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Seismic Behaviour and Retrofitting of a School Masonry Building subject to the 2012 Emilia-Romagna Earthquake

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Abstract

In this paper the seismic vulnerability of a school building located in Bomporto (a district of Modena), built around the second half of the XIX century and hit by the Italian Emilia-Romagna earthquake, has been assessed. Non-linear seismic analyses of the building have been conducted using two calculation programs, namely 3DMacro, with diagonal springs representing the behaviour of masonry walls under compression and tension, and 3MURI, the latter allowing for a masonry walls modelled with the equivalent frame technique, where masonry piers and spandrels are idealised with beam elements connected each to another with rigid links. First, the results achieved from two programs have been compared with the aim of identifying the most probable building seismic response. Moreover, in order to identify the main structural parts which should be retrofitted, a comparison between in-situ visual damage and numerical results has been undertaken. Subsequently, two different structural reinforcement interventions, the first with composite materials with the form of bidirectional FRP strips and the second with bidirectional steel strips, that have been first designed and then applied to the building studied and then compared with each other in terms of cost and sustainability parameters. Finally, a parametric analysis varying the geometrical configuration of the FRP intervention (strip width and spacing) has been performed with the purpose of identifying, through a cost-to-benefit comparison, the optimal retrofitting solution.

Keywords: Emilia-Romagna earthquake, school building, 3DMacro model, 3MURI model, non-linear analysis, retrofitting intervention.

1 The 2012 Emilia-Romagna earthquake

In May 2012, two major earthquakes hit the Emilia-Romagna region in northern Italy. They started on Sunday 2012 May 20th and Tuesday 2012 May 29th and affected the provinces of Ferrara, Bologna, Modena and Reggio Emilia [1].

These earthquakes happened in the Po plain, a not very active seismic zone for Italian standards, which contemplate it as a low-to-moderate hazard area with a very high exposure. Previous seismic events were dated in 1346, where a magnitude 5.8 earthquake took place approximately at the location of the 2012 May 29th event, and from 1561-1574, where a sequence of four events was felt with intensity greater than 7 in the region, with the largest being the November 1570 seism of a very shallow depth and magnitude 5.5 (EMS=7.5), occurring about 40 km east of the 2012 May 20th event [2]. A repetition of this event, could lead to even larger damage than that of the recent earthquakes, depending on the proximity of its epicentre to the city of Ferrara.

The first earthquake, recording magnitude 5.9, struck in the Emilia-Romagna region, about 36 kilometres (22 mi) north of the city of Bologna, at 04:03 local time (02:03 UTC). The epicentre was between Finale Emilia, Bondeno and Sermide. It was followed by several aftershocks, two having a magnitude of more than 5. From this earthquake 7 fatalities (4 direct, 3 indirect (heart attacks)), around 50 injured, 6000 homeless and 400-500 million Euros losses (0.3% of Emilia-Romagna's GDP) were recorded.

The main shock of the second earthquake, having a magnitude of 5.8, happened at 9:00 a.m. and was also followed by two events of more than magnitude 5 on the same day. It caused an additional twenty deaths and widespread damage, particularly to buildings already weakened by the May 20th earthquake. The epicentre was in Medolla, at a depth of about 10 kilometres. From this second seism 17 fatalities, 350 injured, 9000 additional homeless and estimates of loss exceeding that of the first event (>500 million Euros) were detected.

Therefore, for both earthquakes, whose seismic sequences are reported in Table 1, felt intensities exceeded the value of 7 on the Modified Mercalli Scale, which has the highest value equal to 12.

Date	Local Time	Magnitude	Epicentre
2012/05/20	01:13:27	4,1	Bondeno (Fe)
2012/05/20	04:03:52	5,9	Finale Emilia (Mo) - Bondeno (Fe) - Sermide (Mn)
2012/05/20	04:06:30	4,8	Finale Emilia (Mo)
2012/05/20	04:07:31	5,1	Bondeno (Fe)
2012/05/20	04:11:46	4,3	Bondeno (Fe)
2012/05/20	04:12:42	4,3	Finale Emilia (Mo)
2012/05/20	04:21:53	4,1	Mirandola (Mo)
2012/05/20	04:25:05	4,0	Bondeno (Fe)
2012/05/20	04:35:37	4,0	Ferrara
2012/05/20	04:39:10	4,0	Finale Emilia (Mo)

Date	Local Time	Magnitude	Epicentre
2012/05/20	05:02:50	4,9	San Felice sul Panaro (Mo)
2012/05/20	11:13:21	4,2	Finale Emilia (Mo)
2012/05/20	15:18:02	5,1	Vigarano Mainarda (Fe) – Mirabello (Fe)
2012/05/20	15:21:06	4,1	Bondeno (Fe)
2012/05/20	19:37:14	4,5	Bondeno (Fe)
2012/05/21	18:37:31	4,1	Finale Emilia (Mo)
2012/05/23	23:41:18	4,3	Finale Emilia (Mo)
2012/05/25	15:14:05	4,0	Mirandola (Mo)
2012/05/27	20:18:45	4,0	Mirandola (Mo)
2012/05/29	09:00:03	5,8	Medolla (Mo)-Cavezzo (Mo)
2012/05/29	09:07:21	4,0	Cavezzo (Mo)
2012/05/29	09:09:54	4,1	Novi di Modena (Mo)
2012/05/29	10:25:51	4,5	Novi di Modena (Mo)
2012/05/29	10:27:23	4,7	San Felice sul Panaro (Mo)
2012/05/29	10:40:58	4,2	Mirandola (Mo)
2012/05/29	11:30:21	4,2	Concordia sulla Secchia (Mo)
2012/05/29	12:55:57	5,3	San Possidonio (Mo)
2012/05/29	13:00:02	4,9	Cavezzo (Mo)
2012/05/29	13:00:25	5,2	Novi di Modena (Mo) - Moglia (Mn)
2012/05/29	13:07:05	4,0	San Possidonio (Mo)
2012/05/31	16:58:21	4,0	Novi di Modena (Mo) - Rolo (Re)
2012/05/31	21:04:04	4,2	San Possidonio (Mo)
2012/06/03	21:20:43	5,1	Novi di Modena (Mo)
2012/06/12	03:48:36	4,3	Novi di Modena (Mo)

Table 1: Seismic sequence of Italian Emilia-Romagna earthquakes (http://it.wikipedia.org/wiki/Terremoto_dell'Emilia_del_2012) [3].

The events listed in Table 1 are located along an E-W trending line 40 km north of Bologna and have similar reverse thrust type source mechanisms. This indicates that they may lie on the same active fault, the so-called Mirandola fault. This is a 'blind fault', as it is covered by sediments of the Po plain and thus not visible at the surface. It was formed as a result of the NS-convergence between the African and

Eurasian tectonic plates which occurs at a rate of roughly 1 cm / year. The rather complex interplay between these plates and the Adriatic microplate lead to flexuring of the crust and N-verging seismically active faults in the Southern Po plain. As a consequence, the Mirandola blind thrust was identified as a potential source for a M>5.5 earthquake, as it was verified in 2009 May.

In addition, the spatial distribution of damages induced by shallow shocks (6-10 km) was amplified locally by alluvial soils (seismic amplification phenomenon). In fact, the area affected by the earthquake is characterized by alluvial soils derived from river deposits left by the great hydrographical basins crossing the valley.

Typically, seismic waves produced by a high intensity earthquake, when encountering soft grounds, such as alluvial soils, tend to slow down their propagation speed. This process necessarily leads to a compensation energy effect, which expresses into a considerable wave amplitude increase, that is a greater ground acceleration gives rise to the so-called seismic amplification phenomenon. This implies a greater shaking of the ground, which can produce really significant damages to overlying buildings, even in the presence of a not particularly strong earthquake.

Some of the smaller towns close to the epicentres saw massive breakages. The earthquakes caused heavy damage to rural and industrial buildings, as well as to buildings and historical monuments, quantified as 12 billion and 202 million euro. Definitively, damage estimated was higher than one provoked by L'Aquila earthquake [4, 5]. Even the school buildings were hit by the fury of the earthquakes, with 223 buildings partially or severely damaged and about 71000 students involved. For this reason, in the present paper attention is directed to this type of buildings, for which a case study is herein presented with the purpose to design, starting from seismic assessment non-linear static analyses, effective retrofitting interventions.

2 Seismic behaviour of masonry school buildings

According to a research study conducted on the territory of Emilia-Romagna, the masonry school buildings located there have structural schemes and constructive details not in line with the provisions of the Italian new seismic code [6].

In the following, the results of some post-earthquake inspections conducted by the Author on school masonry buildings in the districts of Bologna, Ferrara and Modena after the Emilia earthquake are summarised and presented [7]. All examined schools showed similar seismic deficiencies, such as poor connections among walls and between walls and floors, but exhibited a good overall behaviour that prevented their collapse under earthquake.

In particular, the primary school "Maccaretolo", which was built between 1919 and 1945, has a structure with a rectangular plan of 500 square meters developed on three levels. The vertical structure is composed of two heads brick masonry walls surmounted by a wooden roof (Fig. 1a). The exterior of the building showed few damages mainly located on the main façade, where there are some horizontal cracks especially in the contact areas between the wall and the roof cornice (Fig. 1b). On the other hand, inside the building, especially on the first level, a significant crack pattern was detected, it being characterized by both a number of vertical cracks into walls and damages to some ceiling panels and supporting structures (Fig. 1c).

(a) (b) (c)

Figure 1: The primary school Maccaretolo: external view (a), damage on the façade cornice (b) and wall-to-floor and wall-to-wall cracks (c).

The primary school Menotti in Bomporto is a three structural units aggregate (Fig. 2a).

The oldest building was built before 1919. The detected damages were represented by the floor - wall detachment due to their poor connection (Fig. 2b), cracks in the vault key, horizontal cracks in the floors and vertical cracks among walls (Fig. 2c). The central structural unit of the building, built in the 80's, suffered light damages to non-structural parts, such as interior walls, ceilings and plasterboard. The last structural unit, built in the 90's and designed as the school dining hall, did not record any damage to structural and non-structural components.

(a) (b) (c)

Figure 2: The primary school Menotti: external view (a), wall-to-floor detachment (b) and a vertical crack between walls (c).

The primary school Battisti in Bondeno (district of Ferrara) is a very impressive building, built during the fascist epoch with a significant architectural value (Fig. 3). The vertical load-bearing structure consists of two heads masonry brick with thickness of about 28 cm. Minor structural damages, including some vertical cracks on the door lintels and horizontal cracks in the slab – wall intersection areas, were noticed in this building. In addition, significant damages to the ceilings and the collapse of some ceiling panels, were recorded.

(a) (b) (c)

Figure 3: The primary school Battisti: external view (a), wall-to-wall detachment (b) and vertical crack in a wall (c).

3 The school building under investigation

The unsatisfactory performances of buildings previously examined has pushed towards the necessity to inspect in detail the seismic behaviour of Emilia-Romagna schools through a typical case study of the local constructive tradition.

In particular, the economic-social importance of school buildings and, therefore, the need to evaluate their seismic performance in order to avoid failures under earthquakes, was already taken into account by the Author within the EU COST Action C26 “Urban Habitat Constructions under Catastrophic Events [8], which was concluded in 2010 with a final conference in Naples**.** In this framework, the Vesuvius area was selected as a pilot case study and Torre del Greco, one of the most populated town of that zone, was chosen for the seismic and volcanic vulnerability of masonry, r.c. and mixed masonry-r.c. schools [9, 10], whose investigation was carried out through both a quick survey activity performed from outside and detailed numerical investigations.

As in the case of the Vesuvius area, the seismic vulnerability exam of Emilia masonry schools was conducted with reference to a case study, namely the “Sorelle Luppi” school (Fig. 4), developing on two levels plus an attic with total height of 9.94 m from the foundation level (Fig. 5).

Figure 4: The main facades of the "Sorelle Luppi" school building.

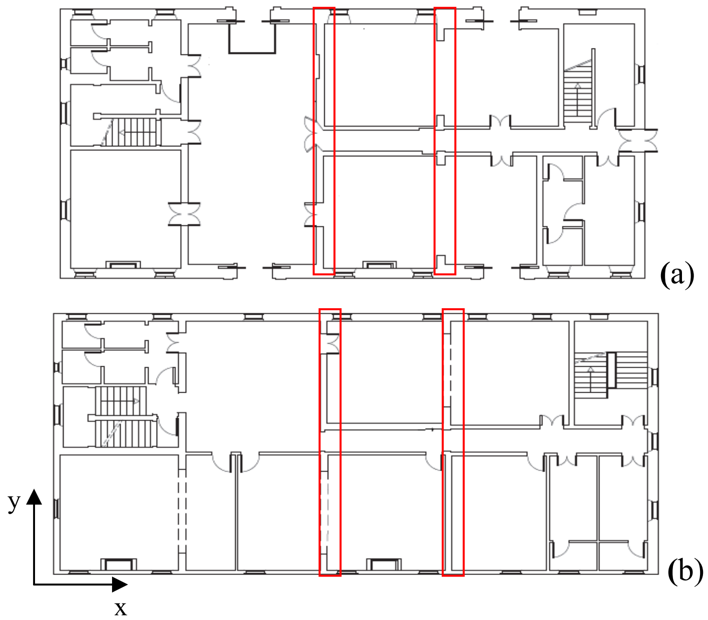


Figure 5: First (a) and second (b) level of the school building "Sorelle Luppi".

The selected building, built around the second half of the 19th century, is placed in a large park site in Solara, a fraction of Bomporto, located in the district of Modena. The structure is made of the typical two-head walls sustaining horizontal structures made of beams with deformable slab without tie beams and chains.

In principle, the building was the manor house of the Luppi family, who gave the name to the school still used today. In the early '70s the municipality of Bomporto bought its property aiming at transforming it into a school through restructuring works started in 1973. For the architectural merit, the school is currently covered by the bond of Superintendence for Cultural and Architectural Heritage. After earthquake, in the building few damages among walls and floors were detected, but vertical and diagonal cracks in masonry piers and spandrels (Fig. 6) and horizontal cracks in some vaults were noticed.

Figure 6: Details of some internal and external cracks.

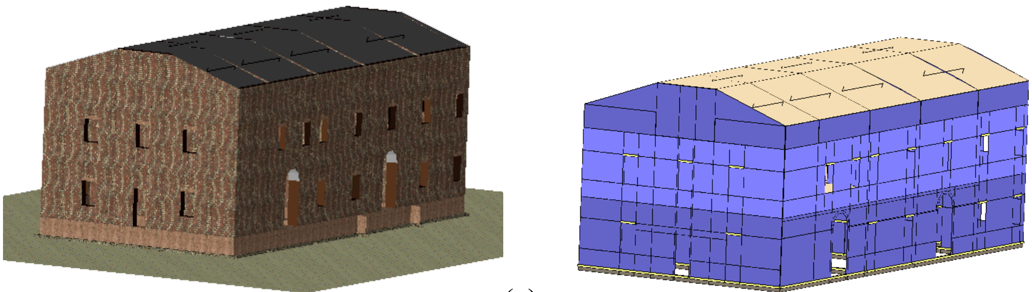
4 Non-linear numerical assessment and retrofitting analyses

In the following the seismic behaviour analysis of the examined case study is investigated through two calculation programs appositely conceived for masonry buildings, namely 3DMacro and 3Muri.

Static non-linear analyses have been performed with these two programs with the purpose to compare the achieved pushover curves each to other. As final outcome of the study, two building retrofitting interventions with either FRP strips or bidirectional steel strips have been designed and the resulting upgraded curves have been put in comparison.

The first step towards the seismic analysis of the structure with 3DMacro software (release 2.0) goes through the definition of its geometrical model (Fig. 7a). Thanks to the plan and wall editor, the building geometric model drawing can be created by means of a simple CAD, where the geometric characteristics of each wall are properly defined.

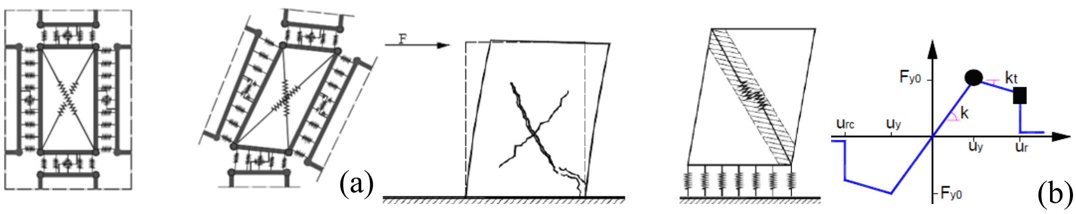
After both the material mechanical properties are assigned to all walls and loads are defined, the computational structural model on which analyses are carried out is automatically generated by the program (Fig.7b).



(a) (b)

Figure 7: The school geometrical (a) and FEM (b) models setup with 3DMacro.

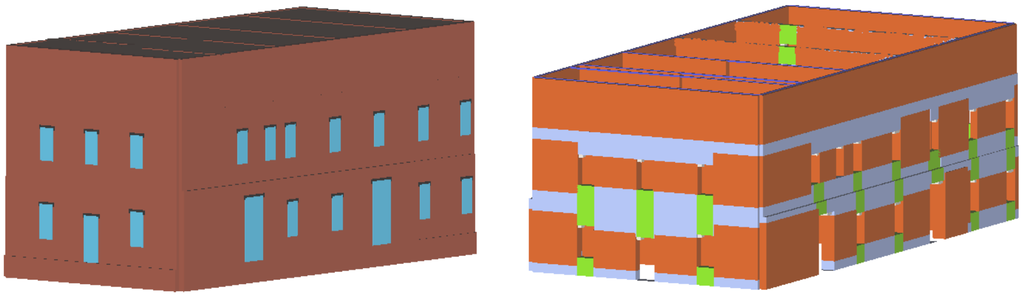
The 3DMacro calculation program models masonry piers and spandrels with diagonal springs representative of the tension and compression masonry behaviour (Fig. 8a). These elements, having either compression-bending behaviour or shear one (Fig. 8b), are connected one to another with distributed springs localized at their interface.



(a) (b)

Figure 8: 3DMacro modelling of masonry elements (a) and shear behaviour of masonry walls (b).

On the other hand, according to the 3Muri program (release 5.5.208), in the first analysis phase the structure geometrical model is defined (Fig. 9a) and, consequently, the mechanical properties are assigned to masonry elements. The computational model (Fig. 9b) is obtained by dividing the walls into macro-elements, which are representative of deformable masonry piers and spandrels and rigid joints not susceptible at damage under earthquake.



(a) (b)

Figure 9: The school geometrical (a) and FEM (b) models setup with 3Muri.

The 3Muri software allows for modelling of masonry walls through the equivalent frame technique, which schematizes masonry elements with one-dimensional type ones. In particular, the walls can be schematized as a frame, in which the resistant elements (piers and spandrel beams) are assembled with rigid nodes. The spandrel beams can be modelled only if they are adequately toothed by the walls, supported by structurally efficient architraves, and, if possible, exhibiting a strut resistant mechanism.

From the observation of masonry buildings damaged by seismic events, two different damage mechanisms, namely shear (diagonal and sliding) and compression-bending, emerge.

The shear mechanism is described by the Mohr-Coulomb model, which is able to collect the progressive degradation of the element resistance and rigidity, through the descriptive quantities of the damage. This law, due to its incremental formulation, is able to model hysteretic behaviour and, therefore, can be used for non-linear dynamic and cyclical pushover analyses. The ultimate shear deformation is based on the maximum drift value expected in the code, equal to 0.4%.

On the other hand, the compression-bending mechanism is rigorously examined, considering the effective redistribution of the compression due to both choking of the section and reaching of the maximum compression resistance. The last displacement associated with the compressive-bending mechanism is based on the maximum drift value expected for this mechanism, that is 0.6%.

The execution of seismic analysis involves two distinct phases. The first phase is a force control procedure, where the load vector, which is either proportional to masses or reverse triangular type, is increased proportionally up to the inability of the structure to support a further load increase. In the second phase, increases of both force and displacement are imposed to the structure in order to ensure equilibrium and evaluate its residual strength with increasing deformation level.

On the basis of the poor building knowledge, a limited knowledge factor with a confidence factor of 1.35 has been assumed for structural analysis, carried out considering a soil type C.

For the sake of example, the school building seismic response gotten by two examined analysis programs is plotted in Figure 10 with reference to the analysis in the direction *x* with positive eccentricity and forces proportional to masses.

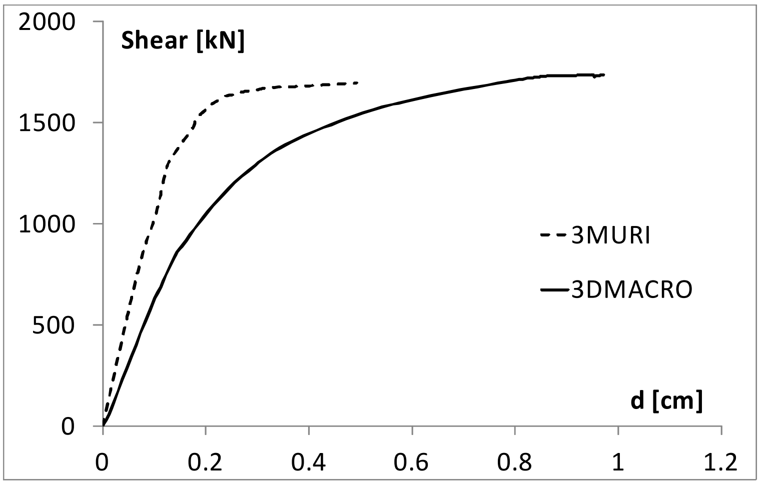


Figure 10: Comparison between building capacity curves in direction *x*.

From the comparison performed in the non-linear static field, it was found that:

- The two programs allow to attain the same maximum base shear value.
- In terms of ductility a difference due to the diverse ultimate deformation limit of masonry panels is noticed. In fact, 3Muri adopts code limitations for panels, neglecting their tensile behaviour, whereas 3DMacro uses an ultimate deformation limit depending on the elasto-plastic law of springs, providing a larger (about two times) ultimate displacement. However, even if this difference in terms of ultimate displacement between the programs has been noticed, they provide more or less the same ductility (equal to about 3).
- The building stiffness given by 3Muri is about twice greater than the 3DMacro one. This difference is due to the way to calculate the modal participation factor, achieved through a modal dynamic analysis by 3Muri and by means of a simplified relationship in relation to the number of floors by 3DMacro. In addition, 3Muri assumes as control node the one having as displacement the average value among displacements of the top storey nodes, while with 3DMacro the top-storey representative point can be chosen only among three points identified by the program itself. In this latter case, as a result, the top-storey barycentre displacements cannot be monitored.
- 3Muri provides more conservative results than 3DMacro ones.
- The school damage state achieved from 3DMacro, represented in Figure 11 together with that provided by the 3Muri software, is able to better predict the real crack pattern exhibited by the building facades under earthquake.

Afterwards, by using the 3Muri software, 24 pushover analyses on the study building have been performed by considering, as stated by the current Italian code, all the possible eccentricities between the barycentre and the stiffness centre, equal to 5% of the maximum building side perpendicular to the earthquake direction, and the usual two lateral force distributions, proportional to either masses or first vibration mode displacements.

In Figure 12 the worst pushover curves in directions x and y, where the maximum displacements required by the SLV earthquake are 11.8 mm and 11 mm, respectively, are plotted.

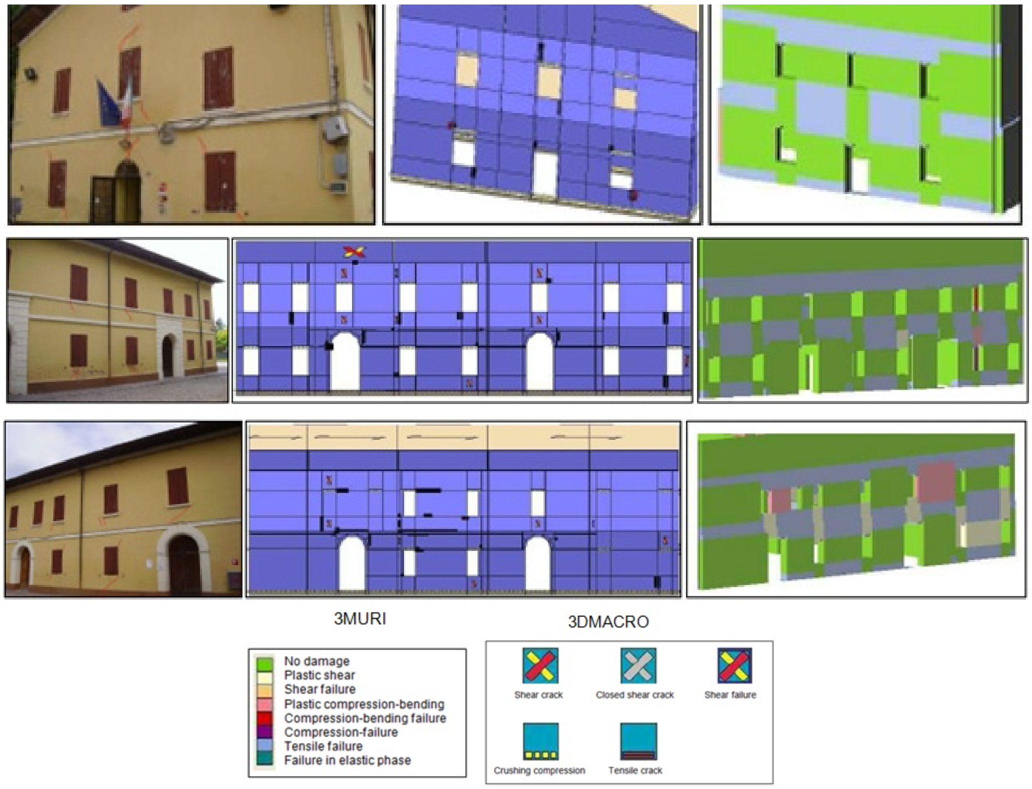


Figure 11: Comparison between building facades damage state after earthquake.

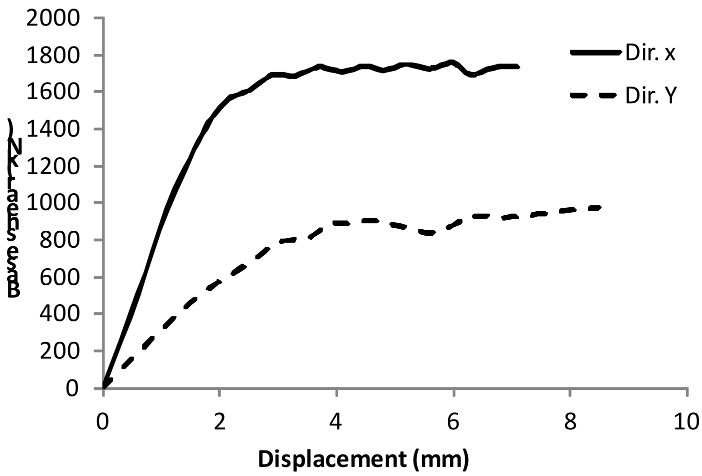
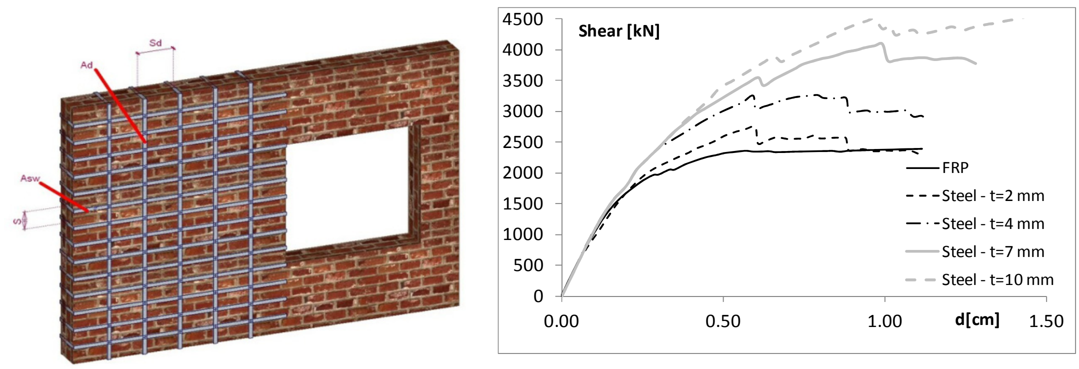


Figure 12: Pushover curves related to the worst analyses in directions x and y.

From analysis results, considering that demand displacements are greater than capacity ones, it is evident that the building needs structural consolidation interventions. Therefore, the two longitudinal walls and the two transverse ones surrounded with a red rectangle in Figure 5 have been reinforced at the first two levels with Fibre-Reinforced Polymer (FRP) materials under form of bidirectional strips (Fig. 13a), having thickness of 0.167 cm, width of 20 cm and pitch of 50 cm. The used fibre-reinforced materials have the following mechanical properties: E = 280000 MPa, f_{fd} = 4100 MPa and ε = 1.46 %.

The employment of these bidirectional strips has satisfied all seismic analyses, conferring to the building a better seismic performance in terms of strength and, especially, of ductility. Thereafter, in order to ensure the same invasiveness and, more or less, the same global response drawn with the use of composite materials, a further structural reinforcement intervention through bidirectional S275 steel strips, fixed to the walls by steel bars and having thickness of 0.2 cm, width of 7 cm and the same pitch of FRP strips, has been used. Subsequently, steel strip thickness has been changed from 2 mm to 10 mm in order to individuate, through a sensitivity analysis, the best thickness solution.

The comparison among retrofitting curves is shown in Figure 13b.



(a) (b)

Figure 13: Retrofitting intervention with bidirectional strips (a) and comparison among building capacity curves with different reinforcing interventions (b).

From the previous figure it is noticed that, even if a unique optimal solution does not appear, 7 mm thick strips represent the best intervention, they allowing to increase significantly the building base shear with respect to that achieved with FRP and with a little bit difference in terms of strength and lesser cost than 10 mm thick strips.

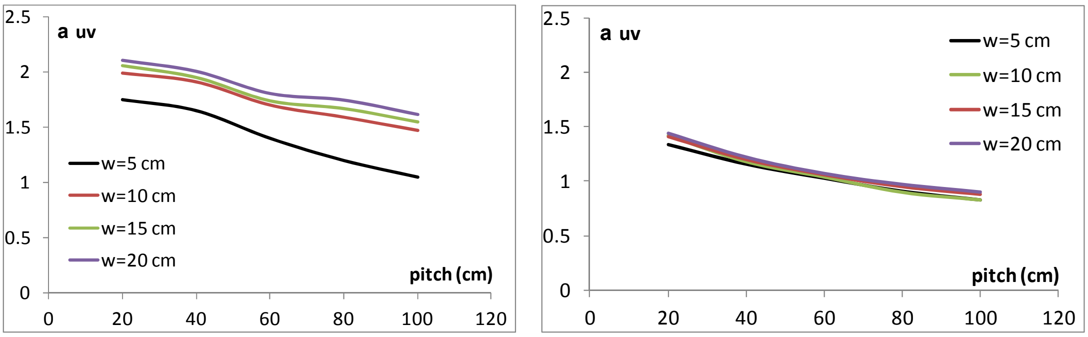
5 Parametric retrofitting analysis with FRP strips

In the final research phase, a parametric analysis on FRP strips with different geometries has been carried out with the purpose to individuate the best retrofitting solution.

In particular, the strip thickness (0.167 cm) has been considered as a fixed parameter, whereas four different strip widths *w* (5, 10, 15 and 20 cm) and five different pitches *p* among strips (20, 40, 60, 80 and 100 cm) have been considered as random parameters.

The combination of these variable geometrical factors has led towards 20 analyses, whose results have been compared each to other in terms of a risk index. In particular, this index, prescribed by the Italian OPCM 3362 code (2004) [11], is called α_{uv} and is considered as an indicator of the building collapse risk. It is calculated as the school capacity acceleration at the life safety limit state over the acceleration demand, that is the site PGA. Values of α_{uv} less than one are related to buildings that do not withstand the earthquake actions, whereas if they are greater or equal than one the building can be considered as totally retrofitted from seismic point of view.

In Figure 14 the seismic risk index has been calculated for different retrofitting solutions with FRP strips. First of all, from these pictures it is apparent that all solutions provide index greater than one in direction *x* only, whereas in direction *y* some solutions do not confer to the school the capacity to sustain the design seism. Moreover, it is noticeable that in direction *x* the strip width increase gives an increase of seismic performance, while in direction *y* all considered strips provide more or less the same seismic safety grade. This is due to the greater extension of the retrofitting intervention in direction *x* than in direction *y*. Therefore, the optimal strip width is equal to 10 cm in direction *x* and 5 cm in direction *y*, since these solutions allow to achieve the maximum benefit with the lesser cost.



(a) (b)

Figure 14: Seismic risk index vs. pitch among strips for the school retrofitted with different FRP strips in directions *x* (a) and *y* (b).

Furthermore, in Figure 15 the seismic risk index has been plotted versus the strip width in order to individuate the optimal strip pitch to be used in the two school building directions. From these pictures it appears that in directions *x* and *y* the minimum pitch able to attain a unitary seismic risk with the lesser cost is 100 cm and 60 cm, respectively.

In conclusion, the parametric analysis has provided the best retrofitting solutions, namely 10 cm wide strips with pitch of 100 cm and 5 cm wide strips with pitch of 60 cm in directions *x* and *y*, respectively.

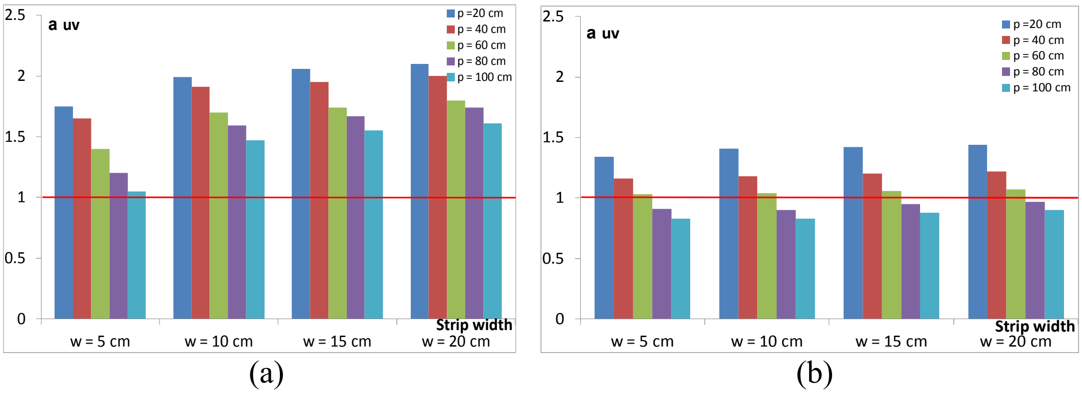


Figure 15: Seismic risk index vs. strip width for the school retrofitted with different FRP strips in directions *x* (a) and *y* (b).

6 Conclusions

After the 2012 May 20th and 29th Emilia-Romagna seismic events, the usability check of a masonry school building placed in Solara, a little district of Bomporto (neighbourhood of Modena), has been carried out in the current paper under the support of the ReLUIS Universities Consortium and the Italian Civil Protection Department.

Seismic analyses on the study building have been conducted by means of the 3Muri and 3DMacro programs. The achieved results have shown that both analysis programs provide a similar earthquake response of the examined building. More in detail, 3Muri gives the most conservative results in predicting the building pushover curves, whereas 3DMacro is able to better foresee the school damage pattern under earthquake.

The detected seismic deficiencies have led to design and apply to the building two different retrofitting solutions for masonry walls, namely jacketing with fibre composite materials and reinforcement with two-directions steel stripes. The comparison between the retrofitting building capacity curves has demonstrated that steel stripes allows to attain the same ductility and a strength level greater than that offered by composite fibre stripes. Moreover, a sensitivity analysis conducted on steel strips with different thicknesses has provided as best solution the one characterised by 7 mm thick strips. Therefore, also considering the lesser cost and the better sustainability performance, steel strips are the best technique for improving the seismic performance of the study building.

Finally, a parametric analysis on FRP strips with different width and spacing has been carried out with the purpose to individuate the best retrofitting solutions, represented by 10 (width) x 60 (pitch) cm and 5 (width) x 100 (pitch) cm strips, respectively, in the directions *x* and *y* of the building.

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