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Sustainable retrofit design of RC frames evaluated for different seismic demand

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Abstract. Seismic upgrading of existing structures is a technical and social issue aimed at risk reduction. Sustainable design is one of the most important challenges in any structural project. Nowadays, many retrofit strategies are feasible and several traditional and innovative options are available to engineers. Basically, the design strategy can lead to increase structural ductility, strength, or both of them, but also stiffness regulation and supplemental damping are possible strategies to reduce seismic vulnerability. Each design solution has different technical and economical performances. In this paper, four different design solutions are presented for the retrofit of an existing RC frame with poor concrete quality and inadequate reinforcement detailing. The considered solutions are based on FRP wrapping of the existing structural elements or alternatively on new RC shear walls introduction. This paper shows the comparison among the considered design strategies in order to select the suitable solution, which reaches the compromise between the obtained safety level and costs during the life-cycle of the building. Each solution is worked out by considering three different levels of seismic demand. The structural capacity of the considered retrofit solutions is assessed with nonlinear static analysis and the seismic performance is evaluated with the capacity spectrum method.

Keywords: sustainable strategies for engineering; minimum cost optimization; pushover analysis; capacity spectrum method; existing concrete buildings; seismic retrofit; FRP strengthening; ETS strengthening; shear walls

1. Introduction

Existing structures in Europe are often inadequate with respect to the seismic performance required by modern codes. The majority of them were designed without any earthquake resistance criterion, because they were built when codes required design for gravity loads only. For this reason, these structures have very low resistance to horizontal actions and they are extremely prone to fragile collapse, because capacity design is not satisfied. Seismic upgrading is necessary to obtain safer structures, especially for public buildings that are strategic for social purposes.

Nowadays, innovative technologies help structural designers to satisfy retrofit goals, for example, the use of Carbon Fiber Reinforcement Polymers (C-FRP) and the Embedded Through

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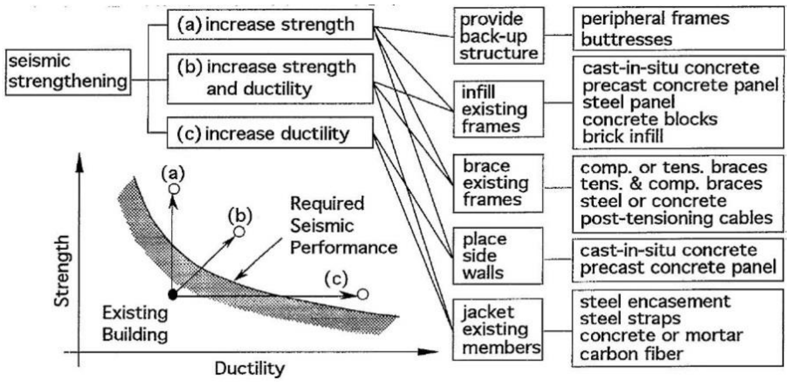


Fig. 1 Seismic strengthening techniques (Sugano 1996)

Section (ETS) techniques, along with traditional solution, such as new reinforced concrete (RC) shear walls. Different retrofit techniques correspond to different levels of safety, but also to different costs.

The aim of this study is the evaluation of the optimal and sustainable retrofit solution, by considering three levels of seismic demand: high, medium and low hazard Italian zones. The suitable solution is identified as a compromise between safety and costs, which include construction and repair costs during the life-cycle of the structure (Pelà *et al*. 2012).

The research of a sustainable approach to seismic retrofit accounting risk attitude, economical framework and safety related topics is a pressing social challenge (Gilmore 2012, Goda and Hong 2006a,b).

In recent years many authors have dealt with seismic rehabilitation techniques, following different approaches. In general, the design strategy can lead to an increase of structural ductility, strength, or both of them, as shown in Fig. 1 (Sugano 1996).

Baros and Dritsos (2008) have elaborated a simplified procedure to select a suitable retrofit strategy for existing RC buildings, by using three methods of rehabilitation: 1) FRP jackets to increase the overall structural ductility; 2) new shear walls to increase the structural strength; 3) RC jackets to increase both strength and ductility. Authors have shown that FRP and RC jackets can increase the ductility capacity up to 150-200% but shear walls are necessary in order to obtain higher capacity.

Caterino *et al*. (2008) have proposed a multi-criteria decision making method for seismic retrofit of RC structures, by using four upgrade strategies: 1) Glass-Fiber Reinforced Polymers (G-FRP) jackets to increase the structural ductility; 2) steel bracing to increase the structural strength; 3) RC jackets in selected columns to increase both strength and ductility; 4) base isolation with high-damping rubber bearing (HDRB) to reduce the transmission of seismic forces from the basement to the structure. Authors' conclusions have been that the use of G-FRP jackets is the best solution followed by RC jackets, base isolation and steel bracing.

Similarly, Mazza (2015) has investigated the optimal combination of different retrofitting techniques for existing low-rise RC frames, such as: 1) C-FRP wrapping of the columns ends; 2) Base-isolation of the building with HDRB bearings; 3) Hysteretic Bracing of the frame structure. Author suggests the application of local strengthening with C-FRP as auxiliary technique to global retrofitting techniques.

In this paper, an existing RC frame with poor concrete quality and inadequate reinforcement detailing is considered for seismic upgrading. The structure considered in this study is a three-story RC frame of a school built in the 60's in Forlì, Italy. This building is representative of common pre-seismic code constructions in Italy, such as the 2D frame investigated is representative of the structural system of the school.

Firstly, a preliminary retrofit is designed in order to remove the eventual vulnerability to gravity loads; then four different retrofit options are designed to perform different safety levels. Two of these options aim at enhancing the seismic structural capacity by improving the displacement capacity making use of columns confinement with C-FRP jackets, whereas the other options increase the strength capacity by adding new RC shear walls. The structural safety of each design option is assessed with the capacity spectrum analysis and the related sustainability is evaluated with a cost analysis by considering three different levels of seismic demand.

2. Description of un-retrofitted frame

The object of this study is a RC frame belonging to the structural system of an existing school building in Forlì, Italy. The structure was built from 1965 to 1971, when only gravity loads were considered for design. The front view and dimensions of the frame are presented in Fig. 2.

In situ investigations, material testing and design simulation of steel reinforcement detailing were performed in order to assess the seismic vulnerability of the school building. Detailed information are reported in Aprile (2010).

The considered frame has three stories and four spans. The inter-storey heights are of 2.73, 3.30, 2.99 m, respectively for ground, first and second floor. The spans are 3.05 m each. All of the stories have Reinforced Brick Concrete (RBC) slabs with negligible in-plain deformation. Storey floors are not matter of this study and more details about their assessment can be found in Aprile (2010).

Ground floor columns have rectangular cross section of 600×300 mm^2, first floor columns of 500×300 mm^2, second floor columns of 400×300 mm^2.The longitudinal steel reinforcement consists in 8 Ø12 mm bars, and stirrups are Ø6 bars 100 mm spaced. Ground floor beams have rectangular cross section of 600×250 mm^2, first floor beams of 500×500 mm^2, second floor beams of 400×260 mm^2. The longitudinal steel reinforcement consists in Ø12 mm bars, (5 at the top and 2 at the bottom side at the ends, 2 at the top and 5 at the bottom at mid span) and stirrups

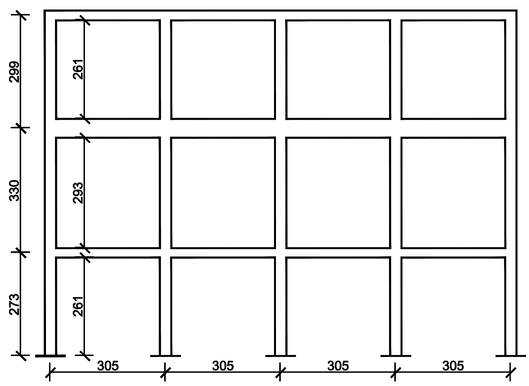


Fig. 2 View of the frame, dimensions in cm

are Ø 8 mm bars 200 mm spaced for ground and first floor; 4 Ø 20 mm bars and stirrups Ø 8 mm bars 50 mm spaced for second floor.

Mean concrete compressive strength is 10.38 MPa and mean steel yield strength is 359.15 MPa for all members, except the second floor beams that are rebuilt with the new roof floor in C25/30 concrete class (characteristic concrete compressive strength is 25 MPa) and with B450C steel reinforcement (characteristic steel yield strength is 450 MPa). The material properties were assessed with in situ and laboratory tests. In particular, 13 concrete cores were drilled from the structure and tested in compression; 24 sclerometric tests and 24 ultrasonic tests were performed on different structural members. The coefficient of variation of the concrete compressive strength in different part of the school was equal to 34%. The mean value of the concrete strength was considered for design by applying a confidence factor of 1.2. Concrete cover thickness is 3 cm for existing members and 5 cm for new members.

Structural loads are 6.1, 10.6 and 8.4 kN/m for ground, first and second floor beams respectively; non-structural loads are 18.1, 5.6 and 3.7 kN/m; live loads are 7.4 kN/m at ground and first floor beams; snow load is 3.0 kN/m for second floor beams only.

Preliminary retrofit is necessary to solve shear and flexural deficiencies of some beams to gravity loads. In particular, shear strength of ground floor and first floor beams is improved with ETS technique and flexural strength of ground floor beams is improved with C-FRP system.

ETS technique (Breveglieri *et al*. 2014, 2015, Barros and Dalfré 2013) consists in embedding steel bars in drilled holes glued with epoxy resin (Fig. 3). Two bars Ø 12 mm in 8.8 steel class (characteristic steel yield strength is 640 MPa) are placed between internal stirrups in order to increase the shear strength of the beam. ETS design is performed according to UNI EN 1992-1-1 (2004).

C-FRP reinforcement (Aprile *et al*. 2001, Aprile and Benedetti 2004, Aprile and Feo 2007) consists in carbon fiber reinforced polymers sheets glued with epoxy resin on the bottom side of the ground floor beams (Fig. 4). FRP sheets have effective cross section of 300×0.131 mm^2, ultimate tensile fiber strength of 4300 MPa, elastic modulus of 234 GPa and ultimate fiber deformation of 1.8%. C-FRP design is performed according to CNR-DT 200-R1 (2013). In the following, the un-retrofitted frame is called Alternative 0 (A0).

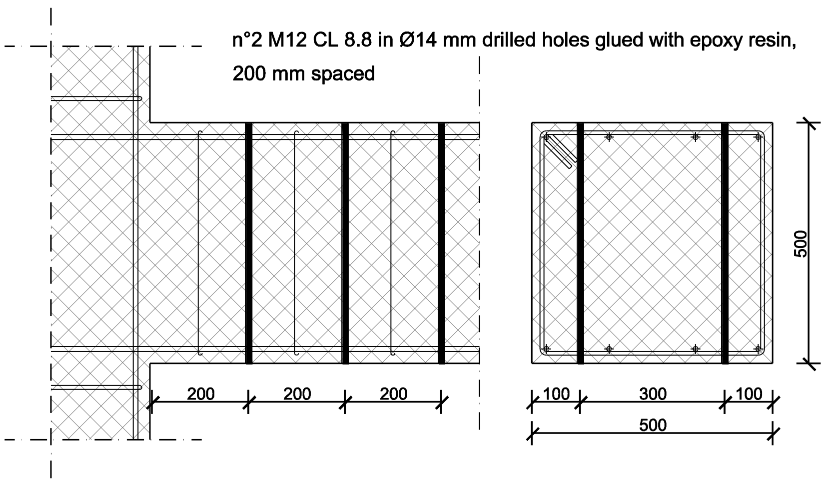


Fig. 3 ETS strengthening, dimensions in mm

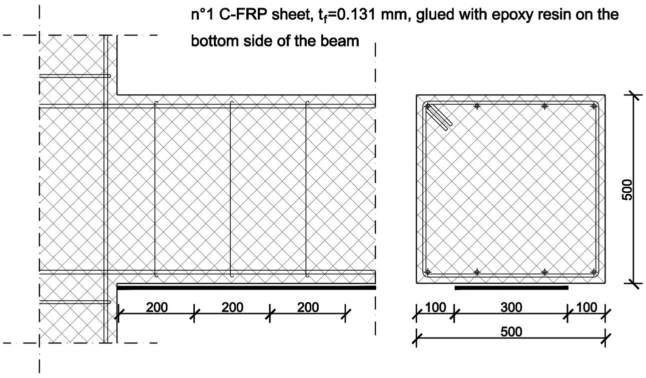


Fig. 4 C-FRP strengthening, dimensions in mm

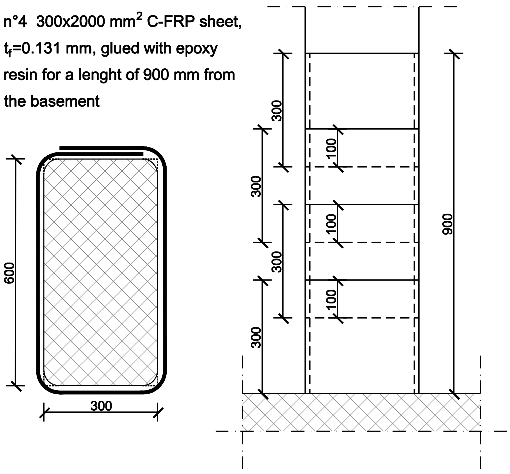


Fig. 5 C-FRP wrapping, dimensions in mm

3. Description of retrofit solutions

In this paper two kind of retrofit strategy are considered: improving the structural ductility with confinement and improving the structural strength and stiffness with shear walls. In both cases, two solutions are designed by considering different levels of reinforcement (light reinforcement and heavy reinforcement). These options give the possibility to assess the sustainable solution.

The first solution consists in wrapping the bottom end of ground floor columns with one sheet of C-FRP (Perrone *et al*. 2009) for a length of 900 mm from the basement (Fig. 5). In the following, this retrofit solution is called Alternative 1 (A1).

The second solution consists in wrapping both the ends of ground floor columns with three sheets of C-FRP for a length of 900 mm and both the ends of first floor columns with one sheet of C-FRP for a length of 900 mm. In the following, this retrofit solution is called Alternative 2 (A2).

The third solution consists in adding two new RC shear walls at the external sides of the frame for all of its height. This retrofit solution simulates the RC frame retrofit by converting bays into RC walls as presented by Fardis *et al*. (2013). Walls have cross section of 750×300 mm^2; they are built with C25/30 concrete class (characteristic concrete compressive strength is 25 MPa) and with

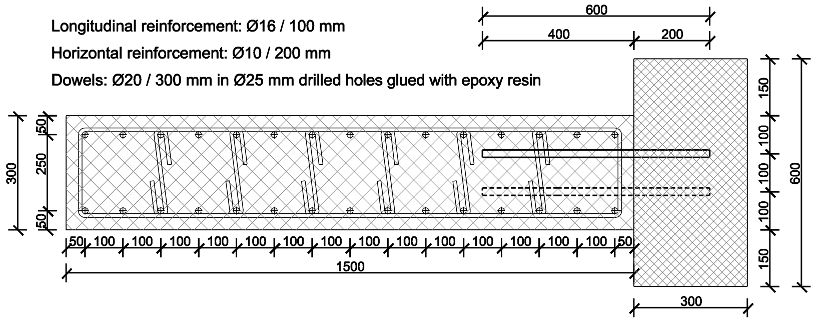


Fig. 6 A4 Walls’ horizontal section, dimensions in mm

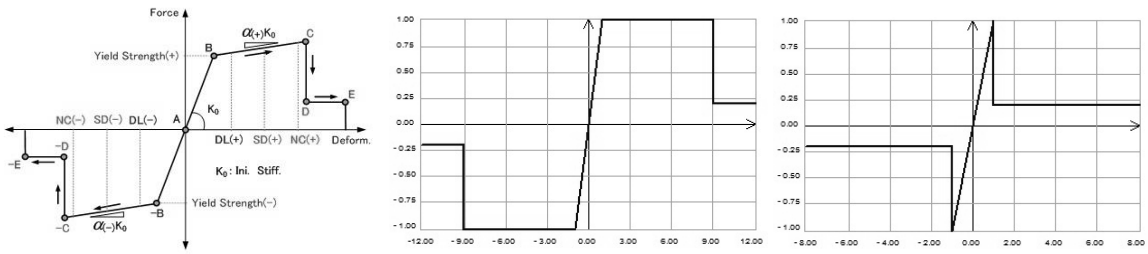
B450C steel reinforcement (characteristic steel yield strength is 450 MPa). Longitudinal reinforcement is composed by Ø 16 mm spaced 100 mm and horizontal reinforcement by Ø 10 mm spaced 200 mm on both sides of the wall. Walls are anchored to external columns with Ø 20 mm dowels spaced 300 mm on both sides. Concrete cover thickness is 5 cm. In the following, this retrofit solution is called Alternative 3 (A3).

The fourth solution consists in adding two new concrete shear walls at the external sides of the frame for all of its height. Walls have cross section of 1500×300 mm^2. Material quality and steel reinforcement detailing are the same of A3. In the following, this retrofit solution is called Alternative 4 (A4). Walls’ detailing is illustrated in Fig. 6.

Shear walls need new foundations with micropiles to transfer seismic actions to soil. Micropiles are Tubfix Ø 250 mm with a length of 14 m; the diameter of reinforcement is Ø 139.7 mm, the thickness is 10 mm and the steel is S355. Foundation design presentation is not a topic of this work but detailed information can be found in Zerbin (2013).

4. Method of analysis and seismic scenarios

A finite element model is implemented using a structural analysis software (Midas Gen v.2.1, 2014); beam and wall type elements are used. In order to assess the frame seismic capacity a lumped plasticity model was adopted, introducing at both ends of each element the nonlinear moment-rotation plastic hinge as suggested in UNI EN 1998-1 (2013); moment-curvature diagram



(a) Generic hinge (b) Moment-rotation hinge (c) Shear hinge

Fig. 7 Nonlinear hinges

is automatically calculated by the software and converted into moment-rotation diagram for each hinge. At the same time, shear hinges are located at beam ends in order to check fragile collapse. Generic nonlinear hinge model implemented in Midas Gen is illustrated in Fig. 7(a); moment-rotation hinge model and shear hinge model used in the analysis are shown in Fig. 7(b)-(c).

A nonlinear static analysis (pushover) is performed using a lateral force pattern proportional to the product of floor masses and the first mode displacements along the in-plane axis (Chopra and Goel 2002). The capacity curve is obtained recording the frame base shear at each displacement incremental step of the control point, which is set at the roof centre of mass. The seismic assessment is carried out comparing the seismic capacity and the demand applying the N2 method, the capacity spectrum method proposed by Fajfar (2000) and Rozman and Fajfar (2009).

Two seismic assessment are performed for each retrofit solution respectively for the life-safety limit state (SLV), with a probability of exceedance of 10% in 50 years, and a return period of 712 years, and the damage-limitation limit state (SLD), with a probability of exceedance of 63% in 50 years, and a return period of 75 years. Life-time is equal to 50 years for existing buildings, which is multiplied by the importance factor equal to 1.5 for school buildings. SLV assessment shows the capacity of the structure during strong earthquakes; SLD assessment shows the damage during the life-cycle of the frame. Furthermore, the Peak Ground Acceleration (PGA) of the maximum demand spectrum that identifies the capacity of the frame is found.

All structural elements are checked for ductile and fragile failure modes. Ductile failure mode is the rotation of plastic hinges at the end of the element; fragile failure mode is the shear rupture of the element. Rotational capacity is defined as 75% of ultimate chord rotation. Ultimate and yielding chord rotation of plastic hinges are evaluated with the equations formulated by Biskinis and Fardis (2009, 2010a,b, 2013) and Panagiotakos and Fardis (2001); more information can be found in Monti and Nisticò (2008), Lam and Teng (2003), Spoelstra and Monti (1999). According to UNI EN 1998-3 (2005), checking ductile failure modes, medium material properties are divided by the confidence factor (CF), while checking fragile modes, material properties are divided by CF and material safety factor, i.e., 1.50 and 1.15 for concrete and steel reinforcement respectively. Tensile and compression strength of joints between beams and columns are checked according to Circ. 2/2/2009, n. 617 (2009), information about this method can be found in Masi *et al.* (2009).

Scope of the present analysis is the evaluation of the maximum earthquake which is tolerable by the structure in SLV for each retrofit configuration, and the level of damage that can occur during SLD earthquakes, in order to assess the design solution that minimizes the total cost. The total cost includes retrofit installation costs and SLD damage repair costs.

Economic evaluations consider only structural retrofit without interior/exterior finishes and technical plants. If the frame collapses under SLD actions the demolition and reconstruction costs are considered. If the frame can resist to SLD actions, then repair costs are considered according to structural element damage (plastic hinges) and non-structural element damage due to interstorey drift. Non-structural masonry wall interstorey drift limit is assumed equal to 0.50% (UNI EN 1998-1 2013); more information about drift limit and damage can be found in Colangelo (2013). Full details about the retrofit costs are reported in Zerbin (2013). It is important to remark that cost evaluation is performed taking into account the whole school structure and the considered frame retrofit cost is drown from the total costs.

The analysis is performed for three seismic hazard scenarios, defined according to the Italian code (DM 14/1/2008).

Scenario 1 (S1) represents the case of structure placed in Italian high hazard seismic zone, characterized by a PGA higher than 0.30 g; in this case study the chosen location is Reggio

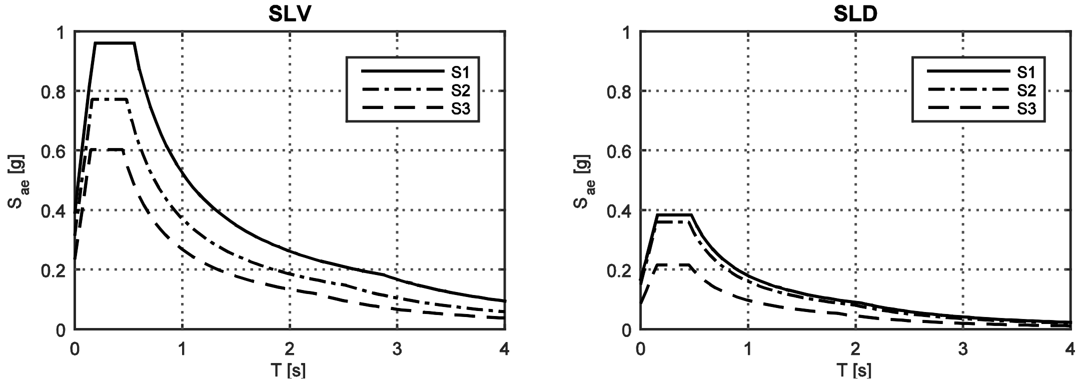


Fig. 8 Elastic spectra in SLV and SLD

Table 1 Elastic spectra parameters

Spectra	a_g [g]	F_o	T_C* [s]
SLV S1	0.319	2.443	0.377
SLV S2	0.232	2.446	0.312
SLV S3	0.162	2.567	0.276
SLD S1	0.112	2.284	0.311
SLD S2	0.100	2.395	0.279
SLD S3	0.058	2.485	0.281

Calabria.

Scenario 2 (S2) represents the case of structure placed in Italian medium hazard seismic zone, characterized by a PGA in the range of 0.20-0.30 g; in this case study the chosen location is Forlì.

Scenario 3 (S3) represents the case of structure placed in Italian low hazard seismic zone, characterized by a PGA lower than 0.20 g; in this case study the chosen location is Ferrara.

All scenarios have the same soil type C with a mean shear wave speed between 180-360 m/s in the first 30 m of soil depth. SLV and SLD elastic spectra are reported in Fig. 8, spectral parameters are reported in Table 1. a_g, F_o and T_c^* are respectively the horizontal peak ground acceleration, the maximum value of amplification factor and the starting period of constant velocity range of the horizontal spectrum.

5. Results

In this section, the results of pushover and economic analysis are described. The pushover curves for un-retrofitted (A0) and retrofit alternatives (A1, A2, A3, A4) are plotted in Fig. 9. The figure highlights that A1 and A2 alternatives, based on C-FRP wrapping, give ductility to the frame but resistance and stiffness are unchanged. Otherwise, A3 and A4 alternatives, based on the addition of new shear walls, give significant strength and stiffness increase, but also ductility increase, as lower as greater is the strength increase.

Economic analysis is focused on the evaluation of retrofit and repair costs. Retrofit costs account initial costs to realise the above mentioned design alternatives. Repair costs account the future costs to repair the damage eventually due to the SLD earthquake or the demolition and

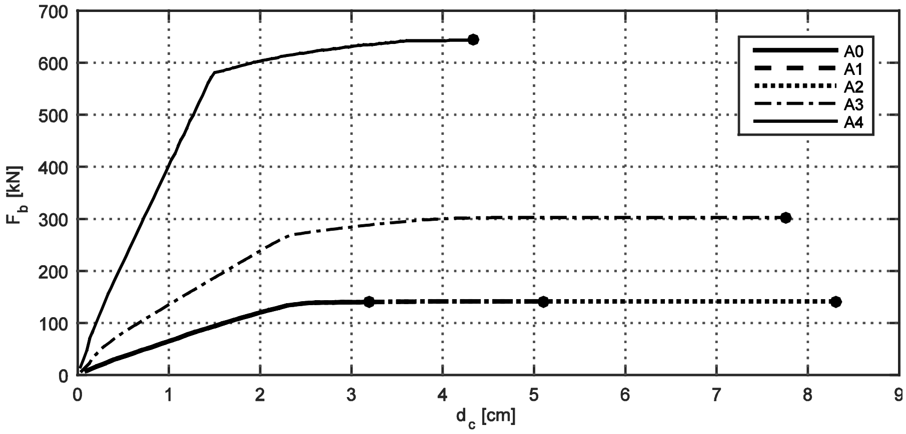


Fig. 9 Pushover curves for all alternatives

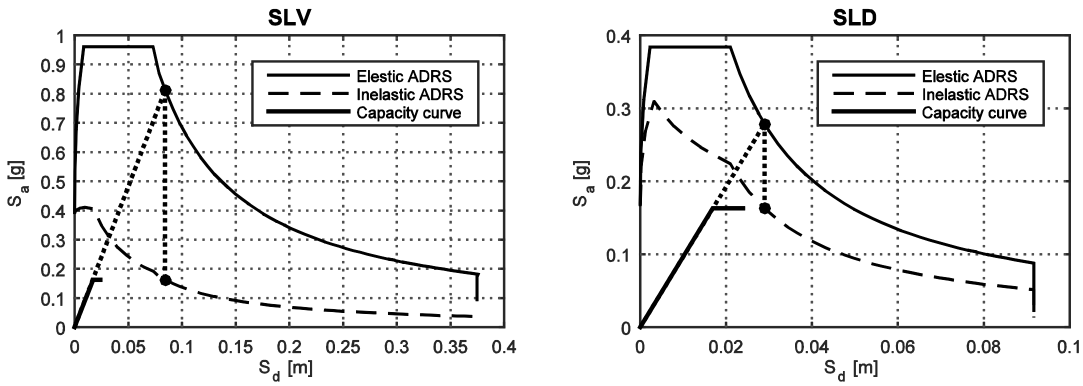


Fig. 10 SLV and SLD pushover and ADRS results of A0-S1

Table 2 SLV and SLD pushover results of A0 in S1, S2, S3

Scenario	SLV			SLD				Costs	
	γ_d	α^*	T_R [years]	γ_d	1st drift	2nd drift	3rd drift	Retrofit [€]	Repair [€]
S1	0.28	0.23	37	0.83	0.42%	0.49%	0.14%	0	37038
S2	0.40	0.37	54	0.93	0.42%	0.49%	0.14%	0	37038
S3	0.56	0.54	177	1.50	0.26%	0.32%	0.11%	0	0

reconstruction if structural collapse occurs. In the following, only the most relevant figures are presented for sake of brevity, and cases which are similar to those already shown are omitted.

In Table 2-Table 6, main results of pushover analysis are resumed for each retrofit alternatives and for all scenarios. Tables show the following significant parameters: displacement capacity-demand ratio (γ_d) for SLV and SLD; PGA capacity-demand ratio (α^*) for SLV, values for SLD are omitted for sake of brevity; earthquake return period (T_R) whose demand equals the capacity for SLV; interstorey drift ratio of each storey; retrofit and repair costs. The values of capacity terms in γ_d and α^* are derived scaling the earthquake demands in pushover analyses: the value of α^* is found when γ_d is equal to one; so α^* equal to one represents the design spectrum or demand.

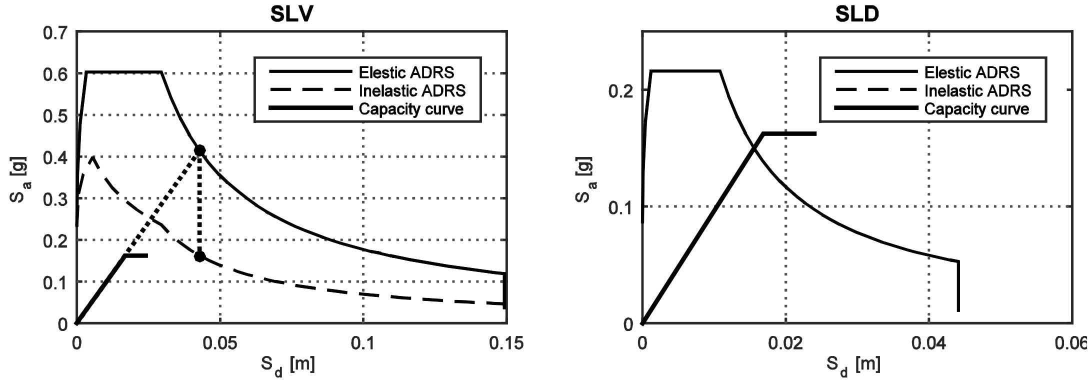


Fig. 11 SLV and SLD pushover and ADRS results of A0-S3

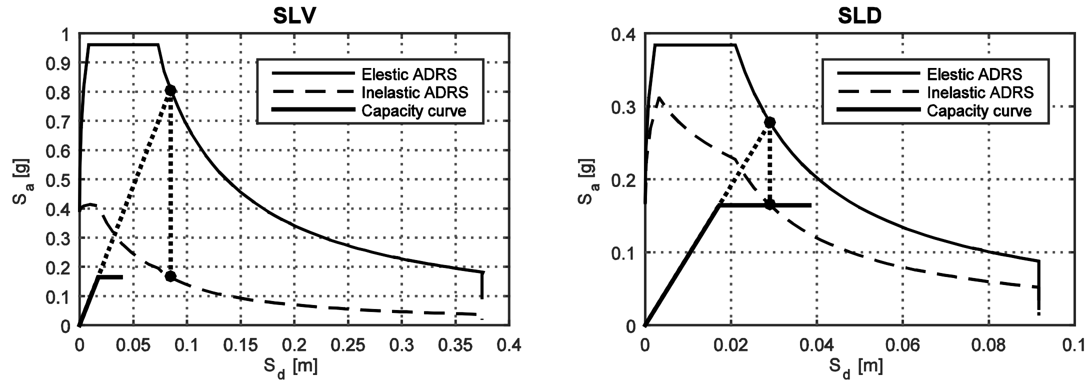


Fig. 12 SLV and SLD pushover and ADRS results of A1-S1

Table 3 SLV and SLD pushover results of A1 in S1, S2, S3

Scenario	SLV			SLD				Costs	
	γ_d	α^*	T_R [years]	γ_d	1st drift	2nd drift	3rd drift	Retrofit [€]	Repair [€]
S1	0.45	0.37	83	1.31	0.56%	0.61%	0.15%	4129	5877
S2	0.64	0.58	154	1.48	0.47%	0.53%	0.14%	4129	5877
S3	0.88	0.86	501	1.50	0.26%	0.32%	0.11%	4129	0

A0 (un-retrofitted frame) shows low strength and non-ductile behaviour; A0 results are summarized in Table 2. In S1 and S2 the seismic demand is bigger than the structural capacity and the frame carries 23% of SLV PGA (Fig. 10) and 37% respectively. In S3-SLV the seismic demand is bigger than the structural capacity and the frame carries 54% of SLV PGA, while in S3-SLD the seismic demand is lower than the structural capacity (Fig. 11). In S1 and S2, first and second floor drifts are close to drift limit of 0.50% given by UNI EN 1998-1 (2013). Retrofit costs are null in all scenarios; repair costs consist in demolition and reconstruction in S1 and S2, while repair costs are null in S3.

A1 (light C-FRP wrapping) shows low strength and ductile behaviour; A1 results are summarized in Table 3. In SLV the seismic demand is bigger than the structural capacity in all scenarios (Fig. 12) and the frame carries 37%, 58%, 86% of SLV PGA respectively. In SLD the seismic demand is lower than the structural capacity. In S1-SLD and S2-SLD, first and second floor drifts exceed the drift limit of 0.50% given by UNI EN 1998-1 (2013). In this case, repair costs are comparable with retrofit costs in S1 and S2, while repair costs are null in S3.

A2 (heavy C-FRP wrapping) shows low strength and high ductility; A2 results are summarized in Table 4. In S1-SLV the seismic demand is bigger than the structural capacity (Fig. 13) and the frame carries 65% of SLV PGA. Instead in S2-SLV and S3-SLV, the seismic demand is lower than the structural capacity (Fig. 14). In SLD the seismic demand is lower than the structural capacity. In S1-SLD and S2-SLD, first and second floor drifts exceed the drift limit of 0.50% given by UNI EN 1998-1 (2013). In this case, repair costs are comparable with retrofit costs in S1 and S2, while repair costs are null in S3.

Table 4 SLV and SLD pushover results of A2 in S1, S2, S3

Scenario	SLV			SLD				Costs	
	γ_d	α^*	T_R [years]	γ_d	1st drift	2nd drift	3rd drift	Retrofit [€]	Repair [€]
S1	0.73	0.65	264	2.14	0.56%	0.61%	0.15%	6330	5402
S2	1.04	1.05	823	2.40	0.47%	0.53%	0.14%	6330	5402
S3	1.44	1.61	2374	3.85	0.26%	0.32%	0.11%	6330	0

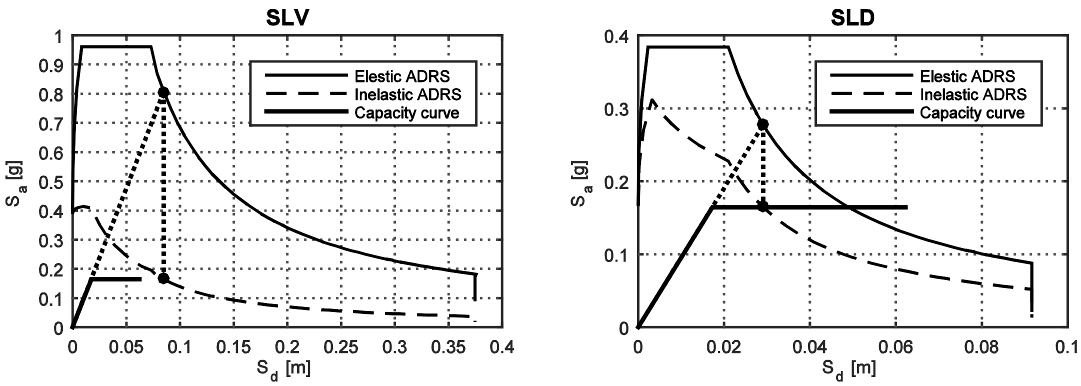


Fig. 13 SLV and SLD pushover and ADRS results of A2-S1

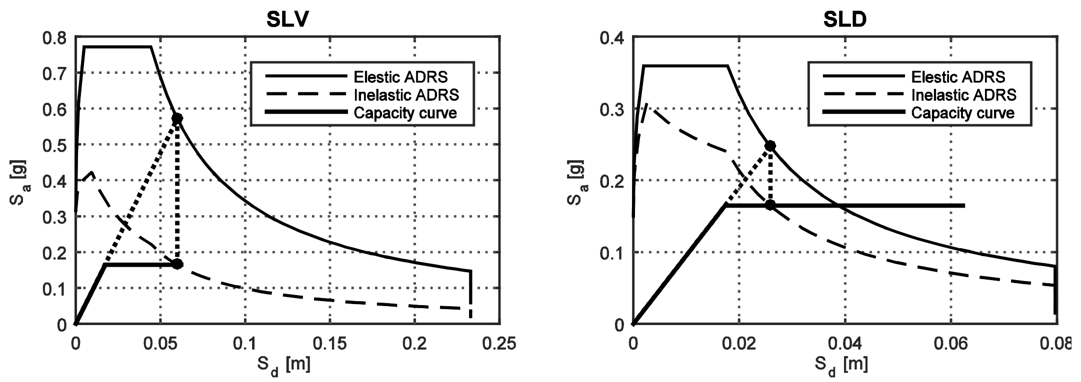


Fig. 14 SLV and SLD pushover and ADRS results of A2-S2

Table 5 SLV and SLD pushover results of A3 in S1, S2, S3

Scenario	SLV			SLD				Costs	
	γ_d	α^*	T_R [years]	γ_d	1st drift	2nd drift	3rd drift	Retrofit [€]	Repair [€]
S1	1.03	1.05	794	2.96	0.20%	0.35%	0.34%	15657	5096
S2	1.41	1.67	2475	3.16	0.18%	0.33%	0.31%	15657	5096
S3	1.88	2.59	2475	5.27	0.10%	0.19%	0.18%	15657	0

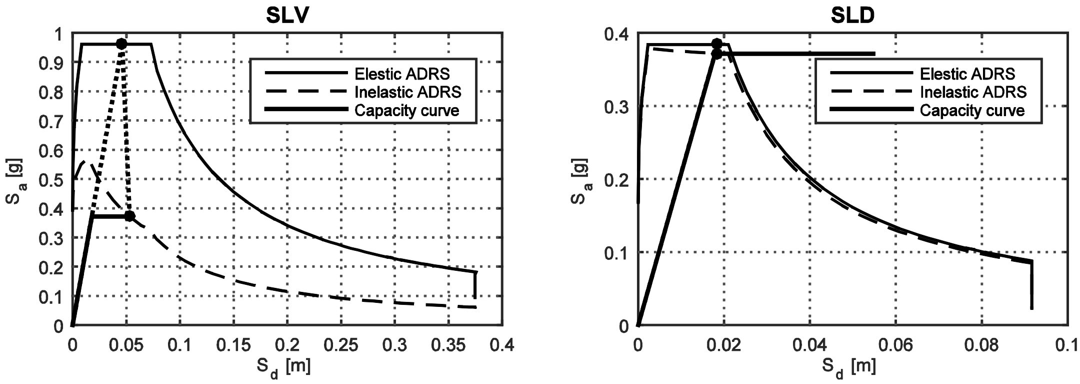


Fig. 15 SLV and SLD pushover and ADRS results of A3-S1

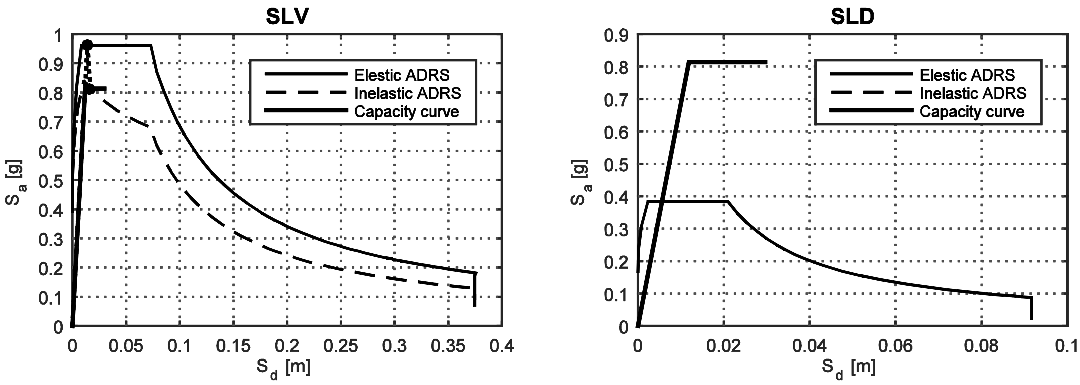


Fig. 16 SLV and SLD pushover and ADRS results of A4-S1

Table 6 SLV and SLD pushover results of A4 in S1, S2, S3

Scenario	SLV			SLD				Costs	
	γ_d	α^*	T_R [years]	γ_d	1st drift	2nd drift	3rd drift	Retrofit [€]	Repair [€]
S1	2.63	2.51	2475	5.29	0.04%	0.10%	0.11%	28118	1443
S2	2.63	2.51	2475	5.65	0.03%	0.07%	0.08%	28118	1443
S3	3.37	3.56	2475	9.40	0.02%	0.05%	0.06%	28118	1206

A3 (thin RC shear walls) shows high strength and high ductility; A3 results are summarized in Table 5. In SLV and SLD the seismic demand is always lower than the structural capacity (Fig.

15) Drifts never exceed the drift limit of 0.50% given by UNI EN 1998-1 (2013). In this case, retrofit costs are more than five times higher than repair costs in S1 and S2, while repair costs are null in S3.

A4 (large RC shear walls) shows very high strength and ductile behaviour; A4 results are summarized in Table 6. In SLV and SLD the seismic demand is always lower than the structural capacity (Fig. 16) Drifts are very small, around 20% of the drift limit of 0.50% given by UNI EN 1998-1 (2013). In this case, retrofit costs are about twenty times higher than repair costs for all scenarios.

6. Discussion

In Fig. 17-Fig. 19, the correlation between costs and PGA capacity-demand ratio (α*) are reported for each scenario, where α* expresses the safety level of the structure. It is clear that retrofit costs increase and the repair costs decrease with safety level increase. From the economical point of view, the sustainable retrofit solution can be defined by the total costs minimum value and depends on the considered scenario. From the safety point of view, the optimal retrofit solution can be defined by the closest value of α* to unit and also depends on the considered scenario.

In S1 (high seismic hazard) sustainable retrofit strategy leads to structural capacity in the range 40-65% of PGA seismic demand, and the optimal retrofit strategy leads to structural capacity carrying 105% of PGA seismic demand. This means that alternatives A1 and A2, based on C-FRP wrapping, are sustainable solutions and A3, based on small RC shear walls, is the optimal solution (Fig. 17).

In S2 (medium seismic hazard) sustainable retrofit strategy leads to structural capacity in the range 60-105% of PGA seismic demand, and the optimal retrofit strategy leads to structural capacity carrying 105% of PGA seismic demand. This means that alternatives A1 and A2, based on C-FRP wrapping, are sustainable solutions and A2 is also the optimal solution (Fig. 18).

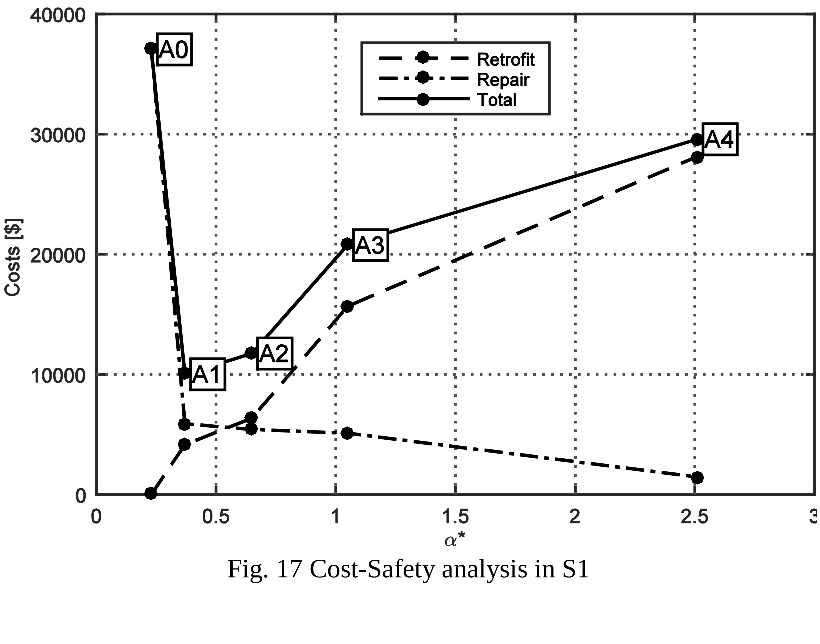


Fig. 17 Cost-Safety analysis in S1

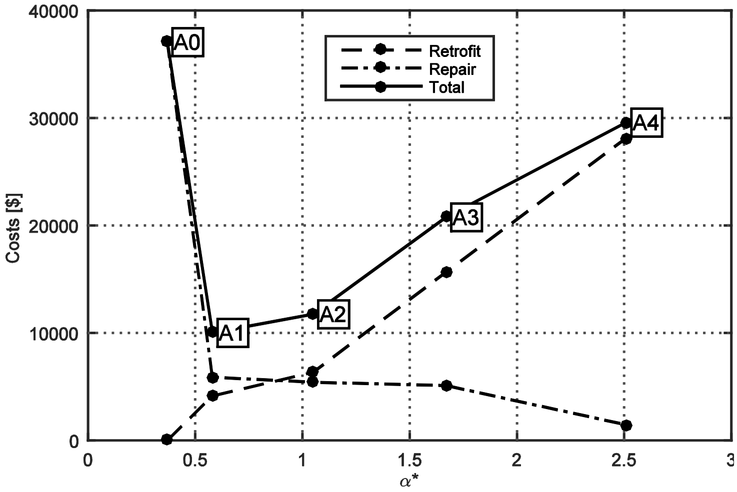


Fig. 18 Cost-Safety analysis in S2

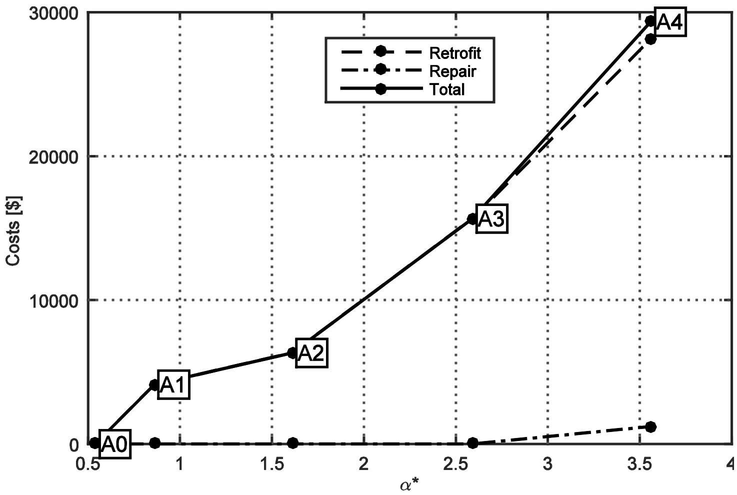


Fig. 19 Cost-Safety analysis in S3

In S3 (low seismic hazard) sustainable retrofit strategy leads to structural capacity of 50% of PGA seismic demand, and the optimal retrofit strategy leads to structural capacity in the range 86-161% of PGA seismic demand. This means that alternatives A0, i.e., the un-retrofitted structure, is the sustainable solution and A1 and A2, based on C-FRP wrapping, are the optimal solutions (Fig. 19).

7. Conclusions

A comparison among four different retrofit strategies has been developed for a RC frame belonging to a 40 years old school building. The original structure has shown poor materials and poor seismic detailing. The N2 capacity spectrum method has been used to assess the structural capacity reached with the retrofit strategies for three different seismic demand levels. An approximate but significative economical evaluation of retrofit and repair costs has been performed for each structural solution and seismic level. Results have been presented and discussed.

The following conclusions can be drawn:

• C-FRP wrapping of columns gives ductility to the frame. It makes the structure adequate to low and medium seismic demand and can increase the structural capacity up to 3 times of the original frame capacity. FRP does not increase the stiffness of the frame, so infilled masonry frame can be heavily damaged during SLD earthquakes. This is the cheapest considered retrofit solution but it may require repair works during the life-cycles of the structure.

• New RC shear walls give strength to the frame. They make the structure adequate even to high seismic demand and can increase the structural capacity up to 10 times of the original frame capacity. Story drifts are very small, so infilled masonry frame are not damaged during SLD earthquakes. This is the most expensive solution mainly due to the need of heavy foundation works.

• The sustainable retrofit strategy is the FRP wrapping for medium and high seismic demand, the un-retrofitted solution for low seismic demand.

• The optimal retrofit strategy is the FRP wrapping for low and medium seismic demand, the RC walls solution for high seismic demand.

• FRP wrapping is in general the best solution to retrofit existing RC buildings because it is low cost and not invasive; it is always sustainable and optimal for the majority of existing RC framed building.

This study should be extended to different structural typologies and to different construction materials in order to promote the development of suitable strategies for rehabilitation of existing structures.

Further insights are necessary to make engineers more awareness of available technical solutions according to sustainable strategy to reduce seismic vulnerability and risk, but mostly to address public and private asset managers towards a sustainable approach of intervention.

Within the frame of the economic recession of the western countries, the choice of a sustainable strategy is preferable to the choice of an optimal strategy in order to raise the average safety level of a wider number of buildings with a limited economical effort.

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