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Influence of Infill Properties on Ductility of RC Existing Frames

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Abstract*: The damage observed in past earthquakes, has shown as the presence of infills significantly influences the seismic performance of RC moment resisting frames. In fact, the presence of full infilled frames, with regular distribution, could improve the seismic response and mitigate horizontal displacements. On the other hand, an irregular distributions of infill could dramatically modify the stiffness and resistance of structures resulting in a brittle collapse mechanism related with soft story or torsional effects. In this study it has been evaluated the seismic behaviour of frames designed to bear only gravity load; a simulated design procedure has been adopted, according to code provisions and design practices in force in Italy between 1950s and 1970s. A parametric study has been performed to take into account the typical mechanical properties of masonry available in Italy. Infill panels have been modelled by means of equivalent struts. A pushover analysis has been carried out to evaluate the capacity curves and collapse mechanisms of infilled frames. The performed analysis allowed to analyse the influence of infill properties on the ductility of existing RC frames. The results emphasize as the properties of masonry infills should be adequately evaluated before doing a seismic analysis as the masonry shear strength significantly influences the global seismic behaviour of RC frames.

***Keywords*—Infilled R.C. frames, masonry infills, ductility R.C. frames, seismic vulnerability.**

I. INTRODUCTION

Reinforced concrete frames with masonry infill are a widespread structural system worldwide. Generally, infills are considered as partition elements without any structural function. Their influence on the lateral stiffness, base shear as well as on possible brittle failure mechanisms of joints and columns, due to local interaction between panels and adjacent structural elements, are often neglected. The observation of post-earthquake damages on reinforced concrete buildings have clearly shown that the presence of infills may significantly affect the performance of structures, both in terms of seismic capacity and demand. Damages observed during L'Aquila (Italy) earthquake (2009) and Lorca (Spain) earthquake (2011) documented as many buildings suffered of structural damage or collapsed for irregularity induced by infills [1,2]. The shear and bending moment variation induced in columns by the infill panels could be of particular importance; these variation could significantly modify the behaviour expected at the design stage. The observed damages and experimental studies highlight the possible brittle failure of the beam-columns joints due to unforeseen diagonal stresses induced by infills [3]. Shing and Mehrabi [4] have analyzed some of the possible failure modes of infilled frames. According to the authors opinion, the behaviour of an infilled frame is heavily influenced by the interaction of the infill with its bounding frame, therefore five main failure mechanisms can be identified. For low values of lateral loads, the panels act as a monolithic element and their behaviour strictly depends on the interface condition between panel and frame. With increasing of the load the infill wall tends to partially separate from the frame and a compression strut mechanism occurs; the cracking and damage in the panel gradually increase up to the attainment of the maximum lateral strength of the infilled frame. Nowadays in literature some methodologies are available to simulate the behaviour of infilled structures; experimental and numerical studies demonstrated that a diagonal strut with appropriate geometrical and mechanical characteristics could be a good solution to take into account the influence of infills in the seismic behaviour [5]. Different authors proposed valuable relationships to evaluate the geometrical properties of struts that must be included in the numerical models [6,7]. The definition of seismic behaviour of infill as well as the identification of the more probable collapse mechanism is strongly related with the mechanical properties of the masonry. The quality of mortar and masonry units are very important and the main properties that must be identified to correctly analyze the infill influence are the elastic and shear modulus, shear and compressive strength. The shear strength results as the combination of two mechanisms, namely bond strength and friction resistance between the mortar and the bricks; different authors proposed range of values that could be attributed to this mechanical property [8,9,10]. In this study the seismic behaviour of gravity load designed frames has been investigated; the frames have been defined by means of a simulated design procedure according to code prescriptions and design practices in force in Italy between 1950s and 1970s. The considered frames have height varying between 7 and 34 meters. A parametric study has been performed to take into account the typical mechanical properties of masonry available in Italy. The performed analysis allowed to analyze the influence of infill properties on the ductility of existing RC frames.

II. DESCRIPTION OF STRUCTURAL MODELS

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The influence of infill walls have been investigated on RC frames designed according to codes available in Italy between 1950s and 1970s [11]. The height of the frames have been varied between 7 and 34 m, with an inter-story height equal to 3.5 m (Fig. 1). The frames consist of four spans in which the length of bays is assumed equal to 4.5 m. A total of five frames have been considered.

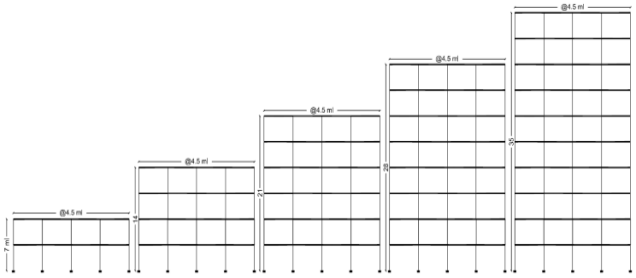

Fig. 1 Models of RC Frames

The concrete compressive strength, f_c, is assumed equal to 25 MPa, the elastic modulus of concrete has been calculated according to the formula proposed in EC8: $22000*(f_c/10)^{0.3}$ [12]. For longitudinal rebars and stirrups have been assumed smooth steel with yield strength equal to 375 MPa. The design values for maximum concrete compressive stress and steel have been evaluated according to R.D.L. (Royal Law Decree) 2229 of 1939 [11]. The dead loads have been calculated considering a width of influence equal to 3 m for each beams and assuming a height of the slab equal to 25 cm. The live loads are assumed equal to 3 kN/m^2. The column dimensions have been calculated only for axial loads, while the beams have been designed to bear bending and shear due to gravity loads. In literature numerous formulations are available to take into account the influence of infill walls on the linear and nonlinear behaviour of infilled RC frames. In this study the single compressive strut model has been adopted; the width of the struts have been calculated according to the relationship proposed by Mainstone [13], Eq.1:

$$\frac{w}{d} = 0.175\lambda_h^{-0.4} \tag{1}$$

where λ_h is the relative panel-to-frame stiffness proposed by Smith [14], Eq.2:

$$\lambda_h = \sqrt[4]{\frac{E_w t_w \sin 2\vartheta}{4EIh_w}} \tag{2}$$

w is the width of the equivalent strut; d represents the diagonal length of the panel; E_w, t_w, and h_w are the elastic modulus, the thickness and the height of the masonry infill panel, respectively; EI represents the stiffness of RC columns; θ indicates the angle whose tangent is the height-to-length aspect ratio. To evaluate the nonlinear behaviour of RC elements, the lumped plasticity approach has been followed: the structural elements have been modelled by elastic elements with two rotational hinges at both ends of the beams. The ultimate plastic rotation was determined according to EC8 (Eq.3) [15]:

$$\vartheta_{um}^{pl} = \vartheta_{um} - \vartheta_y = \frac{1}{\gamma_{el}} \cdot 0.0145 \cdot (0.25^{\upsilon}) \cdot \left[\frac{\max(0.01;w')}{\max(0.01;w)}\right]^{0.3} \cdot f_c^{0.2} \cdot \left(\frac{L_v}{h}\right)^{0.35} \cdot 25^{\alpha\rho_{sx}\frac{f_{yw}}{f_c}} \cdot (1.275^{100\rho_d}) \tag{3}$$

where: γ_{el} is equal to 1.5 and 1.0 for primary and secondary seismic elements, respectively; $\upsilon=N/(A_c \cdot f_c)$ is the dimensionless axial force; $w=(A_{st} \cdot f_y)/(A_c \cdot f_c)$ is the mechanical reinforcement ratio of the tensile (or compression) longitudinal reinforcement; fc is the compressive strength of the concrete; f_y is the yield strength of the longitudinal reinforcement; L_v is the moment/shear ratio at the end section, its value was calculated from stress distribution obtained by a linear dynamic analysis; h is the depth of the cross-section; $\rho_{sx}=A_{sx}/(b_w \cdot s_h)$ is the geometrical percentage of stirrups, s_h is the stirrups spacing; f_{yw} is the tensile strength of shear reinforcement, and ρ_d is the percentage of inclined shear reinforcement. The moment-rotation relationship assumed for the plastic hinges is shown in Fig. 2. The relation is considered linear elastic up to the yielding rotation (θ_y) while a perfectly-plastic behaviour is assumed between the yielding and ultimate rotation (θ_{um}). After the ultimate rotation is reached the moment capacity of the cross section instantaneously falls to $0.2M_y$ and it remains constant until a rotation of $3\theta_{um}$ is attained. According to Rozman and Fajfar [16] this last value is chosen arbitrarily and it does not affect the results but it is very important for the numerical stability of the analysis.

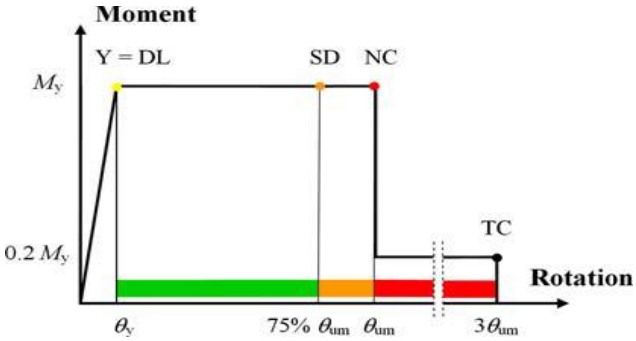

Fig. 2 The Moment-Rotation Relationship of a Plastic Hinge [16]

The infill panels are modelled with a single compressive strut. The nonlinear behaviour is taken into account with the force-displacement relationship proposed by Fardis and Panagiotakos [6]. The proposed lateral force-displacement envelope is composed by four branches and has been validated by experimental analysis of infilled frames (Fig. 3).

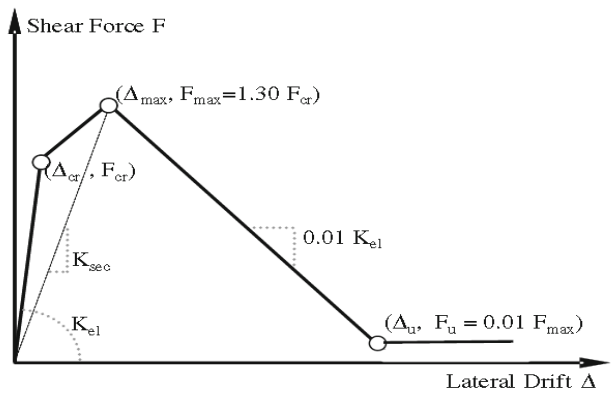

Fig. 3 Lateral Force-Displacement Relationship Proposed in [6]

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The first branch is representative of the initial shear behaviour of uncracked infill panels; the second branch is related with the development of the equivalent strut due to the disconnection between the frame and the panel, this branch follows the first cracking, up to the point of maximum strength. The third branch correspond with the instable behaviour of the infill panels up to the residual strength; finally, the fourth branch, represented with an horizontal branch, is the residual strength and it is assigned for the numerical stability of the analysis. The main parameters that allow to define the F-d relationship are reported in the following:

- The initial stiffness of uncracked infilled panels:

$$K_{el} = \frac{G_w \cdot t_w \cdot l_w}{h_w} \quad (4)$$

where G_w is the elastic shear modulus of infill material and l_w the length of infill panel;

- The shear cracking strength:

$$F_{cr} = f_{tp} \cdot t_w \cdot l_w \quad (5)$$

where f_{tp} is the shear cracking stress;

- The stiffness of the second branch of the curve:

$$K_{el} = \frac{E_m \cdot b_w \cdot t_w}{d} \quad (6)$$

where E_m and d_w are the elastic modulus and the clear diagonal length of infill panels, respectively, while b_w is the width of the diagonal strut evaluated according to the relationship proposed by Mainstone [13];

- The maximum strength, defined as:

$$F_{max} = 1.30 \cdot F_{cr} \quad (7)$$

- The stiffness in the post-capping degradation branch, depending on the elastic stiffness through the parameter α:

$$K_{soft} = -\alpha \cdot K_{el} \quad (8)$$

where α assume values between 0.005 and 0.1.

- The residual strength assumes values between 0 and $0.1F_y$. In the present study α=0.01 and a residual strength equal to $0.01F_{cr}$, are assumed.

The strut are pin-connected to the column at a distance l_{column} from the face of the beam. This distance is defined by Eq. 9-10 and it is calculated considering the strut width without any reduction factors:

$$l_{column} = \frac{b_w}{cos\theta_{column}} \quad (9)$$

$$tan\theta_{column} = \frac{H_w - \frac{b_w}{cos\theta_{column}}}{L_w} \quad (10)$$

The panel thickness is assumed equal to 200 mm, this values can be considered as a typical values for non-structural infill walls [17].The shear strength of masonry infill has been varied to study the influence of masonry properties on the global seismic behaviour of infilled frames. Four shear strength, τ_{inf}, have been taken into account, in particular: 0.1,0.2,0.3 and 0.4 MPa. The elastic modulus has been assumed equal to 1200 MPa. According to the studies of Tomazevic [18], no correlation is assumed between the shear modulus and the shear strength.

Two configurations have been considered for the frames:

- Bare frame configuration (indicated with B), this configuration is assumed as references case;
- Full infilled configuration (indicated with F), the infill panels are uniformly distributed within the frame, and any opening is considered.

To identify each case study a specific label has been assigned. The first letter indicate the frame configuration (B if bare frame, F if infilled frame), the number that follows identifies the frame height (2,4,6,8,10 stories), the third symbol provide information about the shear strength of the infill, τ_{inf} (S1=0.1 MPa, S2=0.2 MPa, S3=0.3 MPa, S4=0.4 MPa). Globally 25 have been analysed.

III. RESULTS

A pushover analysis has been carried out for each considered frame configuration. The results of the analysis allowed to define the capacity curves, and to investigate about the influence of infill properties on the collapse mechanism and the ductility of the frames. The ductility has been evaluated as the ratio between the displacement corresponding with the first failure in a structural RC element and the displacement at which the first yielding occurs. The results are presented in the following for each analysed height. All analyses have been performed by means SAP2000 [19].

- **10-Stories frames (H=35 m)**

The behaviour of the bare frame is previously analyzed to evaluate the influence of the infill and of its mechanical properties. The ductility results equal to 7,8. The frame collapses following a soft-story mechanism at the 6th story (Fig. 4). This collapse is due to the reduction of columns section along the height. The introduction of infill panels significantly changes the seismic behaviour of the frame, as demonstrated by the capacity curves (Fig. 5) and the collapse mechanisms.

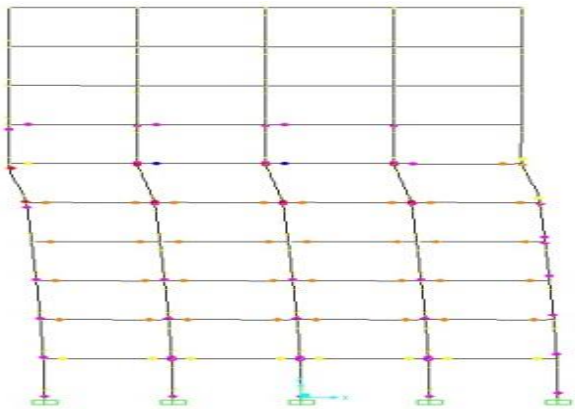

Fig. 4 Collapse Mechanism. B_10 Frame

Yielding Rotation Severe Damage Rotation Near Collapse Rotation

As expected, the infills influence the global stiffness of the frame as well as the base shear force. Because of the elastic modulus is constant the initial elastic stiffness doesn't change, while the maximum shear force increases with respect to the bare frame from 26,4% up to 200,4% for the model in which the shear strength of masonry (τ_{inf}) is equal to 0.1 and 0.4 MPa, respectively.

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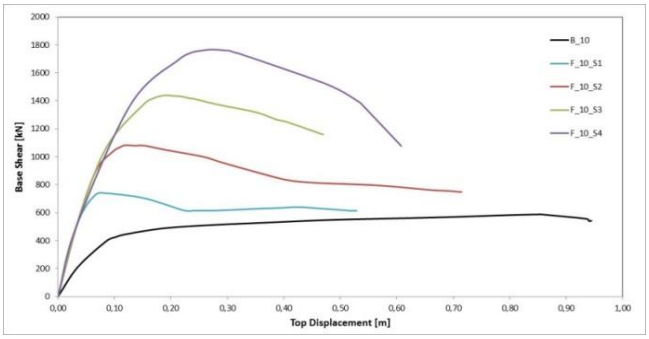

Fig. 5 The Capacity Curves at Different Values of τinf (F_10)

In Fig. 6 the collapse mechanism for models F_10_S1 and F_10_S4 are reported. It is worth to be noted that the collapse mechanisms of infilled frames totally change if compared with that of the bare frame. In fact, from a qualitative standpoint, the presence of infill walls lead to a more uniform inter-storey displacement and to avoid the soft-story mechanism at the upper floors. For low values of τ_{inf} the failure mechanism involves more storey, while for high values the collapse involves only the lower floors. In particular, for the examined frames, the collapse mechanism occurs at the 2nd and 3rd floors for a value of shear strength equal to 0.4 MPa.

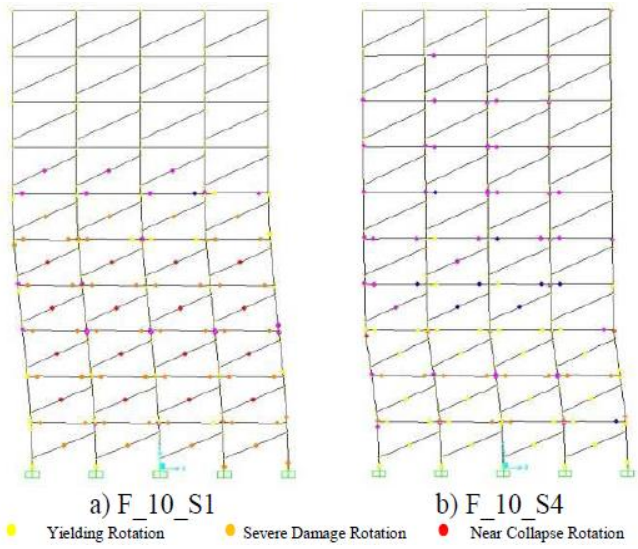

Fig. 6 Collapse Mechanism of Infilled Frames with H=35m

In Table 1 the ductility values (μ) are reported.

Table 1 Ductility 10-Storey Frames

	B_10	F_10_S1	F_10_S2	F_10_S3	F_10_S4
μ	7.8	11.3	9.3	5.4	3.6

While the maximum shear force significantly increases with increasing τ_{inf} the ductility doesn't exhibit the same trend. Only for low masonry resistances (S1-S2) the ductility increases if compared with that of bare frame. This result is also confirmed looking at the failure mechanisms; for low values of τ_{inf} the structural damages are better distributed along the height, while for τ_{inf} equal to 0.3 and 0.4 MPa the damages are concentrated at the lower floors.

- **8-Stories frames (H=28 m)**

Also in this case, the failure of the bare frame involves an upper story. In particular, the failure occurs at the 4th floor with a soft-story mechanism. As for the previous case, this behaviour is due to the variation of columns cross section within the height of the frame. The capacity curves plotted for the 8-storey frames show a similar behaviour of the curves obtained for the 10-stories frames (Fig. 7). Due to the strength degradation related with the failure of infills, the curves of infilled frames tend to reach in same cases the same values of the base shear obtained for the bare frame; in particular this behaviour occurs for F_8_S1 and F_8_S2. The maximum strength increases of 35%, 102%, 169%, 233% for the infilled frames with τ_{inf} equal to 0.1 MPa, 0.2 MPa, 0.3 MPa and 0.4 MPa, respectively.

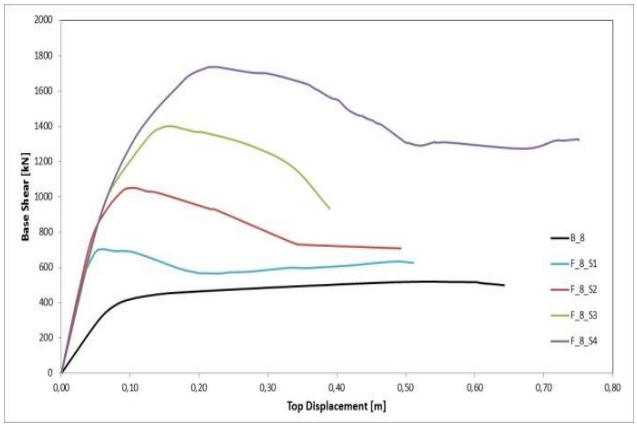

Fig. 7 The Capacity Curves at Different Values of τinf (F_8)

As reported in Fig. 8 the presence of infill walls drastically changes the collapse mechanisms. For a low value of the masonry shear strength, τ_{inf} (Fig. 8a) the collapse involves the first 3 floors; the ultimate rotation is reached at the upper hinges of the columns of the 3rd floors and at the ground hinges. All infills included in those floors attain the failure. On the other hand for the highest value of τ_{inf} the collapse involves essentially the 1st and 2nd floors. A soft-storey mechanism is observed at the 2nd floors, in fact only at that floor the infills attain the failure, while at the 1st and 3rd floor the maximum capacity of the infill panels is reached but without collapsing.

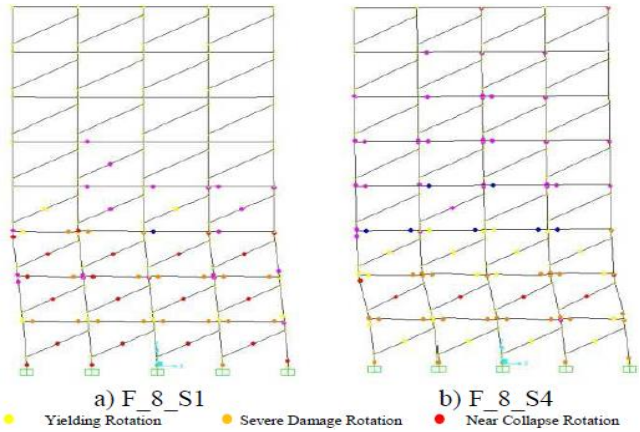

Fig. 8 Collapse Mechanism of Infilled Frames with H=28m

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The values of ductility (Table 2) indicates that for low and medium values of masonry shear strength the infill increases the inelastic capacity, while higher values of τ_{inf} cause a negative effect on the global ductility.

Table 2 Ductility of Full Infilled 8-Storey Frames

	B_8	F_8_S1	F_8_S2	F_8_S3	F_8_S4
μ	9.0	11.8	9.4	6.5	5.1

- **6-Stories frames (H=21 m)**

In the bare frame with height equal to 21 m (B_6) the collapse mechanism involves the 1st and 2nd floor, in particular a soft-story mechanism occurs at the 2nd floor. The maximum base shear strength and the corresponding displacement are equal to 417,5 kN and 35,1 cm, respectively. As for the other cases the ductility has been evaluated referring to the capacity curve, looking at the first yielding and at the first RC element that reaches the maximum rotational capacity; μ results equal to 6,0. In Fig. 9 the capacity curves of the MDOF (Multi Degree Of Freedom) systems varying the masonry shear strength are plotted.

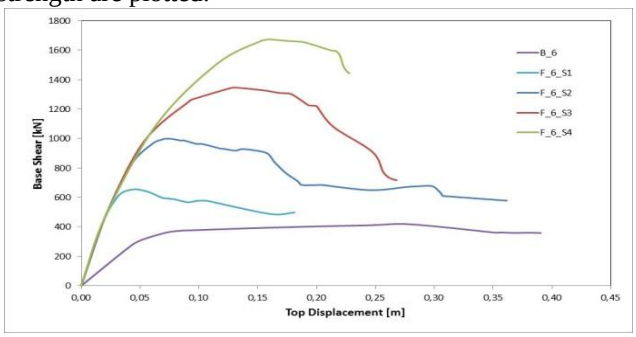


Fig. 9 The Capacity Curves at Different Values of τinf (F_6)

From the capacity curves it is possible to observe a reduction of plasticization with the increasing of τ_{inf}. This behaviour is also confirmed by the values of μ (Table 3). Only for F_6_S1 the ductility is higher than that for B_6, for the remaining cases μ assumes lower values. Comparing the values assumed by μ, with those obtained for frames with 10 and 8 stories, an higher reduction of ductility may be observed. This behaviour is related with the higher stiffness of shorter frames as also confirmed by the collapse mechanisms (Fig. 10).

Table 3 Ductility of Full Infilled 6-Storey Frames

	B_6	F_6_S1	F_6_S2	F_6_S3	F_6_S4
μ	6.0	7.1	4.1	3.3	2.8

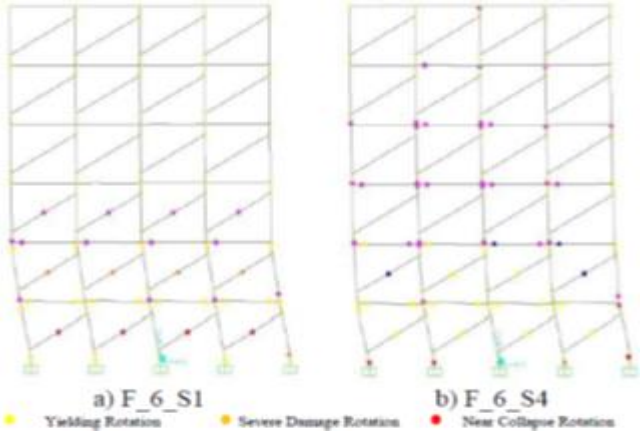
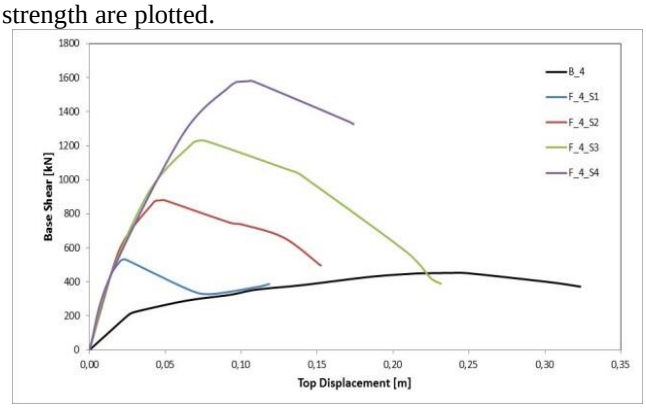


Fig. 10 Collapse Mechanism of Infilled Frames with H=21m

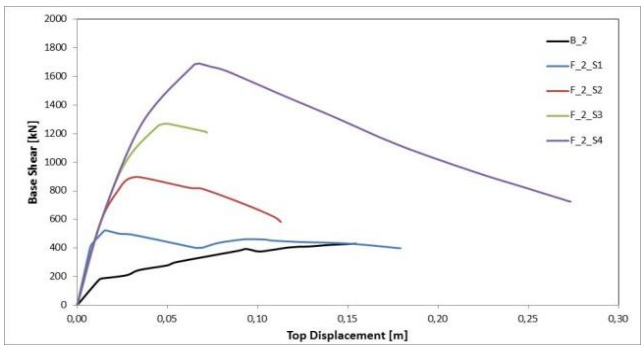
The collapse mechanism involves the 1st and 2nd floors in the case of τ_{inf}=0.1 MPa, while only the 1st floor for τ_{inf}=0.4 MPa. It is worth to be noted that for τ_{inf}=0.1 MPa the infills reached the failure; the same observation cannot be made for τ_{inf}=0.4 MPa; in this case the RC element collapsed before the infills failure.

- **4 and 2-Stories frames (H=14-7 m)**

In this section the results for the frames with 4 and 2 stories are summarized. In both cases the failure occurs always at the 1st story. The ductility of bare frames are 12.3 and 14.7 for B_4 and B_2, respectively. In Figure 11a-11b the capacity curves of infilled frames at different values of the infill shear strength are plotted.



A) Capacity Curves for F_4



B) Capacity Curves for F_2

Fig. 11 Capacity Curves for Frames with H=14 and H=7 m

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Analysing the collapse mechanisms it can be observed that the failure occurs in all cases at the 1st floor. For τ_{inf}=0.1 MPa the infill at the 1st floor fails before the ultimate rotation of the RC elements has been reached, while in all other cases the structural collapse occurs before the failure of the infill panels. In Tables 4 and 5 the values of ductility for the full infilled frames are reported.

Table 4 Ductility of Full Infilled 4-Storey F rames

	B_4	F_4_S1	F_4_S2	F_8_S3	F_4_S4
μ	12.3	8.3	5.4	4.9	2.9

Table 5 Ductility of Full Infilled 2-Storey Frames

	B_2	F_2_S1	F_2_S2	F_2_S3	F_2_S4
μ	14.7	12.8	10.4	7.9	5.6

In all cases μ assumes values lower than that of the bare frame. This behaviour is due to the excessive reduction of ultimate displacement induced by the stiffness of infill walls.

IV. CONCLUSION

In the present work the influence of infills shear strength on the seismic behaviour of RC existing frames has been studied. As expected, the stiffness and strength of frames increase with the introduction of infill walls; in particular the capacity curves show as the increase of shear strength of masonry infills significantly influences the base shear force. The collapse mechanisms are also changed by the presence of infill panels. For low values of masonry shear strength a more uniform damage distribution has been observed, in particular for higher frames. The increase of infill shear strength causes a concentration of damage at the lower stories (1st and 2nd). Despite the resistance of frames increases with the masonry shear strength, the ductility doesn't show the same trend. For frames with 10-8-6 floors the ductility increases for low values of masonry shear strength, while in the other cases the ductility is negatively influenced by the infills contribution. As already demonstrated by previous studies, this wok allows to conclude that the influence of infills on stiffness and strength can't be neglected in the seismic evaluation, in particular referring to existing RC frames. The properties of masonry infills should be adequately evaluated before proceeding with the analysis as the masonry shear strength significantly influences the global seismic behaviour of RC frames.

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