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EFFECTS OF "REINFORCED" INFILLED FRAMES ON EXISTING RC BUILDINGS

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Abstract. *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 and on is provided. Nowadays, it is well-established that infill panels contribute in increasing the overall in-plane resistance to horizontal actions within a variable range of the displacement intervals before they start to degrade. Anyway, in most cases there is also an effect of significant alteration of the overall seismic behaviour and collapse mode, which should be properly taken into account.*

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 both on the reinforced configuration and on the original structure and critically discussed.

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 standards and legislations have revealed to have an inadequate seismic performance. In addition, for existing structures, when there aren't specific structural monitoring systems, it becomes very difficult to evaluate the performance of the materials in time [1,2].

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. 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. 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 [3,4].

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 [5,6], in which it is suggested that infill panels, under in-plane horizontal actions, should be considered as completely disconnected by the surrounding frame.

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. Dolsek and Fajfar [7,8], 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. 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 nodes 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 and the surrounding RC elements along the top and lateral edges (Figure 1a), or in the application of plaster reinforced with a regular steel wire mesh (Figure 1b).

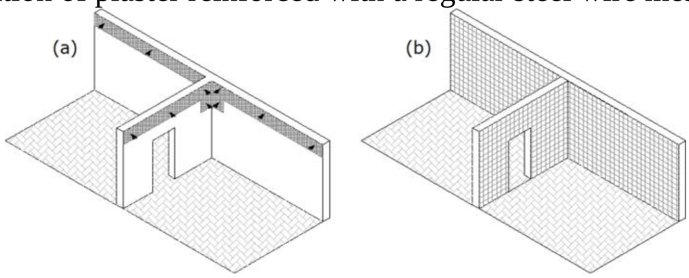

Figure 1: Scheme of the perimetric connection of infill panels to the surrounding RC frame: application of a steel wire mesh and tassels (a); reinforced plaster (b).

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 columns' end sections. 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 columns' ends is significantly reduced, if not canceled at all. Under this new stress distribution, however, the collapse mechanisms attained by the whole structure can be modified even significantly.

In conclusion, it is evident that the contribution of infill panels – especially if they can be defined as "strong" (i.e., 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 a number of parameters: number of struts; width b_w of the equivalent strut; constitutive law of the panel [9].

In 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. An extensive 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 [3,4].

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 MODELING OF INFILL PANELS: A SHORT STATE OF THE ART

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 and infill masonry panels by means of proper discretization techniques and non linear constitutive law of the materials [10,11,12].

The equivalent diagonal strut method [13,14] 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 (for example, the case of the panel with an opening). A promising approach could be that of exploiting the results coming from a detailed but effective computational modelling [15,16,17,18] for interpreting and calibrating strut-like macro-models and the parameters.

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).

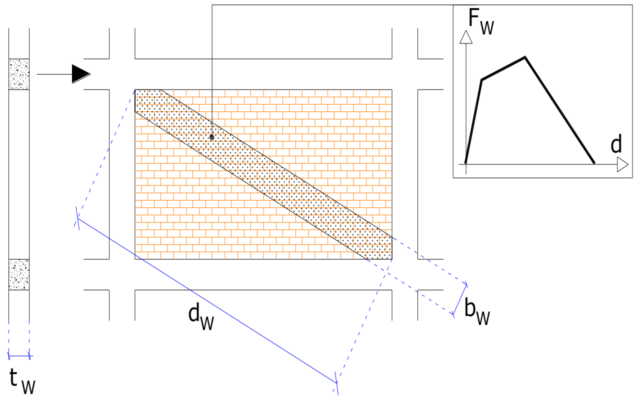

Figure 2: The equivalent diagonal strut model.

In the most recent approaches, the width b_W of the equivalent strut is expressed in terms of the ratio b_W/d_W. A well-acknowledged method is the one proposed by Bertoldi et al. (1993)[19].

$$\frac{b_W}{d_W} = \frac{K_1}{\lambda H} + K_2 \tag{1}$$

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 [20]. The most used expression for λ, anyway, is the one defined by Stafford Smith & Carter [14], which was actually the basis for all the successive research studies.

$$\lambda = \sqrt[4]{\frac{E_{W\theta} t_W \sin 2\theta}{4 E_c I_C H_W}} \tag{2}$$

where E_C is the elastic modulus of the concrete, I_C is the moment of inertia of the columns surrounding the panel, $E_{W\theta}$ is the elastic modulus of the masonry, calculated as a function of the slope angle θ 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 [21] 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 [22].

The model proposed by Bertodi et al. (1993)[19], 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:

$$F_W = (\sigma_W)_{min} t_W b_W \cos\theta \tag{4}$$

- Crushing at the center of the panel:

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

- Crushing at the corners f the panel:

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

- Sliding on the mortar bed joints:

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

- Diagonal tension:

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

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.

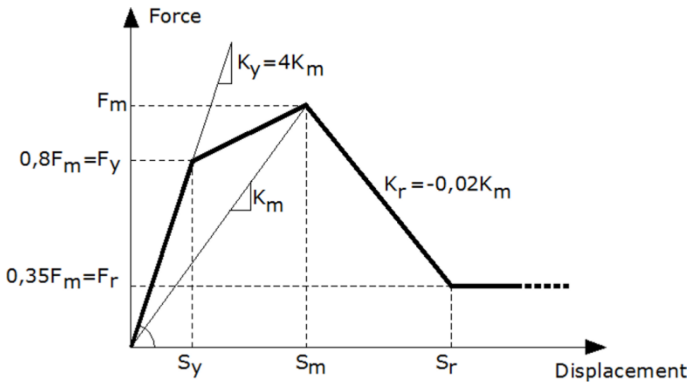

Figure 3: The Force-displacement law proposed by Bertoldi et al. [19]

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 [23].

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, the materials, …of the buildings, but also detailed data about the structural performance parametrs (such as the displacement capacity, strength, …). The pie chart in Figure 4 shows 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 (CVS),

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 CVS values are mostly below the unit, highlights the strong seismic vulnerability of the school buildings located in the area.

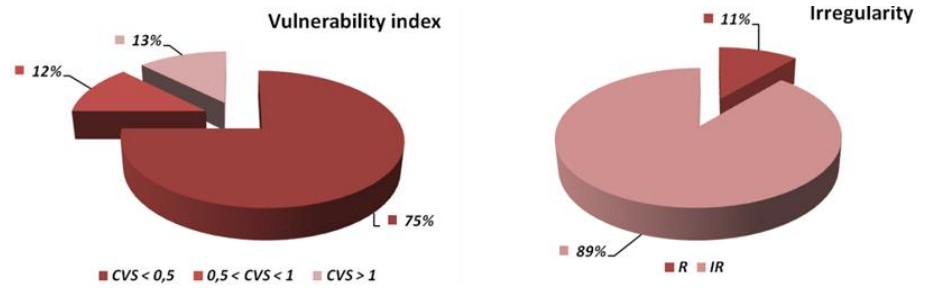

Figure 4: Distribution of some significant parameters for the analyzed sample

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 chosen as case study is located in the city of Cerignola (Province of Foggia, Puglia, Italy). It is apart from other constructions, with 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, and being the considered area classified at that time as not seismic prone, 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. 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*; *30cm x 60 cm*); those of the second level have sections of dimensions *30 cm x 50 cm* and *40cm 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 5, the longitudinal reinforcement for the main structural elements is shown. The transversal reinforcement is the same for all the elements ($\phi 6$ stirrups, uniformly spaced every *20 cm*).

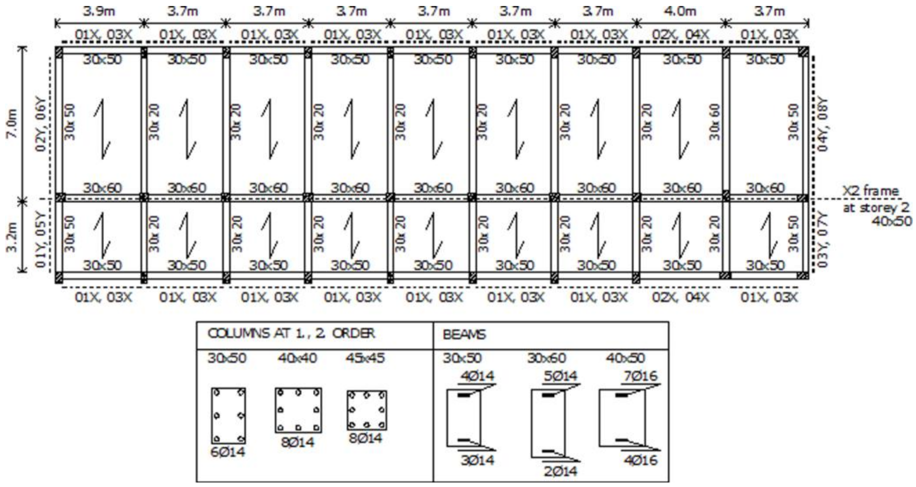

Figure 5: The case study: structural plan of the ground floor and scheme of the longitudinal reinforcement in the main structural elements.

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*" [5,6].

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 [24]. 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 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). It is actually very similar to the infill panels used in the research work Bertoldi et al. (1993) [19] 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.

f_{Wv} *[MPa]*	f_{Wu} *[MPa]*	f_{Ws} *[MPa]*	E_{Wh} *[MPa]*	E_{Wv} *[MPa]*	G *[MPa]*	W *[MPa]*
1.5	0.25	0.31	991	1873	1089	6.87

Table 1. Mechanical parameters adopted for the infill wall

3.3 Numerical modeling

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" [25]. 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); "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 6, the three-dimensional model of the structure used for the analysis in the + X direction is shown).

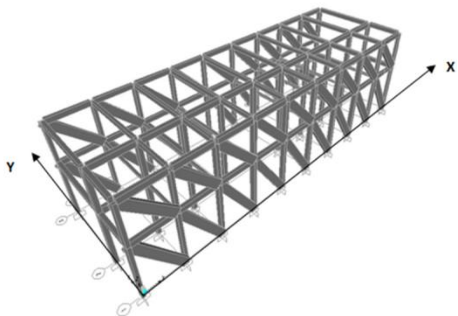

Figure 6: The 3D frame model with the single equivalent struts in +X

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 et al. (1984) [26]; elastic – hardening diagram for the steel. The non linear behaviour of columns and beams was described according to a lumped plasticity approach. According to this, the post-elastic behaviour is modelled by introducing plastic hinges, in which all non linearity is localized at the ends of the elastic beams representing the structural elements.

In order to appraise the alteration of the structural behaviour in the presence of the reinforced infill panels, the increased mechanical parameters of the infill 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, in both kind of interventions, 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)[27] 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. does not fully reproduce the actual behaviour in the case of a reinforced panel.

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 center of the panel.

In Figure 7 respectively for the reinforced structure (R-IS model) and the unreinforced structure (IS model), the hysteretic curves obtained for the 4 struts for the pushover analysis in the X direction are shown.

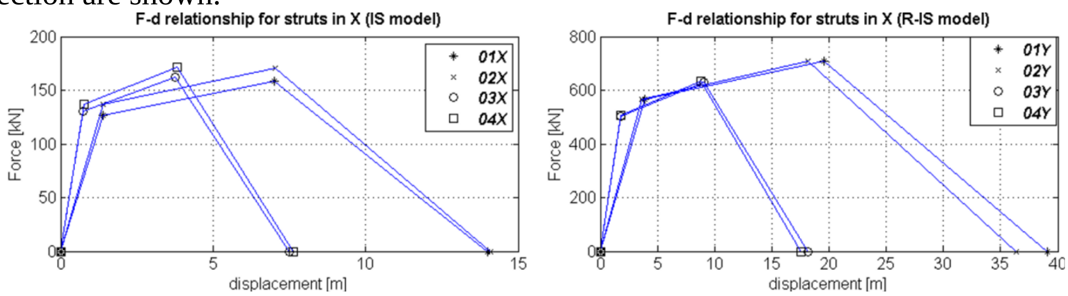

Figure 7. the force–displacement relationship of the diagonal struts (under compression) for IS and R-IS models in the X direction.

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 RESULTS

The performance capacity has been calculated by non linear pushover analysis at the Limit States of Damage (SLD), Life Safety (SLV) and Near Collapse (SLC), for the three models (B, IS, R-IS). 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).

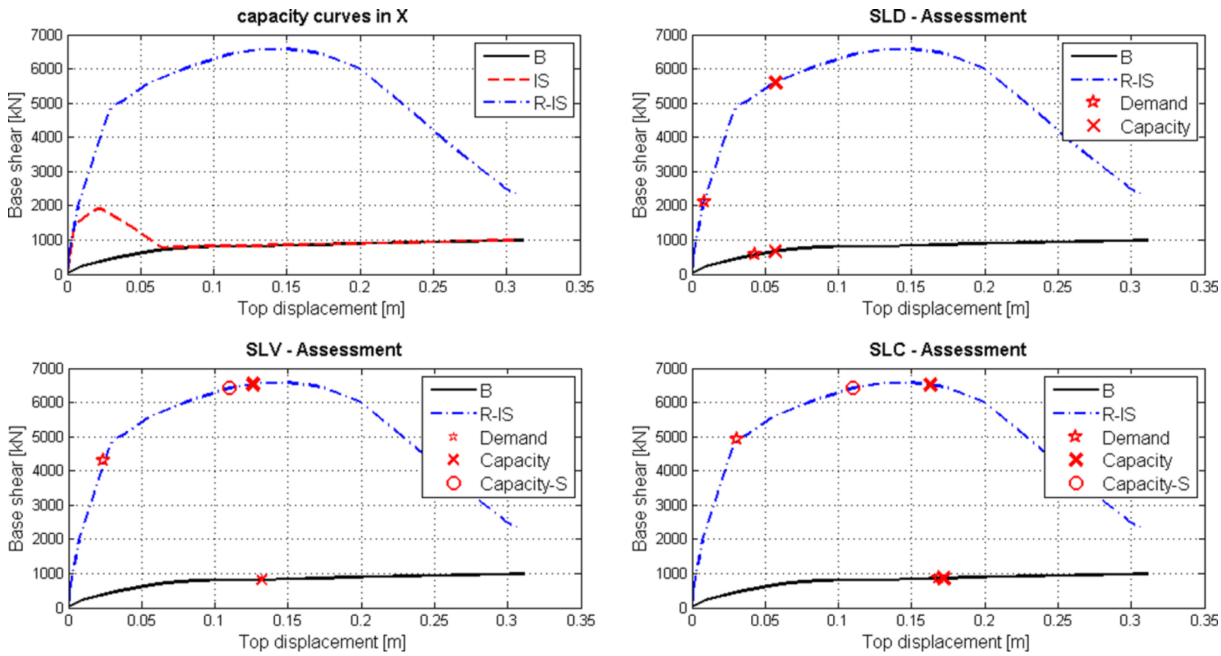

Figure 8: Pushover analyses in the X direction: comparison between the capacity curves for the three models B, IS, R-IS and graphic performance assessment at the different limit states.

As previously mentioned, the presence of the infill increases of stiffness and strength of the structure. This is confirmed by the curves shown in Figure 8, which shows the results of the pushover analyses in the X direction (where there are more infill panels). 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 figures, the seismic demand and the actual structural capacity at the different limit states are also indicated. It should be precised 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 [28] (original version implemented in EC8) both for the infilled frame and the bare structure. 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"). The results graphically shown in Figure 8 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.

Pushover in X direction	SLD		SLV		SLC	
	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

Table 2. Seismic demand and structural capacity at the different limit states for the pushover analyses in the X direction for the models B and R-IS.

The reinforcement intervention increases the structural ductility with respect to the bare frame: in particular, the increments at the different limit states are equal to $4.79\mu_{SLD}^{B}$, $4.58\mu_{SLV}^{B}$ and $3.50\mu_{SLC}^{B}$ in the X direction.

5 CONCLUSIONS

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:

- The solidarization of the infill panel to the RC frame allows 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 node). 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.
- 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.

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