Seismic response of an existing RC frame building struck by 2016 Central Italy earthquakes

A.V. Bergami[1]; D. Lavorato[1]; G. Fiorentino[1]; C. Nuti[1]

[1] *Department of Architecture, University Roma Tre, Rome (Italy)*

ABSTRACT: A large number of research studies have been devoted to the modelling and analysis of infilled Reinforced Concrete (RC) framed buildings under seismic actions; it is well known that infills play a significant role in the overall structural performance. The present work reports the results of the nonlinear static assessment performed on a masonry infilled RC frame building retrofitted with dissipative braces, located in the area struck by the 2016 Central Italy seismic sequence. The building is an interesting case study because it is equipped with dissipative braces and with a monitoring system; the monitoring system recorded the building response to the seismic sequence and consequently the evolution of the structural response with the progressive deterioration of the infill. Numerical analyses were performed by using nonlinear 3D models, considering both the bare and the infilled frame, in order to appraise the interaction of the infills with the RC elements and the dissipative contribution offered by the dissipative bracing system; an alternative retrofitting approach, finalized to prevent non-structural damage, according to the Bergami-Nuti procedure is finally proposed.

KEYWORDS: masonry infilled frames, dissipative braces, seismic assesment, seismic sequence

1 SCOPE OF THE WORK

The aim of this work is to discuss the seismic response of De Gasperi Battaglia School building located in Norcia (Umbria, Italy), an existing masonry infilled Reinforced Concrete (RC) frame building equipped with dissipative braces.

In the paper the building history and characteristics are provided. The structure, equipped with a monitoring system, was retrofitted few years before the seismic events happened in 2016; a complete survey of the building was performed and an additional system of dissipative braces were installed to obtain a seismic retrofitting according to the Italian technical code NTC 2008. The seismic events of 2016 Central Italy sequence struck the building and in particular two major events: 24 August 2016 (Ea), at 3:36 (magnitude M_W 6.0) with epicenter located along the Valle del Tronto, between the municipalities of Accumoli (RI) and Arquata del Tronto (AP), and 30 October 2016 (Eo), the strongest event (magnitude M_W 6.5), with epicenter between the municipalities of Norcia and Preci (a near field event for the case study). In the paper is discussed and analyzed how the infill contribution strongly influenced the seismic performance of the building during the seismic sequence and, therefore, how this “variable” can compromise the design hypothesis: dynamic response, modal analysis, displacements and therefore the dissipative braces contribution (dissipative braces are commonly designed neglecting the infills in the analyses. An alternative approach for the design of the dissipative braces is finally proposed.

2 INTRODUCTION

A great part of the existing structures built in Italy is composed by masonry infilled RC frames that have been designed without considering seismic induced actions and seismic criteria for strength and ductility design; the inadequate safety level can be ascribed to the poor constructive detailing, the simplified design approach (e.g. lack of a real tridimensional framing system or any design of beam-column joints) and of course to structural deterioration. In this context the contribution of the infills to the global and local seismic response of the building, hence, assumes a fundamental role considering that the design of the reinforced concrete frame buildings (RCF) is usually performed without considering the infills and, as previously discussed, it was intrinsically defective. The influence of the infill walls can be, however, very different, depending on their mechanical features, geometrical configuration and spatial distribution within the building. The scientific community is still working on this issue at many levels. After seismic events that have occurred in the last 10 years in Italy, the observation of the damage patterns revealed that the performance of RCF buildings was significantly influenced by the presence of infill walls. Two categories of buildings can be distinguished: modern buildings designed following a specific seismic code and existing buildings designed only to resist to gravity loads. In both cases, the structural response of RCF may be positively or negatively conditioned by the non-structural masonry infills.

In literature this specific issue has been discussed widely discussed in Bergami et al. (2013-2017) also proposing solution for the infill modelling and the dissipative bracing design.

In this paper, with the aim of appraising the influence of the infill panels over the global response under the seismic actions, the case study of an existing school building in Norcia is discussed; the building is equipped with a monitoring system that recorded the Central Italy seismic sequence of 2016. In particular the effectiveness of the dissipative bracing system design to retrofit the building will be analyzed considering the contribution offered by the infills during the seismic sequence.

3 DESCRIPTION OF THE BUILDING

3.1 *General description of the structure*

The building chosen as a case study is school building located in a high seismic risk area (Norcia, Province of Perugia, Central Italy). The school building was built in the '60s (at that time, the in force seismic codes were the Law No. 64 of 02.02.1974, and the Ministerial Decree 03/03/1975, which nowadays have been completely overcome); compatibly with the calculation methodologies of the time, the structure was designed by analyzing 2D frames schematizing the reinforced concrete frames arranged in the transverse direction of the building. The building was originally an aggregate, designed considering three separate blocks aligned longitudinally, that has been solidarized into a single body in the years following the construction. Although not explicitly required by the legislation of the time, the designer also took into account the seismic action by applying an horizontal acceleration of 0.07 g dimensioning the frames.

After the Central Italy earthquake 1997 (Umbria-Marche) that severely damage it, was widely investigated and studied:

- destructive and non-destructive investigations
- a monitoring system was installed so the building was equipped with 11 accelerometers (5 monoaxial, 5 biaxial and 1 free triaxial field) for a total of 18 channels of recording.

In decade 2003-2013 a structural retrofitting was designed and realized with the insertion of some dissipative braces. The devices are BRAD® (buckling restrained braces) type 14/40-b produced by Fip Industriale spa (Italy).

The building consists of a basement (level -1), a ground floor (level 0), two additional floors (level 1 and 2) and a not practicable attic (level 3) and the main structure consists in a set of reinforced concrete frames with masonry infill, arranged in two main directions

Figure 1. Main façade of the building

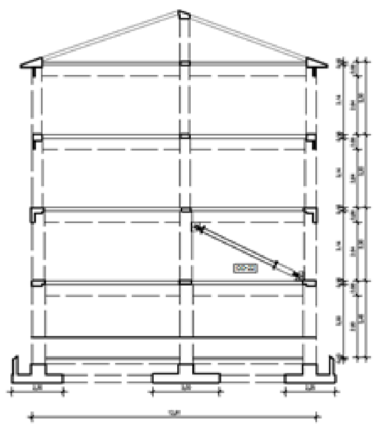

Figure 2. Typical transversal frame of the building structure with dissipative braces

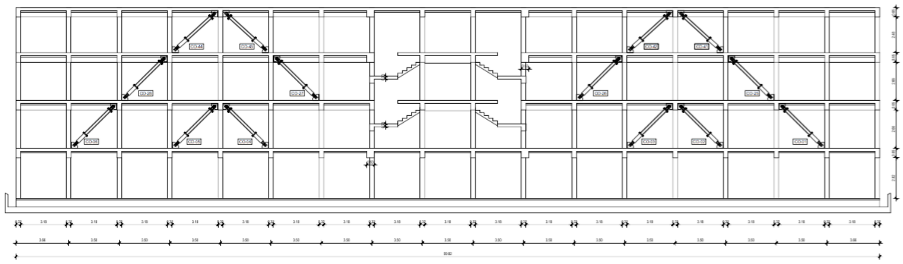

Figure 3. Typical longitudinal frame of the building structure with dissipative braces

With regard to the masonry infill panels, a complete review and classification was performed, identifying the constructive features, materials and recurrent dimensions. The characteristics of the infills (typology, geometry and position) have been defined for all the eight types of infill identified (four type external walls and four type of internal partitions – Figure 4-7). The mechanical parameters of the infills have been derived according to other studies on similar materials (Bergami et al., 2016) and are summarized in Table 1 and 2.
From a structural survey the mechanical parameters reported in Table 3 have been determined.

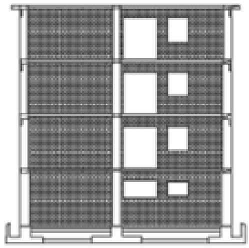

Figure 4. Transversal façades: layout of the masonry walls

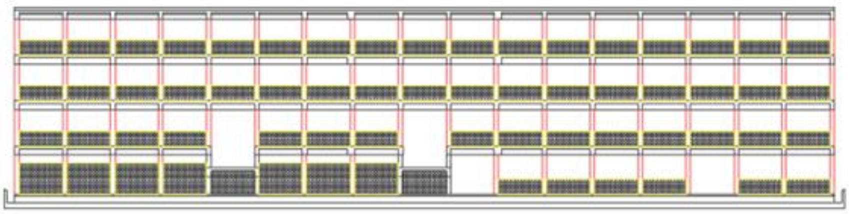

Figure 5. Longitudinal façades: layout of the masonry walls

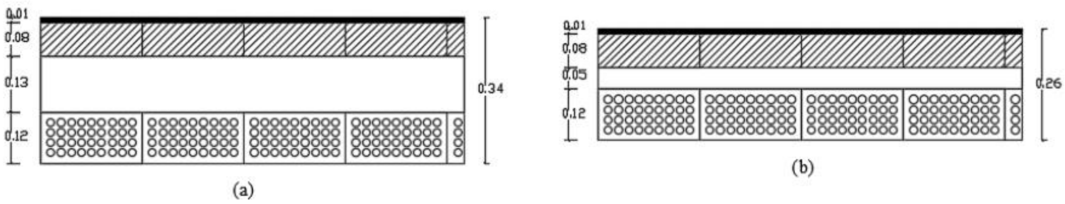

Figure 6. Masonry walls of the external frames: walls of the first interstorey (a), walls of the higher levels (b). Dimensions in meters [m]

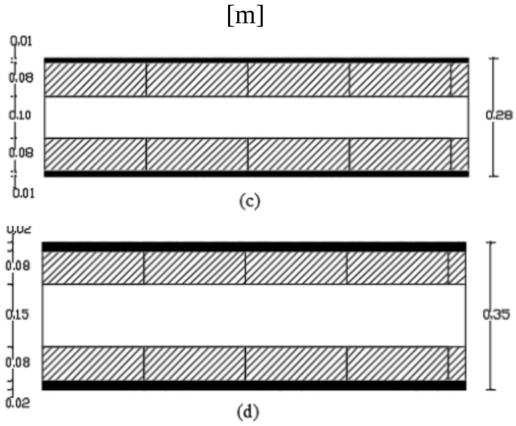

Figure 7. Masonry walls realized in the internal frames: type 1 (a), type 2 (b). Dimensions in meters [m]

According to the design report the rebars are classified as FeB38k (Italian Technical Code D.M. 1996).

Table 1. Masonry - external layer [Mpa]

Masonry type 1 (external layer) – data from literature		
f_v	7.93	Average vertical comp. strength
f_h	3.09	Average horizontal comp. strength
E_v	10167.0	Elastic modulus - vertical
E_h	4888.0	Average vertical comp. strength

Table 2. Masonry - internal layer

Masonry type 2 (internal layer) – data from literature		
f_v	1.9	Average vertical comp. strength
f_h	3.11	Average horizontal comp. strength
E_v	4804.25	Elastic modulus - vertical
E_h	4325.50	Average vertical comp. strength

Table 3. Structural materials

Concrete			
f_{cm}	26.63	MPa	Average compression strength of the concrete
E_c	22000	MPa	Elastic modulus of the concrete
Rebars			
f_{ym}	374	MPa	Average yelding stress of rebar
E_s	210000	MPa	Elastic modulus of rebar

3.2 *Dissipative bracing system*

The dissipative braces installed in the building have ben characterized according to the data provided by the manufacturer (Fip industriale spa; Table 4); the verification tests performed on the devices installed have been also provided allowing a detailed characterization of the devices in the numerical model developed for this study.

Table 4. Dissipative braces: main characteristics

F_1	119	kN	Yelding force – bilinear cycle
K_0	60	kN/mm	Stiffness of the first branch of the bilinear cycle
$F_{a,3}$	129	kN	Average force @ cycle 3 and displacement d_{bd}
d_{bd}	±20	mm	Design displacement
$F_{c,3}$	140	kN	Compression force @ cycle 3 and displacement d_{bd}

Figure 8. Dissipative device BRAD 14/40-b installed in the building

3.3 *Monitoring system*

As previously mentioned, the building was equipped with 11 accelerometers (5 monoaxial, 5 biaxial and 1 triaxial free field) for a total of 18 recording channels distributed as described in Figure 8.

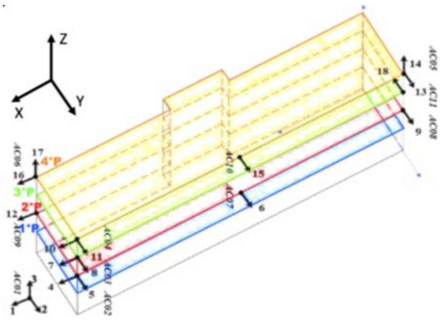

Figure 9. Layout of the monitoring system / Schema del Sistema di monitoraggio.

The monitoring system registered the building response to the whole seismic sequence of central Italy (2016-2017) and therefore also the main seismic events, 24th August 2016 and 30th October 2019, selected in this work as case study.

4 THE SEISMIC RESPONSE OF THE BUILDING

As previously mentioned, the building is equipped with a monitoring system that was able to record the building response to the 2016 sequence that struck the town where the school is located; the response spectra of both the seismic events considered, compared with the response spectra used to design the seismic retrofitting of the school, are plotted in Figure 10. As shown in this figure, both the seismic events are very intense for the periods lower than 0,4s. As can be observed, the events are comparable only if the x direction is considered; the *Eo* as similar components (it is a near field event) whereas the *Ea* is strongly oriented along x and the y component is strongly lower. Comparing the design elastic spectra and the event spectra, one can observe that both the earthquakes, and in particular *Eo* , have been intense if the SLV spectra (elastic spectra for the life safety limit state) is considered as a capacity indicator of the building and SLC spectra (elastic spectra for the collapse prevent limit state) is considered as a capacity indicator for the dissipative devices.

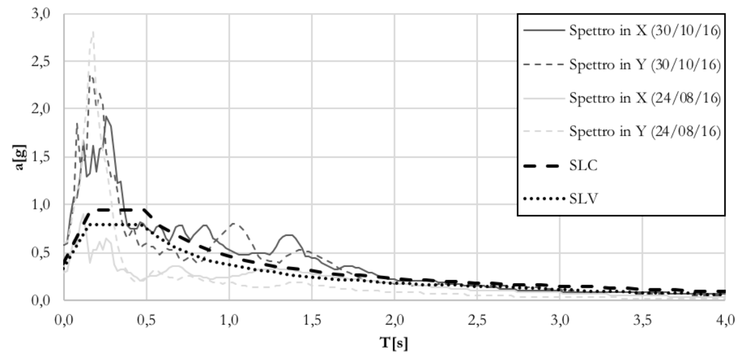

Figure 10. Response spectra of the seismic events considered compared with the response spectra used to design the retrofitted building (according to the Italian technical code, NTC 2008) for both the collapse prevent (SLC) and life safety (SLV)

In the cited range of periods (T<0,4s), the *Eo* has been intense for both the direction (x and y) whereas the *Ea* only for the y component, therefore along the x directions minor damage are expected (in particular for the infills oriented along the longitudinal direction. Therefore the modal response of the building is a very relevant parameter because only for periods higher than 0,4s the design procedure performed for the retrofitting; the evolution of modal properties during the seismic sequence is therefore fundamental. Is indeed well known that, during a seismic sequence, the structural and non-structural elements are damaged and, consequently, the stiffness of the framing structure and the stiffening contribution of the infills can decrease.

This considerations have been confirmed by a numerical study of the data available from the monitoring system and a damage survey.

Indeed, the analysis of the response to *Ea* and *Eo* demonstrate that, within the sequence of seismic events, the building changed its dynamic response, and in particular the fundamental period of the structure, due to a double effect: cracking of the elements in c.a. and progressive deterioration of the infill.

The relevance of the contribution offered by the infill was found by analyzing the records of two events of the seismic sequence: *Ea* (24 August 2016) and *Eo* (30 October 2016).

The most intense event, for the School was, certainly, *Eo* which had its epicenter precisely in that area (it can be classified as a near fault event).

In line with what observed in the previous paragraph, performing an FFT analysis the dynamic response of the building and the main period of the structure were identified (Figure 11 and 12).

The building is characterized by natural periods that, analyzing the first earthquake (august 24th) and the strongest earthquake (October 30th) evolved from 0.38s to 0.57s in the long. direction and from 0.27s to 1.07s in the tansv. direction.

The findings confirm the loss of stiffness of the building and in particular an evolution of the most significant damage in the transv. direction were the *Ea* was more intense.

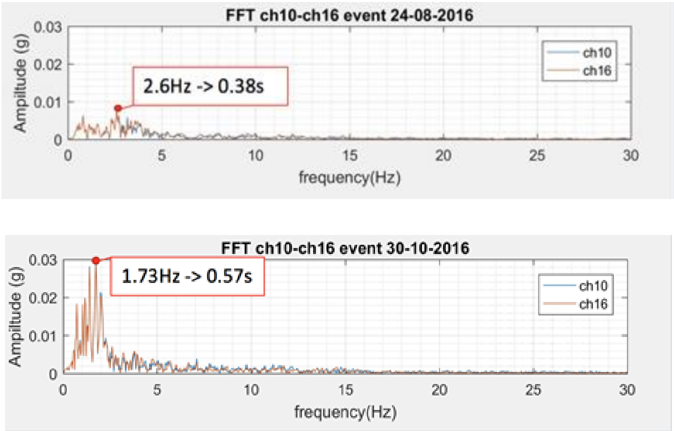

Figure 11. FFT analysis: evaluation of the fundamental period for the two main seismic events considered – transverse (y) direction

The accelerations recorded by the free field sensor and by all the other sensors installed inside the building have been analysed and in particular the data (displacements and accelerations) along longitudinal (x) and transversal (y) direction. Comparing data registered from the instruments disposed at the same level, but in opposite sides of the plan, a negligible torsional effect has been observed for the *Ea* whereas a more relevant for *Eo*.

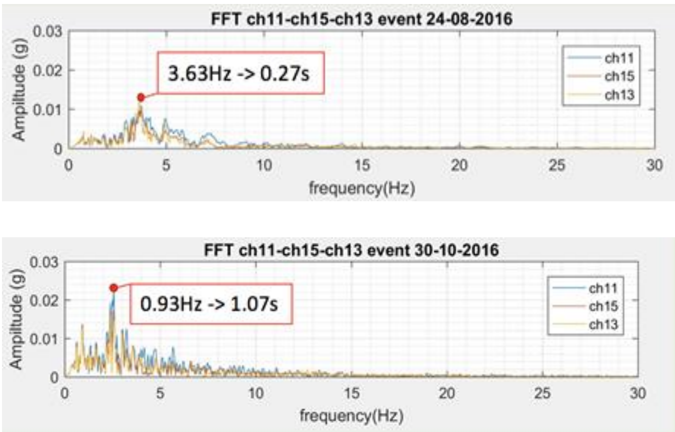

Figure 12. FFT analysis: evaluation of the fundamental period for the two main seismic events considered – long. (x) direction

Therefore, according to FFT results (Figure 11 and 12) it is reasonable to assume that the building, which subsequently have been stuck by the seismic sequence that preceded the Eo event, has progressively reduced its horizontal stiffness (more significantly in the y direction).

Adopting as damage parameter the drift and in particular:

– İnterstorey drift = 0.1% selected as indicator for structural and non structural damage;
– İnterstorey drift = 0.3% selected as indicator for severe damage)

The evolution of the damage state has been identified.

In the *Ea* (Figure 13) the structure was damaged (a drift of 0.1% is reached in all the storey and, only for the y direction, at the first level a soft storey mechanism was activated (drift of 0.65%).

With the *Eo* event (Figure 14), the response is simlar for both the directions; the range of displacements is wider and therefore the interstorey drift, as a consequence of the progressive stiffness reduction (according to the damage state previously discussed). With the *Eo* the non structural damage is more relevant and the contribution offered becomes negligible.

The numerical analyses previously discussed are confirmed by the damage survey performed after the two seismic events. In Figure 15-17 the damage state of the infill walls is shown.

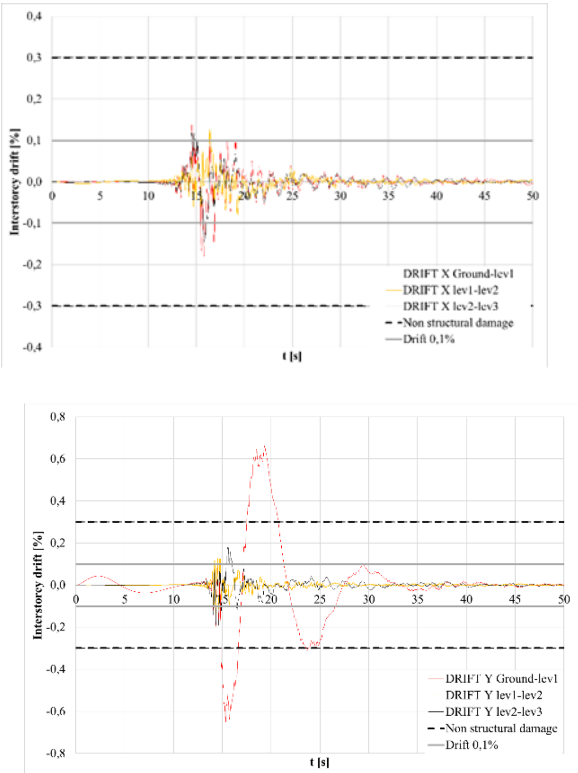

Figure 13. Interstorey drift evaluated processing the monitored data: seismic events of 2016/08/24 (direction x and y)

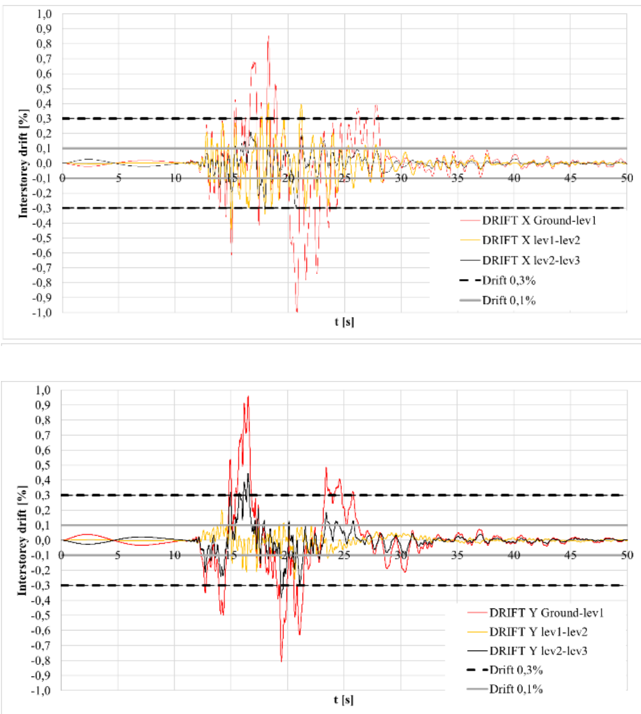

Figure 14. Interstorey drift evaluated processing the monitored data: seismic events of 2016/10/30 (direction x and y)

After the *Eo* many infill panels were severely damaged and, in some cases, expelled out of the framing plane. Therefore the interaction infill-frame was still active after the *Ea* but was substantially eliminated during the *Eo*. Therefore, according to what observed and derived from the data analysis, the building response evolved from the reponse of a masonry infilled frame to the response of a bare frame. This evolution is relevant and, if not considered during the design phase, can invalidate the design bases compromising the performance attended.

Figure 15. Damage state on the external infill after *Ea* (left) and *Eo* (right)

Figure 16. Damage state on the external infill after *Ea* (left) and *Eo* (right)

Figure 17. Damage state on the internal infill after *Ea* (up) and *Eo* (bottom)

5 NUMERICAL MODELLING

A numerical model of the building has been realized using ‘SAP2000’ (Computer &Structures Inc., 2010) considering different configurations:

a) bare frame – the RC frame only that is the structural system considered by the designer of the retrofitting system with dissipative braces;
b) infilled frame – that is the real structural system existing before the retrofitting project;
c) retrofitted building – that is the building existing during the seismic events;
d) retrofitted building designed according to an alternative procedure (Bergami et al., 2013) in order to prevent damages on the infills.

In the numerical model the nonlinearity are concentrated in elements extremities: the plastic hinges (in the numerical model fibre hinges were used). These plastic hinges represent points where the nonlinear material deformation occurs, being situated through elements’ extremities because of higher bending forces. In the generic section, the stress and strain states are obtained by integrating the axial response of all the fibres of the

section. Section are characterized by unconfined concrete (concrete cover), confined concrete (concrete area comprised within the transversal reinforcements) and steel.
In order to model masonry infill as an equivalent strut, a global modelling approach was used: the approach consist in replacing, in order to simulate infill-structure interaction, the wall with a strut.
Pin jointed struts have been used defining the nonlinear behaviour of the infill materials; the strut elements has tension limit set to zero and zero mass and wigth (the infill has been incorporated as equivalent distributed load on the beams). In SAP 2000 the strut element is defined by means of a non-linear link element characterized by a user defined load-deformation diagram that describes the non-linear behavior of infill specifically defined for the masonry of the case study.

5.1 *Infills: modelling*

The characterization of the nonlinear behavior of infills is critical; in particular to simulate the post-peak behaviour of the building response to seismic action. Therefore the definition of the constitutive model (load-displacement and stress-strain curves) represents an important phase to gain the real structural behaviour. The model adopted in this work is a curve, proposed by Combescure (1996), consisting of 4 branches (Figure 18) to be calibrated, as suggested by the author, on the basis of experimental data or a literature review.

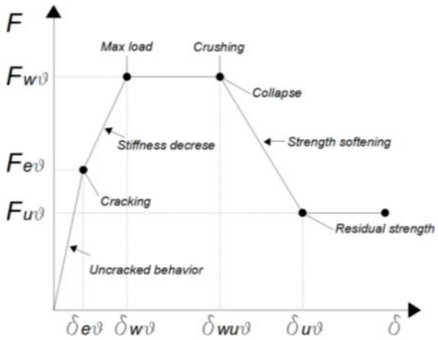

Figure 18. Strut constitutive law (Combescure 1996)

with compression strength and diagonal elastic modulus defined as follow:

$$F_{w\theta} = f_w \cdot t_w \cdot b_w \qquad (1)$$

The cracking load of the infill has been assumed as the 50% of the maximum load and therefore:

$$\mathrm{F}_{e\theta} = \frac{\mathrm{F}_{w\theta}}{2} \qquad (2)$$

Displacement in correspondence with the first crack may be defined as:

$$\delta_{e\theta} = \frac{F_{e\theta}}{2K^*{}_{w\theta}} \qquad (3)$$

The second branch of the curve has an inclination equivalent to the stiffness of the infill, therefore the horizontal displacement value for which the peak load is obtained is given by:

$$\delta_{w\theta} = \frac{f_w}{E_{w\theta}} d_w \qquad (4)$$

with:
d_w *length of the diagonal strut*
and, according to *Combescure* (1996):

$$\delta_{wu\theta} = 0,005 \frac{h_w}{\cos\theta} \qquad (5)$$

$$\delta_{u\theta} = 18\delta_{w\theta} \qquad (6)$$

$$F_{u\theta} = \frac{F_{w\theta}}{10} \qquad (7)$$

5.2 *Dissipative braces: modelling and design*

The building has been modeled considering both the existing configuration (Retrofitted building) or an alternative solution in which the dissipative braces have been re-designed according to a different procedure: the Bergami-Nuti procedure.

The Bergami-Nuti procedure is a pushover based method and, in this case, the use of a multimodal incremental analysis named IMPA (Bergami, 2017) was selected being the building irregular and therefore sensitive to higher modes. Appling the procedure, differently from other approaches, the presence of the infills has been considered and therefore the infilled frame has been adopted has representative of the structure to be retrofitted. Defining the performance target, with this procedure it is possible to impose a displacement limit that will avoid damage on both RC elements and masonry panels. Therefore the target displacement has been selected adopting the following parameters: reducing the top displacement to control, in all storey, the interstorey drift under 0.2%. In both the numerical model, the retrofitted build. and the alternative retrofitted build. The dissipative braces have been modelled according to the real characteristics of existing or selected devices. Therefore the constitutive link to be attributed to the non-linear link elements with which they were modeled was reconstructed. It should be noted that the dissipative brace consists of two elements in series: the dissipative device and a metal profile used to connect the device to the structural frame. It is therefore necessary that the numerical model holds against the serial behavior of these two elements.

5.3 *Seismic vulnerability*

The numerical model has been used to assess the seismic vulnerability of the building considering two different configurations fr the existing building (before the retrofitting) and for the retrofitted building (as it is or according to the Bergami-Nuti design procedure for dissipative braces)

For brevity, only few results are shown in Figure 19; the capacity curves derived performing a pushover analysis on each building model demontrate how relevant is the contribution offered by the infill in terms of stiffness and base shear and that, according to the Bergami-Nuti procedure an optimized bracing system can be derived obtaing the same performances, in terms of global behaviour, but preserving the integrity of the infills: the deformed shape is regularizedand therefore the sam top displcement is reached with a regular distribution at all levels and without overpassing the damage limit.

The real experience, according to the design approach adopted for the existing retrofitting system, demonstrated that the damage on the infill can compromise the funtionality anc it is a relvan risk for life (pannel collapse).

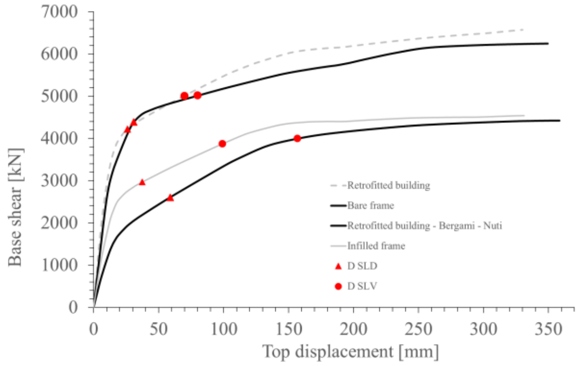

Figure 19. Non-linear static analysis (longitudinal direction): comparison of the configurations considered in terms of capacity curves and demand estimation (D) for each limit state considered

6 ACKNOWLEDGMENTS

The authors gratefully acknowledge the funding received by The Laboratories University Network of Seismic Engineering (ReLUIS): research project ReLUIS/DPC 2019-2021 Reinforced Concrete Existing Structures.

7 REFERENCES

Bergami A. V., Fiorentino g., Lavorato d., Briseghella B, Nuti C. (2020). Application of the Incremental Modal Pushover Analysis to Bridges Subjected to Near-Fault Ground Motions. APPLIED SCIENCES, vol. 10, ISSN: 2076-3417, doi: https://doi.org/10.3390/app10196738

Bergami, A.V., Nuti, C., Lavorato, D., Fiorentino, G., & Briseghella, B. (2020). IMPAβ: Incremental Modal Pushover Analysis for Bridges. APPLIED SCIENCES, 10(12), 4287

Bergami A.V., Forte A., Lavorato D., Nuti C. *Proposal of an Incremental Modal Pushover Analysis (IMPA)*. Techno Press, Earthquake & Structures, Vol. 13, No. 6 (2017) 539-549, ISSN: 2092-7614 (Print).

Bergami A. V., Forte A., Lavorato D., Nuti C. Non linear static analysis: application of existing concrete building. Atti di ITALIAN CONCRETE DAYS, 2016. 27 - 28 october 2016, Roma.

Bergami, A.V., Nuti, C., Liu, X. *Proposal and application of the Incremental Modal Pushover Analysis (IMPA)*. IABSE Conference, Geneva 2015: Structural Engineering: Providing Solutions to Global Challenges - Report pp. 1695-1700

Bergami A.V., Lavorato D., Nuti C., Fiorentino G. *Proposal of a non linear static analysis procedure for bridges: the Incremental Modal Pushover Analysis for bridges*. Assessing and Extending the Service Life of Bridges, a special issue of Applied Sciences (ISSN 2076-3417), 2020.

Bergami A.V., Nuti C. (2015). Experimental tests and global modeling of masonry infilled frames. Techno Press, Earthquake & Structures, ISSN: 2092-7614, Vol. 9, No. 2, pp. 281-303.

Bergami A.V., Nuti C. (2014). Design of dissipative braces for an existing strategic building with a pushover based procedure. Journal of Civil Engineering and Architecture, USA, ISSN: 1934-7359, Vol. 8, No. 7 (Serial No. 80), pp. 815-823

Bergami A.V., Nuti C. (2013). A design procedure of dissipative braces for seismic upgrading structures. Techno Press, Earthquake & Structures, ISSN: 2092-7614, Vol. 4, No. 1, pp.85-108.

Lavorato D, Fiorentino G, Pelle A, et al. A corrosion model for the interpretation of cyclic behavior of reinforced concrete sections. Structural Concrete. 2019;1–15. https://doi.org/10.1002/suco.201900232

Lavorato, D., Bergami, A.V., Fiorentino, G., Fiore, A., Santini, S., & Nuti, C. (2018). Experimental tests on existing RC beams strengthened in flexure and retrofitted for shear by C-FRP in presence of negative moments. INTERNATIONAL JOURNAL OF ADVANCED STRUCTURAL ENGINEERING, 10(3), 211-232.

Lavorato, D., Bergami, A.V., Forte, A., Quaranta, G., Nuti, C., Monti, G., Et Al. (2018). Influence of Materials Knowledge Level on the Assessment of the Characteristic Value of the Shear Strength of Existing RC Beams. In PROCEEDINGS OF ITALIAN CONCRETE DAYS 2016.

Lavorato D, Fiorentino G, Pelle A, et al. A corrosion model for the interpretation of cyclic behavior of reinforced concrete sections. Structural Concrete. 2019;1–15. https://doi.org/10.1002/suco.201900232

Lavorato, D., Bergami, A.V., Fiorentino, G., Fiore, A., Santini, S., & Nuti, C. (2018). Experimental tests on existing RC beams strengthened in flexure and retrofitted for shear by C-FRP in presence of negative moments. INTERNATIONAL JOURNAL OF ADVANCED STRUCTURAL ENGINEERING, 10(3), 211-232.

Lima, C., Angiolilli, M., Barbagallo, F., Vecchi, F., Verderame, G.M., Bergami et al. (2020). Nonlinear Modeling Approaches for Existing Reinforced Concrete Buildings: The Case Study of De Gasperi-Battaglia School Building in Norcia. Lecture Notes in Civil EngineeringVolume 42, Pages 82-95.

Xue, J., Lavorato, D., Bergami, A.V., Nuti, C., Briseghella, B., Marano, G.C., et al. (2018). Severely damaged reinforced concrete circular columns repaired by turned steel rebar and high-performance concrete jacketing with steel or polymer fibers. APPLIED SCIENCES, 8(9), 1671.

Uva G, Porco F, Fiore A. Appraisal of masonry infill wall seffectin the seismic response of RC framed buildings: acasestudy.EngStruct2012;34:514–26.

Fiore A, Netti A, Monaco P. The influence of masonry infill on the seismic behaviour of RC frame buildings.EngStruct2012;44:133–45.

Cavaleri L, Di Trapani F, Macaluso G,Papia M. Reliability of code-proposed models for assessment of masonry elastic moduli. IngSism2012;29(1): 38–59.