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THE INFLUENCE OF INFILLED PANELS IN RETROFITTING INTERVENTIONS OF EXISTING RC BUILDINGS: A CASE STUDY

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During the seismic events occurred in the last decades, the existing RC building stock has often exhibited a significant brittle collapse of the nodal connections and of the secondary elements such as infill walls. In the attempt of mitigating this vulnerability, it is becoming quite common the attempt of intervening on the critical elements with generalized strengthening interventions, even without complete and specific seismic calculations. This paper presents some considerations about the role of strengthening interventions involving the solidarization of the infill panels to the RC frame on the seismic retrofitting of existing buildings. Within this context, an appraisal of the actual resulting displacement capacity, and of possible alteration induced on the global collapse mechanism is provided. With reference to a real case study concerning a school building (which was part of a wide vulnerability assessment investigation performed in the Province of Foggia, Italy), an appraisal of the effect of strengthening interventions is here discussed. In particular, the seismic analysis by non linear static procedure has been performed and critically discussed, both on the reinforced configuration and on the original structure.
Keywords: Masonry Panel, Infilled Frames; Seismic Assessment; Nonlinear Static Analysis; Equivalent Strut Models, Existing Building.

1. Introduction

The seismic vulnerability of existing buildings is surely a direct consequence of the deterioration of materials and poor constructive details, but, most of all, is related to the theoretical and technical reference framework according to which the structural design has been performed. This is especially true when speaking of seismic design concepts that have made substantial progress in the last decades, whereas the structures designed and constructed according to old technical standards have revealed to have an inadequate seismic performance.

Recent earthquakes in Italy and all over the world have clearly highlighted that the seismic response of existing RC framed buildings are strongly influenced by the presence of infill panels, which significantly contribute to the global strength and, consequently, to the formation of the collapse mechanism **(Mezzina et al., 2009; Loh, Tsai, Chung &Yeh, 2003)**. Another important effect is related to the regularity of the structural geometric configuration, both in plan and elevation, which might be significantly altered by the presence of very stiff and irregular infill walls, triggering unexpected storey mechanisms and/or torsion effects **(Korkmaz, Demir & Sivri, 2007; Karayannis, Favvata, & Kakaletsis, 2011)**. The additional problem related to mechanisms that directly involve infill panels is that they may be brittle, to an extent more or less noticeable depending on the specific mechanical properties and stiffness and of the numerical model adopted **(Uva, Porco & Fiore, 2012)**. In current design practice of RC framed structures, the contribution in terms of stiffness and strength of non-structural elements like infill walls is not included in the numerical model. Such an approach is actually contained in the present Building Codes **(D.M. 14/01/2008; CEN, 2005)**, in which it is suggested that infill panels, under in-plane horizontal actions, should be considered as completely disconnected by the surrounding frame. Indeed, one of the reasons behind this choice is the lack of effective and well-established models for the simulation of the interaction between frame and infill elements. Nevertheless, the relevance of the possible contribution in terms of stiffness and strength is recognized: both Italian NTC (*see §7.2.6*) and Eurocode 8 (*see §4.3*) prescribe the incorporation of the infill panels in the structural model whenever their presence significantly increases the lateral stiffness of the structure. Actually, this is only a conceptual performance requirement, with no specific indication about the modelling approach: the same statement of "*significant*" increase is not associated to a precise threshold level or specific criterion. In the awareness that damage can be significantly clustered in the storeys where infill panels are lacking or have a reduced extension, the Italian Building Code (*paragraph: §7.2.3*) suggests that the design actions shall be increased by *25%*. In other words, considering the difficulty of improving the numerical model in order to account for the stiffness/strength variations introduced by the "non-structural" elements, it is conservatively suggested to amplify the design forces.

Instead, the awareness of the role of infill panels in the structural seismic response is maturing in the scientific community, and also in the professional field, with regard to the possibility of improving the seismic behaviour of existing buildings. **Dolšek & Fajfar (2005, 2008)**, for example, have proposed the extension of the "N2 Method" (which is actually included in many building codes as the reference method for the non linear static analysis) to infilled RC frames, while **Celarec & Dolšek (2012)** have proposed a simply iterative pushover for seismic performance assessment of infilled RC frames. An example in which the contribution of infill walls has been exploited in seismic strengthening and repair interventions is provided by the case of L'Aquila Earthquake (April, 2009): special attention has been paid to the connection of masonry panels to the surrounding frame. Here, local interventions have been applied to non-confined nodes and infill panels, in order to mitigate the risk of brittle mechanisms (shear failure of column-beam joints or beam/column end sections under the shear actions transmitted by the infill panel; shear failure).

The adopted techniques mainly consisted in the implementation of effective connections between the panel (**figure 1(a) - right**) and the surrounding RC elements along the top and lateral edges (**figure 1(a)**), or in the application of plaster reinforced with a regular steel wire mesh (**figure 1(b)**).

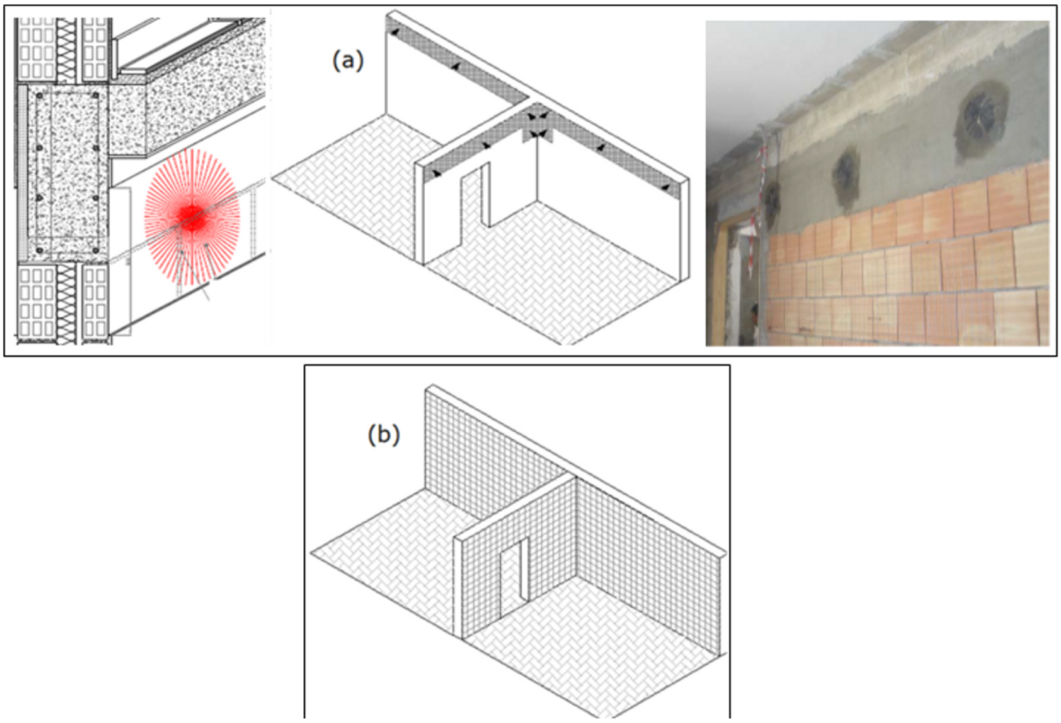

Figure 1. Scheme of the boundary connection of infill panels to the surrounding RC frame: application of a steel wire meshes and tassels (a); reinforced plaster (b).

The above mentioned techniques allow to achieve three objectives: preventing the out-of-plane overturning of the panels; improving the in-plane stiffness and strength; reducing undesired local effects at the end sections of columns. Thanks to the connections, in fact, it is possible to distribute the actions flowing through the infill panel along the top edge, whereas the shear stress concentration that usually affects the ends of columns is significantly reduced, if not entirely cancelled. Under this new stress distribution, however, the collapse mechanisms attained by the whole structure can be modified even significantly **(DPC-Reluis, 2010)**. However, beyond the aforementioned techniques that have become widespread in the post-earthquake period starting in the L'Aquila area, the scientific community is also devoting special attention to the reinforcement of infill panels, confirming the undoubted need to locally reinforce elements from which to draw strength reserves for the purpose of seismic improvement **(Kakaletsis, David & Karayannis, 2011; Antonopoulos & Anagnostopoulos, 2012)** .

In conclusion, it is evident that the contribution of infill panels – especially if they can be defined as "strong" (i.e., when they are characterized by high stiffness/strength values for a sufficient wide displacement range) – can be crucial in the response to medium-high earthquakes. In this context, clearly, the specific numerical approach used to model the infill-frame system under the horizontal actions becomes a fundamental element. For example, macro-models based on the use of the equivalent strut for simulating the presence of infill panels are affected by a strong sensitivity to several parameters: number of struts; width b_w of the equivalent strut; constitutive law of the panel **(Fiore, Porco, Raffaele & Uva, 2012; Uva, Porco, Raffaele & Fiore, 2012)**. In addition to the uncertainties associated with aspects mainly related to the modeling of infill in both design and verification, the structural response of infilled frames is affected not only by the extremely nonlinear nature of the overall behavior, but also by the variability of numerous parameters affecting the constitutive bonds and mechanical properties of the materials. The degree of uncertainty increases significantly when it is necessary to simulate the presence of infills in existing buildings, by virtue of the lack of specific information on the materials used in construction, due to poor maintenance, the heterogeneity and interaction of the constituents (mortar and bricks), and not least, the state of degradation and wear that inevitably progresses with time. There are several literature studies aimed at quantifying the weight of these uncertainties (**Dymiotis, Kappos, & Chryssanthopolous, 2001; Erberik & Elnashai, 2004; Meslem & D'Ayala, 2013; Celarec & Dolsek, 2013),** most of which focused on sensitivity analyses of the variables involved in the response of the infilled system. Celarec, Ricci & Dolsek (2012) in the context of simplified nonlinear procedures for evaluating the seismic performance of infilled frames, demonstrate how the panel shear failure strength, the normal and tangential modulus of elasticity of the masonry, the ratio of the stiffness of the degrading branch to the elastic branch of the equivalent Biella force-displacement law, and the ultimate and yield rotations of the reinforced concrete columns belonging to the frame, are the parameters that most influence the response of the generic infilled frameIn the present paper, with reference to a real case study concerning a school building located in Southern Italy (more precisely, in the City of Cerignola, Province of Foggia, Puglia), the effect of reinforcement interventions on infill panels is investigated. A comparison of the response of different structural configurations analyzed by non linear static procedure is presented. Besides the bare frame, two possible cases have been considered: RC frame with non-reinforced and with reinforced infill panels (respectively, this corresponds to a "strong" and to a"weak" infill behaviour **(Uva et al., 2012*a*; Fiore et al., 2012)**.

The objective is to investigate in detail the behaviour of the framed building in the case in which the infill panels (that are usually considered – and modelled – as non-structural elements having no interaction with the surrounding frame) are solidarized with the frame, becoming a part of the primary structural system, with a particular reference to the modifications induced on the global collapse mechanisms and displacement capacity of the building.

2. Non - linear modelling of infill panels

With regard to the frame-infill system, many models have been proposed in the literature, and can be divided into two classes. The first class includes models based on a macro-modelling approach that will be later discussed. The second is based on the detailed modelling of both RC frames and infill masonry panels by means of proper discretization techniques and non linear constitutive laws of the materials **(Uva & Salerno, 2006)**.

The equivalent diagonal strut method **(Mainstone, 1974; Stafford Smith, 1963)** is based on the observation that, within a masonry panel, the compressive stress substantially follows the diagonal path, and thence adopts one or more equivalent diagonal struts in order to simulate the infill masonry panel. This method belongs to the class of macro-element models, and is indeed the most used, thanks to easy and flexible application possibilities. On the other hand, it should be observed that the advantages related to the simplicity and versatility of the model are counterbalanced by the difficulties rising in the interpretation of the numerical results. Indeed, the most critical problem in the use of macro-models consists in the difficulty of correctly identifying the mechanical properties and the geometrical features of the equivalent diagonal struts, which haven't a direct correspondence with the actual frame-panel system. A significant example is represented by the case of the infill panel with an opening, for which the model of the equivalent strut becomes completely abstract, and can be applied by means of an artificial adjustment of the mechanical parameters, which have no physical correspondence with the reality **(Durrani &Luo, 1994)**.

The fundamental parameters of the methods are represented by the geometric features of the strut (length d_W, thickness t_W and width b_W), the stiffness λ, the hysteretic constitutive law F_W-d which governs the non linear cyclic behaviour of the panel (**figure 2**).

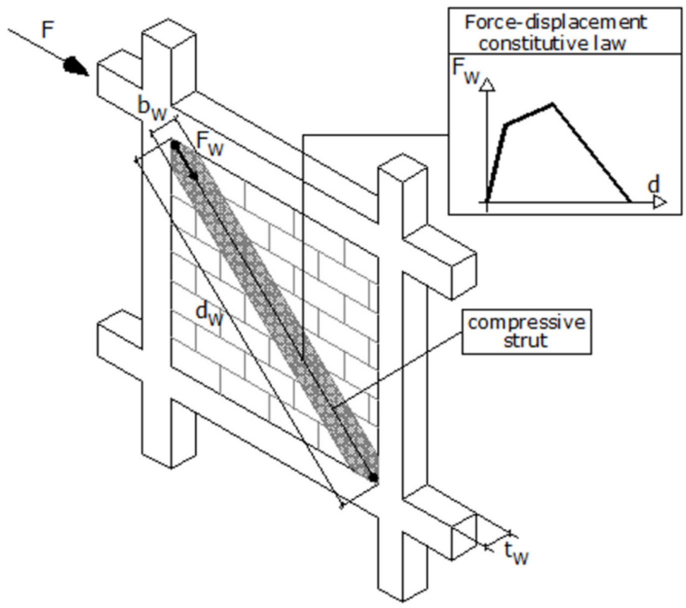

Figure 2. The equivalent diagonal strut model.

In the most recent approaches, the width b_W of the equivalent strut is expressed in terms of the ratio b_W/d_W , whereas in the past it was usual to qualitatively appraise it as a quote of the length d_W **(Mainstone, 1971)**. A well-acknowledged method is the one proposed by **Bertoldi, Decanini & Gavarini (1993)**, supported by several numerical and experimental analyses, in which the width of the equivalent strut was calibrated in order to represent the cracked state of the infill panel under cyclic actions. In the case of panels without openings, the following semi-empiric expression is provided:

$$\frac{b_W}{d_W}=\frac{K_1}{\lambda H}+K_2 \qquad (1)$$

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Where the parameters K_1 and K_2 are expressed as a function of λh (h= distance between the axis line of the top and bottom beam of the frame). The λ factor defines the relative stiffness between the infill panel and the surrounding frame, and can be calculated according to different expressions proposed by different authors **(Mehrabi, 1996)**. The most used expression for λ, anyway, is the one defined by **Stafford Smith & Carter (1969)**, which was actually the basis for all the successive research studies

$$\bullet = \sqrt[4]{\frac{E_{W\theta} t_W \sin 2\theta}{4E_c I_C H \boldsymbol{h}_W hhh\boldsymbol{hh}_{hh}}} \qquad (2)$$

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where E_C is the elastic modulus of the concrete, I_C is the moment of inertia of the columns surrounding the panel, $E_{W\square}$ □is the elastic modulus of the masonry, calculated as a function of the slope angle □ of the diagonal strut to the horizontal.
With regard to the hysteretic law F_W-d which describes the cyclical behaviour of the strut under axial loads, several models can be found in the literature, which are derived from the phenomenological observation of experimental tests in which scale models are dynamically brought to collapse. Among the different proposals, examples can be found in which the law is expressed in terms of axial strain/stress **(Crisafulli, 1997)** and formulations in which, regardless of the geometrical and mechanical characteristics of the infill, a predominant failure mode (which can consist in the crushing at the center or at the corners of the panel) is a-priori defined **(Panagiotakos & Fardis, 1996; Paulay & Priestley, 1997)**.
However, the experimental evidence has pointed out that crushing represents only one of the possible failure modes of the infill panel. Thence, it should be first necessary to evaluate the ultimate load associated to each of the possible failure, and then to calculate the strength of the panel as the minimum of these loads. Some proposals in this direction, based on semi-empirical approaches, identify a set of different failure modes of the infill panel subjected to horizontal in-plane loads: 3 failure modes, 4 failure modes and 5 failure modes **(Saneinejad & Hobbs, 1995; Mehrabi,1994; Liauw & Kwan, 1985)**. However, the empirical formulations characterizing the aforementioned models are dependent on the variability of a considerable number of parameters, making them particularly challenging from an application point of view. The right compromise between experimental feedback, clarity in defining failure modes and number of variables involved in the evaluation of failure stresses is represented by the model proposed by **Bertoldi et al. (1993)**.
The model proposed by **Bertodi et al. (1993)**, which is adopted in the present research work, considers four different types of failure mechanisms and to each of them associates an ultimate stress value σ_w, which is constant along the diagonal strut:

$$F_W = [\![(\sigma]\!]_W)_{\min t_W b_W \cos}\theta \qquad \textbf{(4)}$$

- Crushing at the center of the panel:

$$\sigma_{W1} = \frac{1,16 f_{Wv} \tan\theta}{K_1 + K_2 \lambda H} \qquad \textbf{(5)}$$

- Crushing at the corners of the panel:

$$\sigma_{W2} = \frac{1,12 f_{Wv} \sin\theta \cos\theta}{K_1(\lambda H)^{-0,12} + K_2(\lambda H)^0,88} \qquad \textbf{(6)}$$

- Sliding on the mortar bed joints:

$$\sigma_{W3} = \frac{(1,2\sin\theta + 0,45\cos\theta) f_{Wu} + 0,3\sigma_v}{\frac{K_1}{\lambda H} + K_2} \qquad \textbf{(7)}$$

- Diagonal tension:

$$\sigma_{W4} = \frac{0,6 f_{Ws} + 0,3\sigma_v}{\frac{K_1}{\lambda H} + K_2} \qquad (8)$$

in which f_{Wv} , f_{Wu} and f_{Ws} are, respectively, the compressive strength of the masonry in the vertical direction; the shear strength for sliding of mortar joints in the absence of compression (cohesion) and the shear resistance to diagonal cracking; σ_v is the value of the axial stress for gravity loads (it is zero in the case of panels that have no load-bearing function). According to this model, the Force-displacement law (**figure 3**) is defined once two parameters are known: K_m and F_m which are, respectively, the stiffness and the peak strength of the equivalent strut. Residual force F_r in order to guarantee the numerical stability, it can be assumed equal to *35%* of F_m. In the proposed case studies, the residual force is assumed to be 0.

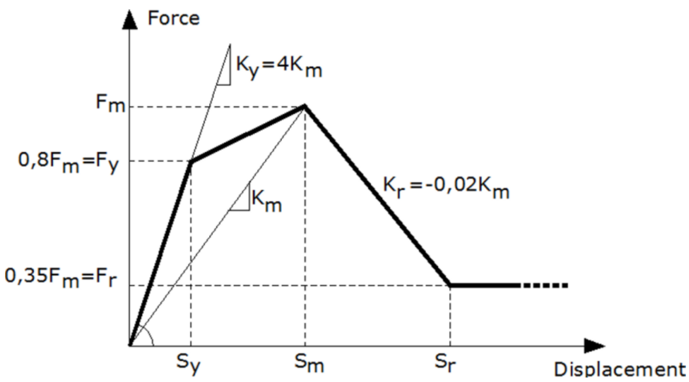

Figura 3. The Force-Displacement envelope curves of the equivalent strut proposed by Bertoldi et al. (1993)

By looking at the law, a strong sensitivity of the proposed model to the materials' mechanical properties appears. In particular, as long as the mechanical parameters of the panels increase, the required ductility μ decreases. However, even for low values of μ, infilled frames may exhibit a significant inelastic behaviour, with a non-uniform distribution of the ductility demand, which is clustered at specific storeys. This phenomenon also modifies the maximum available displacements at the lower floors, with the possible development of soft storey mechanisms.

3. The case study

3.1 Vulnerability analysis of school buildings in the Province of Foggia

The case study proposed in this paper is one of the 20 school buildings (80% of which has a RC framed structure) included in a specific regional research program aimed at the seismic safety assessment of a sample of school buildings in the Province of Foggia. This was part of a wider research Project was funded by *Regione Puglia* (Fund CIPE 20/2004) and managed by the *Autorità di Bacino della Puglia* (Basin Authority of Puglia) in cooperation with a number of Public Institutions (Department *Dicatech* of the University "*Politecnico di Bari*", Municipality of Foggia, Administration of the Province of Foggia) targeted at the multi-hazard risk assessment of current building stock, critical bridge infrastructures.

The vulnerability assessment of the school buildings has provided information about the actual safety level, both with regard to the vertical and the seismic loads, and moreover, has allowed to collect a precious database which includes not only information about the geometry, the history and the materials of the buildings, but also detailed data about the structural performance parametrs (such as the displacement capacity, strength, …). The pie charts in **figure 4** show the percentage distribution of some constructive and typological features of the analyzed sample (age of construction, number of storey, irregularity), from which a representative "*reference building type*" can be deduced: age of construction before 1980; presence of strong in-plane irregularity; low rise building (number of storeys<4). An indicative value of the Seismic Vulnerability Coefficient (SVC), widely utilized in several application and research issues (**Raffaele, Porco, Fiore & Uva, 2013**), defined as the ratio between the seismic action corresponding to the attainment of the limit structural capacity and the seismic demand, both evaluated in correspondence of the Limit State of Life Safety (LS). The chart, in which it can be seen that the SVC values are mostly below the unit, highlights the strong seismic vulnerability of the school buildings located in the area.

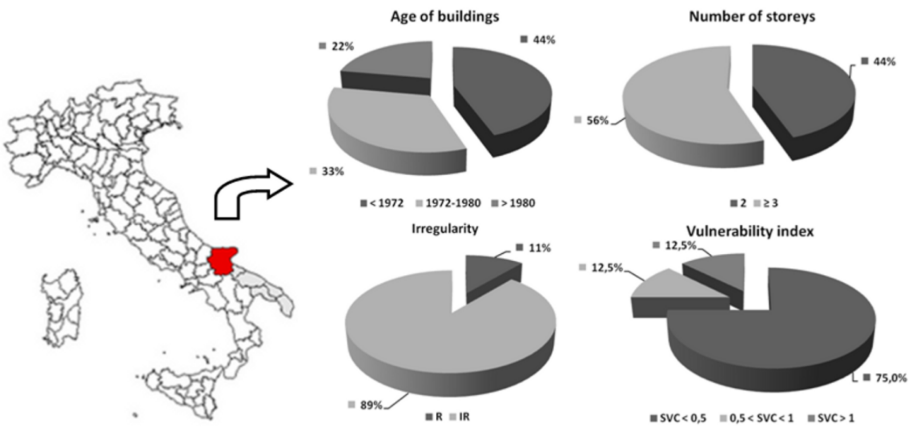

Figure 4. Distribution of some significant parameters for the analyzed sample within the area under research project (Province of Foggia in Southern Italy).

The seismicity of the investigated area, expressed in terms of "maximum horizontal acceleration at the site - a_g", is comprised between *0.173g* and *0.253g*. On the basis of the Italian scale, thence, the seismic hazard of the region, can be classified as low-medium.

3.2 Description of the case study and investigation about the quality of in-place materials

The school building (figure 5) chosen as case study is located in the city of Cerignola (Province of Foggia, Puglia, Italy). It is a structure having two storeys (ground floor and first floor) containing the classrooms and the related facilities and flat, non accessible roof. According to the available documents, the building was constructed in the years '68-'70. According to the technical regulations of that period (being the considered area classified at that time as not subject to seismic risk) the structural design was simply based on the verification of the safety level with regards to vertical loads, whereas no specific design and calculation criterion for the seismic actions was considered. The building has an extension of about *356* m^2 in plan, and a height of about *8.5 m*. The structural system is provided by a RC frame, with mixed slabs (cast-in place concrete and hollow tile bricks) having a thickness of *20 cm* at the intermediate levels, and *43 cm* at the roof.

Figure 5. A view of the case study

There are no significant irregularities in elevation (protrusion, recesses, stiffness/mass variations) which may affect the vertical regularity. The beams of the first level have a rectangular section (*30 cm x 50 cm*; *30 cm x 60 cm*); those of the second level have sections of dimensions *30 cm x 50 cm* and *40 cm x 50 cm* (frame #X2). In the transverse direction, there are connecting slab beams *30 cm* wide. There are three types of column sections: *30 cm x 50 cm*, *40 cm x 40 cm*; *45 cm x 45 cm*, that remain constant for all their height. On the basis of retrieved data and in-situ inspections, the reinforcements' arrangement for the structural elements has been derived. In **figure 6**, the longitudinal reinforcement for the main structural elements is shown. The transversal reinforcement is the same for all the elements (ϕ *6* stirrups, uniformly spaced every *20 cm*).

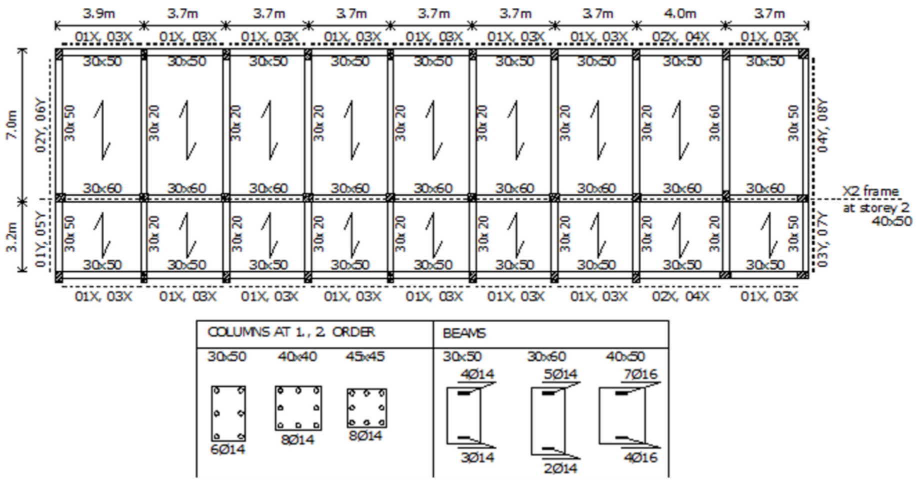

Figure 6. The case study: structural plan of the ground floor and scheme of the longitudinal reinforcement in the main structural elements.

After completing the general geometrical survey and the direct inspections aimed at investigating the geometrical features of the hidden structural elements, a detailed experimental program was planned, including on site non-destructive testing and laboratory tests on the specimens pulled put from the structural elements, in order to evaluate the mechanical properties of materials and achieve a "*Knowledge Level 2*" **(D.M. 14/01/2008; CEN, 2005).**

Destructive (drilled cores) and non-destructive tests (rebound hammer and ultrasonic pulse velocity test) have been performed on the most significant structural elements (with regard to the stress level under both vertical and seismic actions). The data acquired by non destructive methods have been used in order to support and integrate the estimate of in-place concrete strength provided by the compressive tests on drilled cores. In the literature, several methods and procedures have been proposed for the correlation of significant data (for example, the compaction degree) in order to obtain reliable estimates of the compressive strength and assess the possible presence of different homogeneous classes of concrete **(Uva, Porco, Fiore & Mezzina, 2013)**.

In the present case, the numerical processing has involved the use of rebound hammer index, ultrasonic pulse velocity and compressive core strength. A good homogeneity of the material has been found, allowing to define a single homogeneous concrete class, characterized by a compressive strength equal to *22 MPa*. Tensile tests over the steel bars extracted have provided a reference strength value of *301 MPa*.

With regard to the infill walls, the endoscopic investigation (**figure 7**) in has revealed that the infill consists of a cavity wall (the external layer is made of solid bricks *12 cm* thick, the internal one is made of hollow bricks *8 cm* thick).

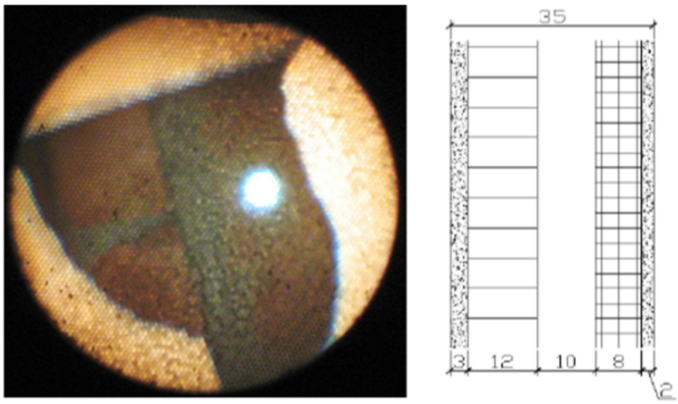

Figure 7. Pictures about endoscopic investigation

It is actually very similar to the infill panels used in the research work Bertoldi et al. (1993) and, in the absence of specific on site tests, the mechanical parameters provided in this reference have been adopted, which are collected in **Table 1**.

Table 1. Mechanical parameters adopted for the infill wall

	f_{Wv} *[MPa]*	f_{Wu} *[MPa]*	f_{Ws} *[MPa]*	E_{Wh} *[MPa]*	E_{Wv} *[MPa]*	**G** *[MPa]*	**W** *[MPa]*
masonry infill	1.5	0.25	0.31	991	1873	1089	6.87
strengthened masonry infills	1.95	0.32	0.40	1288	2435	1415	8.93

3.3 *Numerical modelling*

The numerical modelling was carried out by using a finite element approach, implementing proper spatial models of the building's structure within the solver "SAP2000" **(Computer & Structures, 2010)**. In particular, 9 three-dimensional models of the building have been initially considered: *one* for the analysis of the bare frame; *four* for the analysis of the structure in the actual configuration (unreinforced infill panels); four for the analysis of the structure with the infill panels reinforced according to the techniques described in the introduction and shown in **figure 1**

The case of the infilled frame has involved the definition of four different numerical models depending on the direction of the analysis (directions +X and +Y; -X and -Y). The presence of the infill panels has been considered only in the frames parallel to the direction of the analysis, in order to overcome possible problems of convergence in the numerical solver. Actually, the results found in the + X and + Y directions are very similar to those of the dual directions -X and-Y, and therefore are not presented in the paper. Henceforward, all the discussion will concern three structural models (regardless of the direction of analysis), that will be indicated, respectively, by the letters "B" (bare frame), "IS" (initial infilled structure, i.e. model having behaviour nearer than those of the current structure); "R-IS " (reinforced infilled structure). In the "IS" and "R-IS " cases, the infill panels have been modeled by means of equivalent diagonal struts arranged along one of the two diagonal of the panel, in order to react to compression according to the direction of the pushover analysis (as an example, in **figure 8** , the three-dimensional model of the structure used for the analysis in the + X direction is shown).

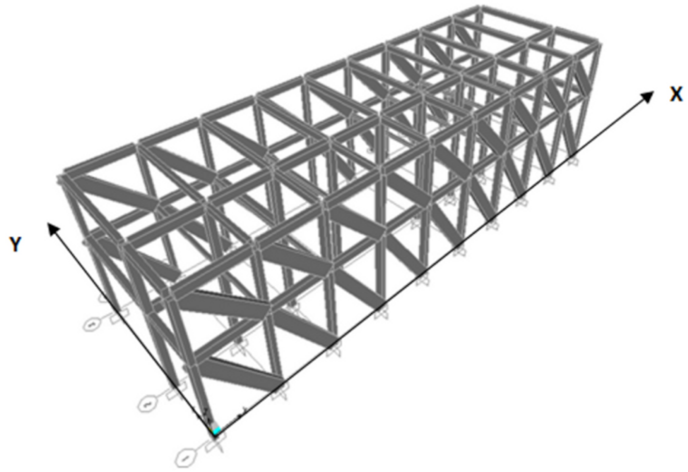

Figure 8. The 3D frame model with the single equivalent struts in +X

The nonlinear behavior of the frame elements adopted in the modeling, was defined according to diffuse plasticity (fiber) modeling. In more detail, the inelasticities were concentrated in the end sections of the elements through the adoption of fiber elements. The stress-strain bond characterizing the post-elastic behavior of the section is given by integrating the stress-strain bonds of all the fibers in which the cross section is discretized. According to this approach, the section turns out to be divided into fibers related to unconfined concrete (in areas outside the transverse reinforcement), confined concrete (areas inside the transverse reinforcement), and fibers at the reinforcement. The number of fibers into which the sections have been discretized is determined by an algorithm internal to the solver in order to avoid stability and convergence problems in finding solutions.

With regard to the constitutive laws for the materials, and on the basis of the results of in-situ tests, the following choices have been made: stress-strain law for confined concrete proposed by **Mander, Priestley & Park (1984)**; elastic – hardening diagram for the steel.

Preliminarily, the response under vertical loads has been evaluated. The different structural elements were classified in two groups according to the structural response: "ductile" elements (flexural behaviour is predominant) and "brittle" (shear behaviour is predominant). The analysis has revealed the presence of several “brittle” columns, which are particularly vulnerable to shear stress. In addition, it should be mentioned that the column-beam joints are particularly lacking with regard to the constructive details, and don’t comply with the requirements of the current building code. These circumstances don’t allow a sufficiently ductile global response of the structure. If the application of a proper reinforcement to the unconfined joints is adopted (this kind of intervention is actually very frequent in the repair of existing buildings **(Karayannis, Chalioris & Sirkelis, 2008)**, besides a configuration of the joints in line with current standards, an improvement of the brittle structural elements is also obtained (the jacketing, in fact, involves also the end portions of the primary elements). It has been here assumed that the retrofitting of the joints will be applied by default on the whole frame, and thence, no shear hinge has been included in the numerical model (after joint restoration, the behaviour of the columns classified as “brittle” is again a flexural one). The characteristic parameters of the strut model (width of the strut - b_W; hysteretic constitutive law - F_W-d) have been assumed according to the model of **Bertoldi et al. (1993)**.

In order to appraise the alteration of the structural behaviour in the presence of the reinforced infill panels, the increased mechanical parameters have been assumed according to the indications provided by the Italian Building Code **[4]**, depending on the kind of intervention that is applied. In this case, the reinforcement technique is that of the reinforced plaster, and the code suggests the application of a correction factor of 1.3 to the original parameters of the masonry.

It should be mentioned that the interventions based on the use of tassels or of a steel welded mesh involve an alteration of the collapse mechanism of the panel and, consequently, a substantial modification of the overall failure mode of the frame-infill system. In fact, the panel is solidarized to structural elements along the contour perimeter by means of reinforced plaster and connector flakes that actually reinforce the panel globally. The techniques prevent from the risk of out-of-plane tilting and improve in-plane response by averting possible local effects. The behaviour of the infill wall becomes similar to that of a reinforced panel, which is usually not affected by local collapse mechanisms (especially at the corners). In this sense, the tests performed by **Calvi & Bolognini (2001)** on unreinforced and reinforced panels (the reinforcement consisted in the insertion of vertical steel bars in the vertical holes of the bricks) are particularly relevant. It was shown that the failure mechanism, in the two cases, can be very different according to the drift value. For reinforced panels, the failure was characterized by the crushing, with a concentration of the damage in the central part of the panel, whereas the contact surfaces with the frame presented a very limited cracking. These results confirm that the approach proposed by **Bertoldi et al. (1993)** doesn't fully reproduce the actual behaviour in the case of a reinforced panel. The application of the numerical formulation of Bertoldi et al. to the case study indicates the crushing at the corners as the critical failure mode for almost all panels (**figure 9**), in contrast with the experimental results previously mentioned.

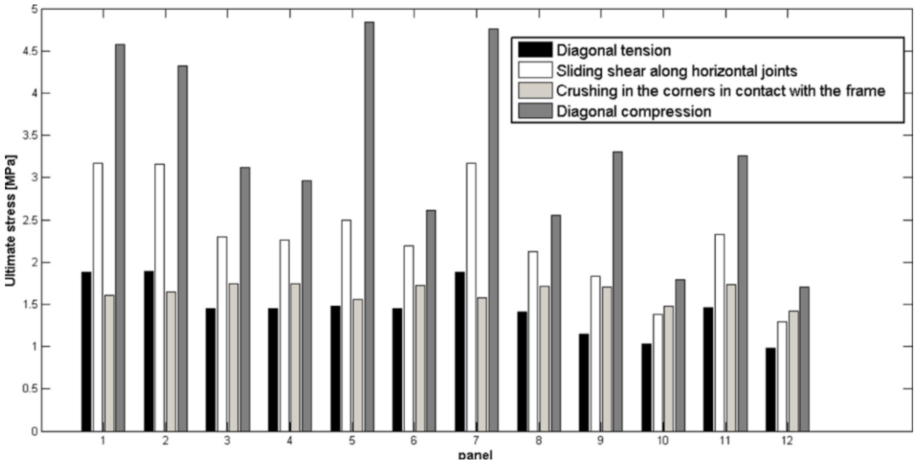

Figure 9. Values of the ultimate stress for the case study according to the formulation of Bertoldi et al.

In order to obtain a more realistic representation of the structural behaviour, a calibration of the hysteretic law F_W-d has been performed for each of the strut, on the basis of the maximum force (*see* **eq. 4**) corresponding to the crushing at the middle of panel. In **figure 10** respectively for the reinforced structure (R-IS model) and the unreinforced structure (IS model), the force-displacement envelope obtained for the 4 struts for the pushover analysis in the X direction are shown.

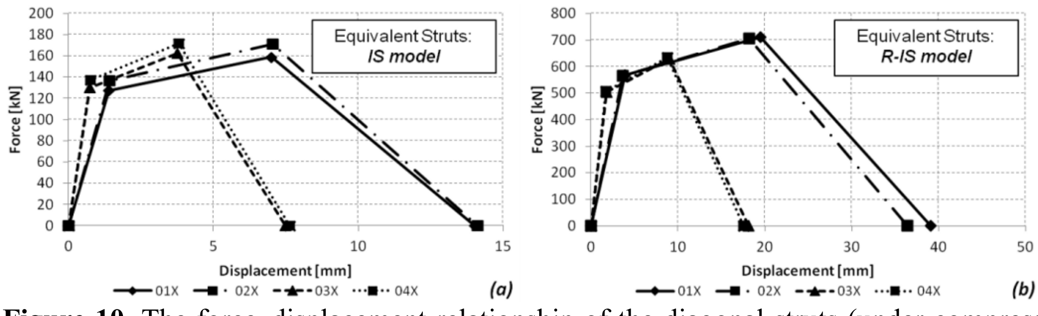

Figure 10. The force–displacement relationship of the diagonal struts (under compression) for IS and R-IS models in the X analysis direction.

According to this assumption, the behaviour of the strut correspond to a "strong " infill, for which the beginning of the plastic phase starts for very large strength values (approximately 4 times higher than in the case of unreinforced panels). Overall, the global behaviour of the structure is comparable to that of a mixed masonry-RC structure, in which the masonry walls are true primary elements side by side with RC columns and beams.

4. Numerical results

Usually, an existing structure which was originally designed under vertical loads only has number structural deficiencies, mainly related to the absence of specific anti-seismic details. In these buildings, often, the structural collapse is driven by the strong vulnerability to shear stress. It is recommended to eliminate the main vulnerability factors by applying proper retrofitting interventions on the infill walls – like those mentioned in the introduction – and by restoring column-beam joints.

With regard to the interventions on the infill panels, different solutions can be adopted, depending on the objective in terms of global seismic response and on the available economic resources. The curves In **figure 11** show the non linear response that can be obtained by retrofitting the joints and adopting two different options for the reinforcement of the infill panels: solidarization of infill panels to the surrounding frame (curve I), replacement of the existing panels with new ones fully disconnected from the frame (curve II).

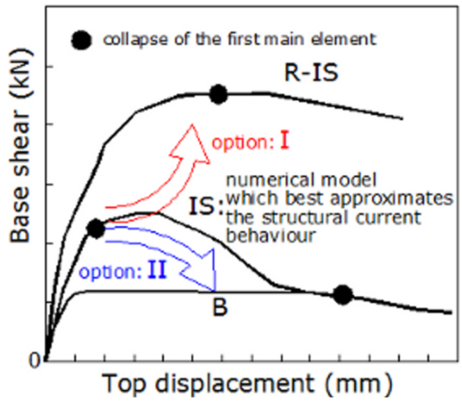

Figure 11. The global structural response obtained in correspondence of the different reinforcement options for the infill panels.

The option for a frame-wall behaviour (curve I) involves a strong increase of the resistance, and this should require a review of the foundation system, in order to verify the altered distribution of the stresses. The overall structural response, of course, is very different from that of the bare frame. If the choice is, instead, of disconnecting infill panels (curve II), the behaviour of the infill walls is completely independent from that of the frame. This solution is very expensive, and often requires the retrofitting of the frame columns, because under strong earthquakes – which require high displacement capacity - it is not possible to rely on the additional contribution of infill panels. The ductility demand is hand over the bare frame only, which is often unable to provide the required capacity. This means, in other words, that it is not sufficient to strengthen only the joints, but it can be even necessary – in this case – to increment the stiffness and strength of the columns.

4.1 Non linear static analyses

The performance capacity has been calculated by non linear pushover analysis at the Limit States of Damage (DL), Life Safety (LS) and Near Collapse (NC) for the three models (B, IS, R-IS). In diffuse plasticity nonlinear static analyses, the above limit states correspond to the attainment, for the first main structural element (particularly the columns ,respectively, of the following boundary conditions: yielding of the longitudinal tensioned reinforcement, maximum compressive value of the confined concrete (relative to the generic longitudinal fiber inside the section bounded by the transverse reinforcement, material failure.

As the lateral distributions of incremental loads, the first fundamental mode was assumed as a shape vector (no significant difference between the cases of bare or infilled structure was found for the shape). It should be noted that the nonlinear response against load distributions other than the one considered, which is in fact mandatory for the purpose of safety verification, was not analyzed because the participating mass is well above the 75% indicated as the limit value by the Italian standard for the applicability of nonlinear static analysis (*paragraph: §7.3.4*). In fact, by the inherent regularity of the construction, the nonlinear response induced by alternative load distributions to the one considered for the analyses (uniform load distributions, proportional to masses, etc.), do not deviate significantly from the results obtained. For these reasons, the proposed results are for the load distribution proportional to the fundamental mode of vibration only.

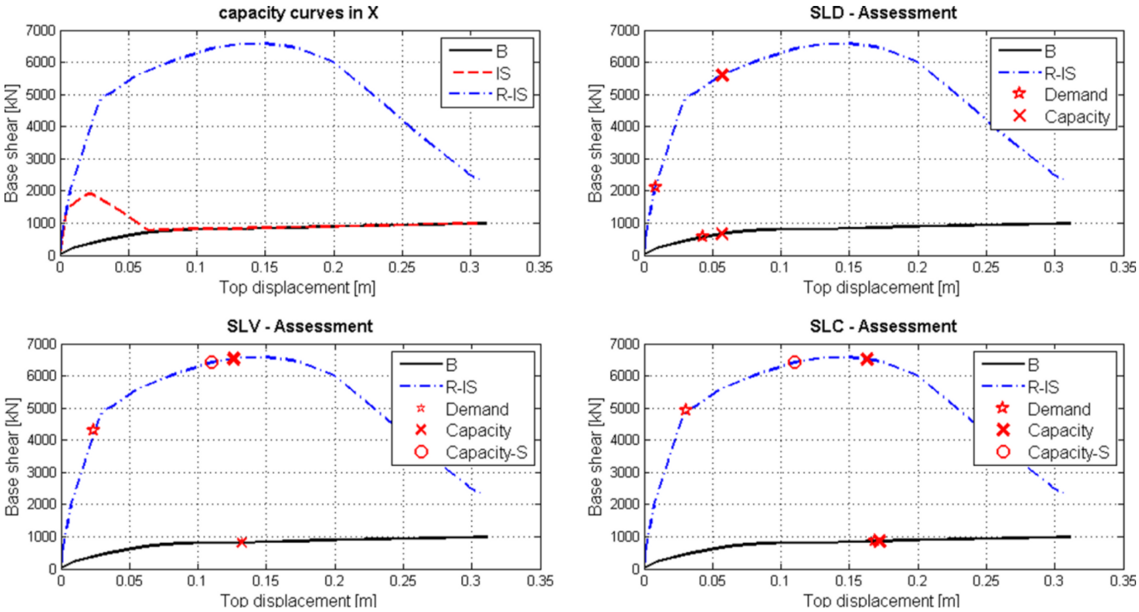

Figure 12. Pushover analyses in the X direction: comparison between the capacity curves for the three models B, IS, R-IS and graphic performance assessment at the different limit states.

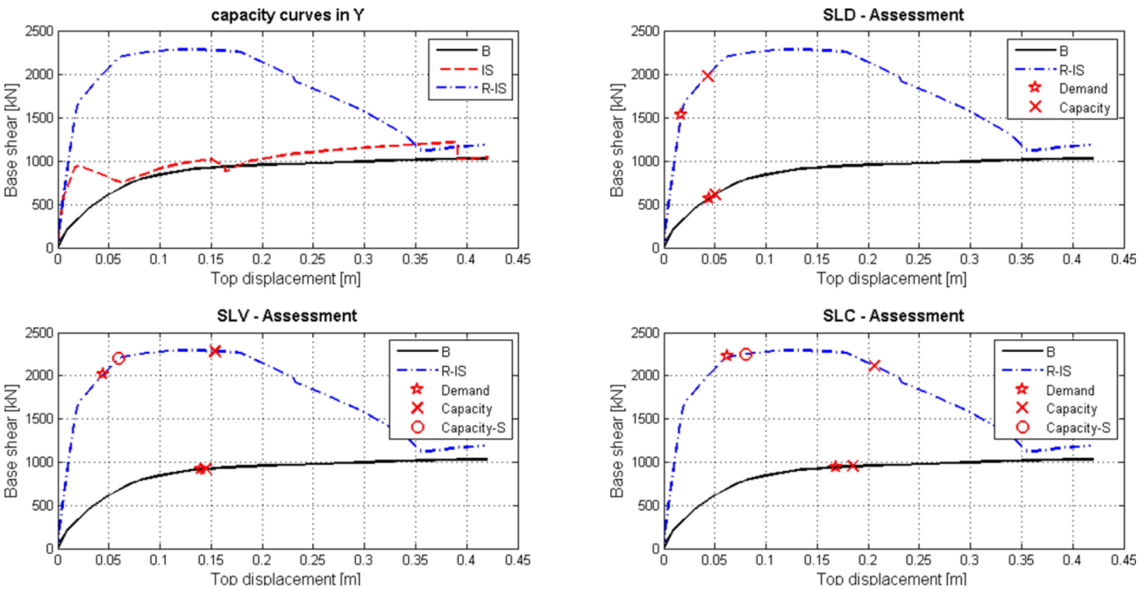

Figure 13. Pushover analyses in the Y direction: comparison between the capacity curves for the three models B, IS, R-IS and graphic performance assessment at the different limit states.

As previously mentioned, the presence of the infill increases of stiffness and strength of the structure. This is confirmed by the curves shown in **figures 10** and **11**, which show the results of the pushover analyses in the X and Y direction, respectively. The ratio between the maximum shear force, V_b^{IS}/V_b^{B} and $V_b^{R\text{-}IS}/V_b^{B}$, is respectively equal to *1.65* and *5.72* in the X direction, and

to *1.14* and *2.14* in the Y direction. According to these results, the strength increases determined by the retrofitting of the panels is, respectively, of *446%* and *287%* in the direction X and Y. In the Y direction, the strength increment is smaller, and this can be justified by the presence of a number of infill panels (only 8) much smaller than that in the X direction (36 panels). In the Y curves, just because of this circumstance, the conjunction of the R-IS capacity curve with the capacity curve of the base frame is observed (as theoretically predicted by **Dolsek & Fajfar (2005, 2008)** even if for very large displacement values, which are not actually significant for the real structure.
The situation is different in the X direction, since the presence of a greater number of infill panels induce an early stop of the analysis for numerical convergence issues (nevertheless, the significance of the results is not affected, since very great displacement values – more than 40 cm - have been already reached).
In the figures, the seismic demand and the actual structural capacity at the different limit states are also indicated. It should be pointed that it has been assumed that the limit structural capacity is attained in correspondence with the collapse of the first primary vertical element. The determination of the seismic demand has been performed according to the N2 Method **(Fajfar & Gaspersic, 1996; Dolšek and Fajfar 2004)** (original version implemented in EC8) both for the infilled frame and the bare structure.

Table 2. Seismic demand and structural capacity at the different limit states for the pushover analyses in the X and Y direction for the models B and R-IS; characteristic parameters of the equivalent SDoF used in the pushover analysis.

Pushover in	DL		LS		NC	
X direction	B	R-IS	B	R-IS	B	R-IS
D *[m]*	0.043	0.009	0.132	0.024	0.169	0.031
C *[m]*	0.057	0.057	0.132	0.126	0.172	0.163
Cs *[m]*	-	0.057	-	0.110	-	0.110
SC	1.32	6.33	1.00	4.58	1.01	3.54
Pushover in	DL		LS		NC	
Y direction	B	R-IS	B	R-IS	B	R-IS
D *[m]*	0.045	0.017	0.140	0.045	0.179	0.063
C *[m]*	0.050	0.043	0.145	0.154	0.185	0.206
Cs *[m]*	-	0.043	-	0.061	-	0.061
SC	1.11	2.52	1.00	1.35	1.03	0.97
Characteristic parameters of the equivalent SDoF						
	Pushover in X direction				**Pushover in Y direction**	
	B	R-IS			B	R-IS
m^* *[kNs²/m]*	561	595		m^* *[kNs²/m]*	512	533
k^* *[kN/m]*	12435	177024		k^* *[kN/m]*	12183	90203
T^* *[s]*	1.335	0.364		T^* *[s]*	1.288	0.483
F_y^* *[kN]*	896	5669		F_y^* *[kN]*	885	1949
d_y^* *[m]*	0.042	0.032		d_y^* *[m]*	0.043	0.021
$d_u^*{}_{DL}$ *[m]*	0.045	0.06		$d_u^*{}_{DL}$ *[m]*	0.048	0.05
$d_u^*{}_{LS}$ *[m]*	0.13	0.14		$d_u^*{}_{LS}$ *[m]*	0.16	0.17
$d_u^*{}_{NC}$ *[m]*	0.17	0.18		$d_u^*{}_{NC}$ *[m]*	0.20	0.23

As already pointed out, the overall structural response can be assimilated to that of a frame-wall system, in which the walls assume a primary behaviour under horizontal actions. Two capacity points have been thence indicated on the curves relative to the R-IS model: collapse of the first primary element (point "X"); collapse of the first infill panel (point "O"). It should be noted in the Y direction, for the limit states of LS and NC, the collapse of the first infill panel occurs long before

Than the collapse of the first column, and this reduces the structural capacity, respectively, of *61.4%* and *71.3%*. The results graphically shown in **figures 10** and **11** are also summarized in **Table 2**, where the explicit quantification of the safety level, expressed as the ratio between the capacity C and the seismic demand D, is also indicated, together with the parameters governing the transformation into the equivalent SDoF system. In particular C_S is the structural capacity corresponding to collapse of the first infill panel (intended as a primary element for R-IS model).

The reinforcement intervention increases the structural ductility (intended as C/D ratio or difference of safety coefficients (SC) between, respectively, R-IS and B model) with respect to the bare frame: in particular, the increments at the different limit states are equal to *5.01*, *3.58* and *2.53* in the X direction, whereas in the Y direction are equal to *2.41*and *0,35* . At the NC limit state, instead, by virtue of the early collapse of the first infill panel – which precedes the primary elements, a slight reduction of the structural ductility is encountered (- *0.06*).

In conclusion, in Figures 13, we show the trends of storey drifts for both sides of the analyses (+X and +Y) and limit states considered.

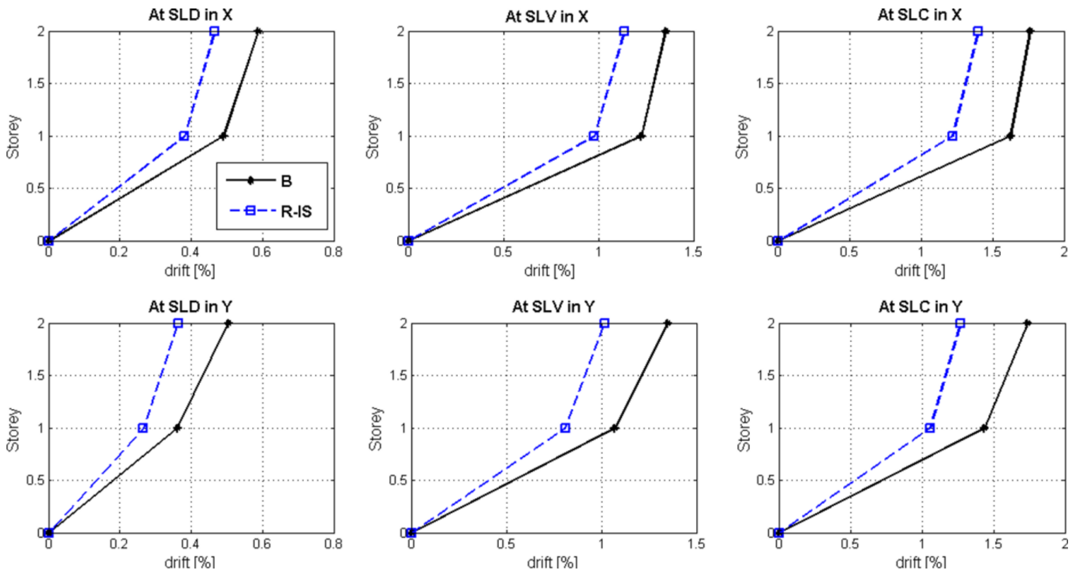

Figure 13. Interstorey drifts

The reduction of the global ductility with respect to the bare frame is greater in the Y direction: the reinforced structure, at the DL limit state exhibits a 28% reduction of the drift at the second level (this is the maximum of all the values). In the Y direction, besides, the interstorey drift tends to increase with the height, in contrast to what was found in the X direction, in which the first floor, due to the presence of a large number of infill panels, is less sensitive to lateral displacements than the bare frame. The maximum reductions, in this case, are found at the first level: 20%, 16% and 25% respectively, at the DL, LS and NC respectively.

Final Remarks

This paper regards about the effects induced by the retrofitting interventions usually performed on existing RC framed buildings which were originally designed under vertical loads only:

(1) Reinforcement of unconfined joints;
(2) Reinforcement of the infill panels by the solidarization to the surrounding frame or – alternatively – replacement with a totally disconnected panel

The first intervention is fundamental in all the structures designed without specific seismic detailing, and is aimed at incrementing the structural ductility. The second has the double

objective of preventing out-of-plane overturning of the panels and exploiting the infill contribution to the lateral strength.
A case study concerning a RC school building dated back to the '70's, and located in a low-medium seismic zone is presented. The results obtained can be summarized as follows:

(1) Both above mentioned types of interventions involve a substantial modification of the structural behaviour, and the actual choice depends on the objective of the intervention and on economic considerations. After the analyses performed on the case study, some general considerations can be made:
   a. The solidarization of the infill panel to the RC frame allows to stiffen the structure and to obtain optimum performance capacity for earthquakes of low intensity. The advantages are similar to those provided by frame-wall systems (relevant strength and stiffness increase for small displacement values of the control point). In contrast, the high base shear can drive an excessive stress concentration on the foundation structures and on horizontal structures, which should be therefore properly verified and possibly reinforced.
   b. The disconnection of the panels from the frame requires that the bare frame is able to resist by itself to high intensity earthquakes, since the infill contribution, in this case, is not effective. This means that most of the columns of the frame shall be properly retrofitted (e.g., by incrementing the resisting section). Moreover, it should be mentioned that the cost of this solution is often very high.

(2) In the case of regular buildings, the focus over the infill elements can effectively reduce the seismic vulnerability by reducing – at the same time – the invasiveness and cost of the retrofitting, thanks to the implementation of quite easy and quick solutions. Anyway, the actual intervention strategy must be critically evaluated, after a detailed investigation and analysis of the structure.

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