure of the bare frame was governed by formation of plastic hinges at the beams ends, then increasing the longitudinal reinforcement of columns does not have influence on the load carrying capacity. The load carrying capacity of the with 2.7% and 6% longitudinal reinforcement are around 23% higher than the frame with 1% of longitudinal reinforcement.

Influence of longitudinal reinforcement ratio of the beams

Figure 14 shows the obtained results for different beams longitudinal reinforcement ratio. Two different percentage of beams longitudinal reinforcement are assumed: 0.5% and 1.5%, the first one is lower and the last one is higher than the one corresponding to the percentage of the beams reinforcement of the tested frame.

As expected, by increasing the longitudinal reinforcement ratio of the beams, the load carrying capacity and ultimate deflection is increased. The load carrying capacity of frame with 1.5% longitudinal reinforcement is around 19% and 72% higher than the one in frame with 0.96% and 0.5% longitudinal reinforcement, respectively.

Influence of longitudinal reinforcement ration of both beams and columns

Figure 15 shows the obtained results for different beams and columns longitudinal reinforcement ratio. In this study, two different percentage of longitudinal reinforcement are assumed: 50% more and 50% less than longitudinal reinforcement ratio of the tested frame.

In the first case, RC frame with 50% more longitudinal reinforcement, and similar to the experimental test observations, masonry has significant impact on frame stiffness and negligible influence on the strength. In the second case, RC frame with 50% less longitudinal reinforcement, it is clear the significant impact of the masonry infill wall on both the stiffness and the strength of the RC frame. Therefore, it can be stated that, for current RC frames, it is expected that masonry infill wall has a significant contribution to the structural robustness, namely by providing an alternative load path in the event such as a column failure.

Conclusions:

Progressive collapse of multistory buildings can occur after local damage to a member typically initiated by extreme dynamic events such as earthquake, natural disasters, and terrorist attack.

This paper studied the effectiveness of traditionally bricks masonry units on the behavior of reinforced concrete (RC) frame subjected to vertical load. The frame was designed according to Eurocode 2 (EC2) and Eurocode 8 (EC8). The frame was designed to have strong columns and weak beams. The results enhance the understanding regarding the behavior of frame with and without infill wall and its contribution on the structural robustness. According to the results obtained by experimental results, it can be concluded that:

- The quality of the mortar and workmanship have a strong influence on the masonry shear strength, however, not a significant influence on the compressive strength.
- The infill wall plays a major role in maintaining the structural system's integrity and reducing the likelihood of a progressive collapse and therefore its contribution should be incorporated in the structural model.
- Traditionally infill wall can significantly increase stiffness and load carrying capacity of a RC frame at a certain deflection around 220% compare to a frame without any infill wall.
- The masonry walls can increase the energy absorption and that the toughness of the infilled frame 270% higher than the ones without infilled wall.
- Compared with the bare frame, the infilled frame has a larger initial stiffness but lower ductility.

Artificial vision system was used as a structural monitoring system. This technique provides important data in terms of interaction of infill wall and surrounding RC frame. The loss of a column causes developing interaction forces between the infill wall and the surrounding frame which this interaction and its propagation have been clearly shown by artificial vision.

A numerical simulation was carried out using the OpenSees software. The values of the constitutive models were calibrated considering the properties obtained from the tests of the material properties, inverse analysis, and the suggestion of EC2. After having been demonstrated that the model is capable of simulating, with high accuracy a parametric study was carried out to investigate the influence of the percentage of longitudinal reinforcement ratio in beams and columns on the load carrying capacity of the infilled frame. Results presented that:

- When the failure is governed by formation of plastic hinges at the beams the high percentage of reinforcement of column does not have effect on the load carrying capacity of the frame.
- while high percent of longitudinal reinforcement of beams can significantly increase of the load carrying capacity of the farm.
- The frame reinforcement details have a pronounced effect on the frame performance.

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