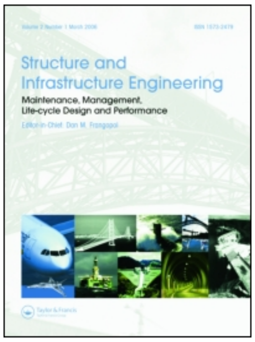

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Optimal seismic upgrading of a reinforced concrete school building with metal-based devices using an efficient multi-criteria decision-making method

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ABSTRACT
In the paper, the seismic retrofitting of an existing reinforced concrete school building located in the district of Naples has been examined. The school, which was designed to sustain gravity load only, is composed of seven constructions separated with seismic joints. One of these constructions has been retrofitted with different intervention techniques, namely reinforced concrete walls, steel concentric, eccentric and buckling restrained braces and steel shear panels, whose non-linear behaviour under seismic actions in terms of performance point detection have been evaluated and compared using Capacity Spectrum Method. Finally, the choice of the best intervention technique from economic, structural and environmental point of view has been done utilising an efficient multi-criteria decision-making (MCDM) method, the so-called TOPSIS method. From the performed analyses it was found that buckling restrained braces provide optimal solution for the seismic upgrading of the examined reinforced concrete school building.

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1. Introduction

Italy is characterised by a medium-high seismic hazard compared to other Mediterranean countries, but its buildings have a very high seismic vulnerability. In fact, the age and the typological – structural characteristics of these buildings are the crucial factors, which often lead to their failure under earthquakes. The disastrous effects of the earthquakes occurred in Italy during the past few years do not depend on their significant seismic intensity only, but also on both the high population density in many areas and the poor attention paid to build seismic-resistant buildings. In particular, more than 30% of the reinforced concrete buildings in Italy are inadequate to withstand design seismic loads prescribed by current Italian regulations. Hence, the mitigation of seismic risk and the study of techniques and strategies for seismic upgrading of existing reinforced concrete (RC) structures are important topics, both from scientific and social point of view.

RC buildings designed with past regulations were usually far from the correct interpretation of their dynamic behaviour, often exhibited a poor performance, characterised by sudden fragile collapses and, therefore, by a low dissipation capability. The poor construction details, together with the absence of any resistance hierarchy rules, have led towards local collapse mechanisms, which are known to be very dangerous. Generally, the principal damages occurring in RC buildings under earthquakes are soft-storey mechanisms, failure of columns, stiffening effect due to infill walls (localised collapse caused by the equivalent strut formed by infill walls), collapse of beam-to-column joints, collapse due to hammering with other constructions and failure of foundations.

The retrofitting is the necessary step for improving the safety of existing buildings. These interventions aim at enhancing building's capacity to become bigger than the demand induced by a severe earthquake. The upgrading strategy has the target to choose the optimal intervention to improve structural seismic behaviour. Generally, using the most common strategies, it is possible to get an improved performance via three possible ways: increasing deformation capacity, increasing the resistance or increasing both. These targets can be accomplished using different technical solutions. The most common seismic mitigation strategies are:

- Composite Materials (Fibre Reinforced Polymers): ensure high tensile strength (for carbon fibres $f_{tk} \geq 3.000$ MPa), high elastic modulus (160–300 GPa), low specific weight and a negligible mass increase.
- Reinforced concrete jacketing: it is generally applied to columns and/or walls in order to increase both the strength and the strain capacity, as well as to improve the beam-column joints.
- Steel jacketing using steel profiles, whose effects on the buildings could be analogous to those obtained from the previous intervention scheme, if the same design requirements are fulfilled.

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- Reinforced concrete infill shear walls, which is a global intervention technique, having often a low intervention cost when reinforcement of foundations is not required. This intervention is particularly efficient in both the reduction of the lateral displacements and the increase in the building resistance.
- Steel Bracing Frames, which as in the previous case, these interventions are also global ones and they generally provide the same advantages as RC shear walls.
- Buckling Restrained Braces (BRB), which provide high deformation capacity and dissipative behaviour with high hysteretic cycles, without axial buckling of diagonals.
- Eccentric Bracing Frames (EBF), which are installed within the existing structure and have the task to concentrate the damage and, therefore, the seismic dissipation capacity, into the links, which can be short, medium or long.
- Steel Plate Shear Walls (SPSW), which consist of a series of plates that, generally located either around a service area or in the perimeter frames of the structure, form a central stiffening core capable of absorbing the distress of horizontal seismic forces.
- Base isolation systems, which are the most effective protection systems, based on a different design philosophy than that of all previous ones. They require a limited intervention level, usually at the building's basement floor only, giving to the structure a greater horizontal deformability and, as a consequence, much lower dynamic forces.

According to the aforementioned issues, the seismic behaviour and the retrofitting of existing RC buildings, with particular reference to those with public functions, is, therefore, an extremely important topic in the field of earthquake engineering. In this context, the present work has been developed as part of the European research project COST Action C26 *'Urban Habitat Constructions under Catastrophic Events'* (Mazzolani, 2010), with the aim of assessing the vulnerability of buildings due to natural and man-made catastrophic actions. In particular, the risk scenario deriving from possible eruptions of the Vesuvius volcano, located in the area of Naples, has been assessed. In situ surveys were conducted for the seismic-volcanic vulnerability analysis of private, monumental and public buildings of Torre del Greco, which is the most populated city in the districts around Vesuvius.

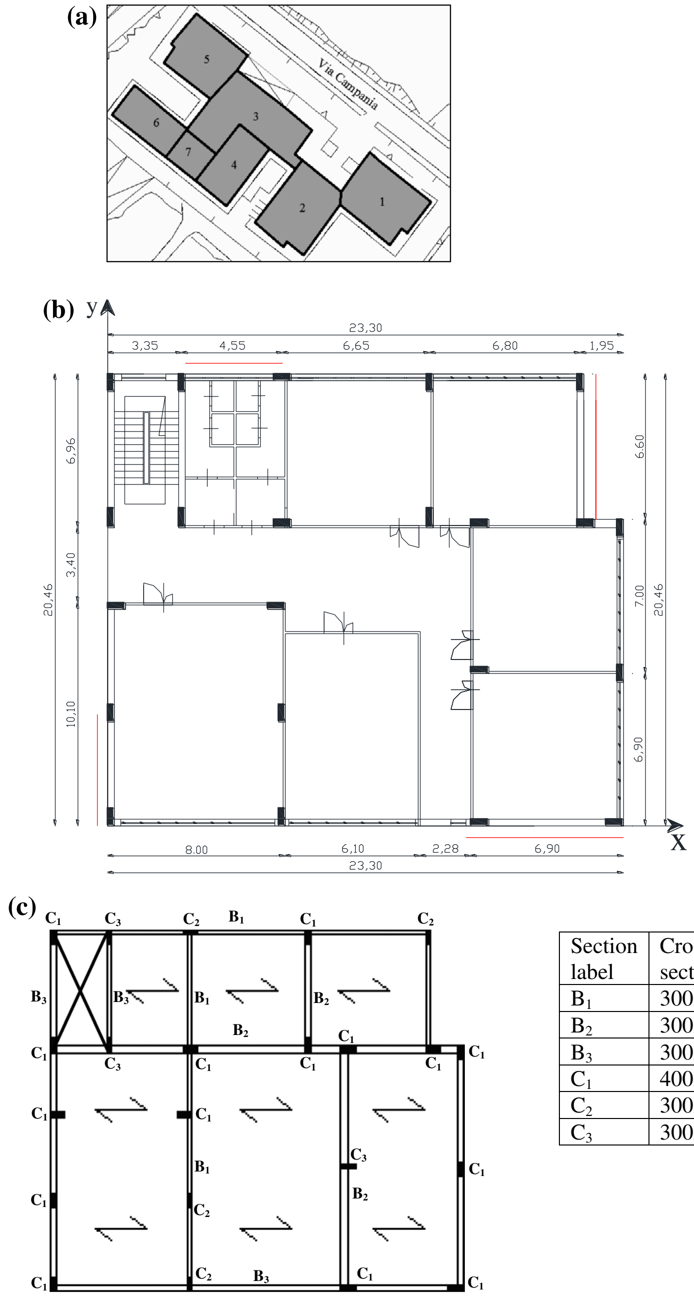
With reference to the school buildings, 10 masonry buildings (primary schools) and 5 RC buildings (secondary schools) were examined (Florio et al., 2010). The majority of such buildings was constructed without any consideration about seismic requirements and, therefore, requires seismic retrofitting interventions. In this paper, the attention is focused on the 'D'Assisi' secondary school, which consists of seven independent RC constructions separated through seismic joints. Assuming as a case study one of these constructions, reinforced concrete infill shear walls and several steel-based retrofitting interventions (concentric braces, eccentric braces, buckling restrained braces and steel plate shear walls) have been designed and their effectiveness was evaluated through seismic non-linear analyses. In particular, as first study target, the difference among interventions in terms of seismic behaviour of retrofitted buildings has been evaluated. Secondly, as a second goal of the present study, the best intervention scheme has been selected from economic, structural and environmental points of view by means of the multi-criteria TOPSIS method.

2. The case study: the 'San Francesco d'Assisi' school in Torre del Greco (Naples)

The structure under study is part of the school complex 'San Francesco D'Assisi' located in Torre del Greco, a district of Naples. The building, which was erected in the late 80s, is divided into seven RC buildings (two used as gyms), independent from each other due to the construction of seismic joints (Figure 1(a)). The structural unit object of the current research is the construction distinguishing three of the seven buildings comprising the school complex. The interiors of the modular RC structure are used as classrooms and teaching laboratories (Figure 1(b)). The selected structural unit has almost rectangular shape with plan dimensions of 19.70 m × 23.00 m and has two storeys. The structural scheme shows an eccentric arrangement of the staircase that confers to the building a plan irregularity. The seismic-resistant vertical structures are RC frames, placed in the plan vertical direction (y), which withstand the inertia loads (Figure 1(c)).

Due to the absence of the original technical drawings, the design of the structural elements in terms of geometrical dimensions and reinforcement (longitudinal and stirrups) have been obtained by means of the building simulated project, which was executed based on the rules used at the construction erection time. The mechanical properties of the concrete were determined using the results of laboratory tests performed on buildings built in the same period within the same territorial region of the investigated construction (De Matteis, Formisano, & Mazzolani, 2009). From the results of the experiments, it was found a C20/25 type for the concrete. The relatively high quality of concrete is justified by the extra care in its fabrication process observed in the construction period of the examined building, which followed the 1980 Irpinia earthquake that affected the city of Naples. In addition, for reinforcing bars, considering the time of construction and the intended use of the construction investigated (i.e. strategic building), a steel type FeB38k (f_{yk} = 375 MPa) was assumed.

The building under investigation has been modelled using the finite element software SAP2000V.17 (CSI, 2014) (Figure 2). The finite element model has been implemented with specific numerical elements considering the real behaviour of the building structural components. In particular, with respect to the modelling of beams and columns, one-dimension elements have been used, while, in order to take account of the effective stiffness of the floors, a diaphragm constraint has been assigned to all nodes of the same floor. In the modelling phase, the presence of the staircase with reinforced concrete knee beams, involving the formation of stocky columns, has also been considered. Pushover analyses have been conducted on the building, thus, plastic hinges have been assigned to each structural element. The properties of the hinges have been evaluated according to the geometrical and mechanical characteristics of each element on the basis of the OPCM 3431 instructions (OPCM, 2005), also integrated within the NTC 2008 code (MD, 2008), which are according to the EC8 Part 3 (2005), which defines the rotational capacity limits of structural elements for different limit states.



Section label	Cross section	A_s	A'_s	Stirrups
B_1	300x700	4Ø18	6Ø18	Ø6/20
B_2	300x800	5Ø18	5Ø16	Ø6/20
B_3	300x600	4Ø16	4Ø16	Ø6/20
C_1	400x800	6Ø20	6Ø20	Ø8/25
C_2	300x800	6Ø20	6Ø20	Ø8/25
C_3	300x900	6Ø20	6Ø20	Ø8/25

Figure 1. (a) The school complex 'D'Assisi', the structural unit under investigation, (b) the architectural layout and (c) the structural scheme.

The kinematic parameters have been defined according to the reference limit state and, therefore, the rotational limits have been identified for the limited damage (LD), the severe damage (SD) and the collapse (CO) limit states. At the LD limit state, the global yielding rotation capacity Θ_y of beams and columns is evaluated by:

$$\Theta_y = \chi_y \cdot \frac{L_V}{3} + 0,0013 \cdot \left(1 + 1,5 \cdot \frac{h}{L_V}\right) + 0,13 \cdot \chi_y \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \tag{1}$$

where χ_y is the yielding curvature of the end section, L_v is the shear span, equal to the distance between the end section and the section where bending moment is zero, h is the section depth, d_b is the average diameter of the longitudinal bars, and f_c and f_y are, respectively, the compressive strength of the concrete and the yield strength of the longitudinal steel (both expressed in MPa).

In addition, at the CO limit state, the total rotation capacity Θ_u with respect to the collapse condition is evaluated as follows:

$$\Theta_u = \frac{1}{\gamma_{el}} \left[\Theta_y + \left(\chi_u - \chi_y\right) \cdot L_{pl} \cdot \left(1 - \frac{0,5 \cdot L_{pl}}{L_v}\right)\right] \tag{2}$$

where χ_u is the ultimate curvature, evaluated considering ultimate strains of concrete and steel, respectively, equal to 35 and 40‰, while χ_y is the yielding curvature considering the steel in the yield state. The length of the plastic hinge L_{pl} is evaluated as follows:

$$L_{pl} = 0,1 \cdot L_v + \ 0,17 \cdot h + 0,24 \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \tag{3}$$

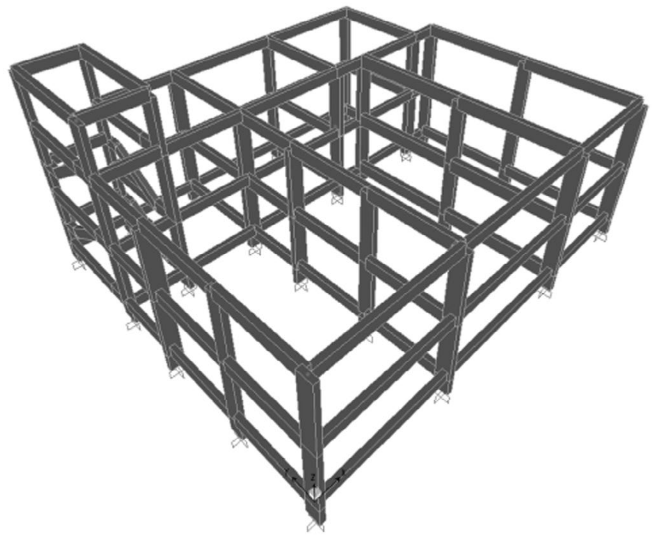


Figure 2. 3D view of the SAP 2000 numerical model of the school.

For each beam, an elastic–perfectly plastic moment–curvature relationship has been assigned, while for each column the axial compression load–bending moment interaction has been considered.

The modelling phase has been completed after assigning the respective masses to each floor and the relative seismic actions, as required by OPCM 3431 and NTC2008 standards. For the purpose of the non-linear static analysis, two seismic force distributions have been considered: one according to the first vibration mode and another proportional to the masses. The modal analysis (Figure 3) shows that in the second vibration mode building torsion occurs, which is indicative of the irregular dynamic behaviour of the structure.

The vulnerability study has been conducted following the directives of the capacity curve method of the US Guidelines ATC-40 (1996). This code provides several methods for the determination of the performance point, which is the performance required to the structure by a given earthquake at the Life Safety limit state. All these methods are based on the calculation of the design spectra corresponding to both different values of the equivalent damping and the iterative determination of the structure expected displacement according to the Capacity Spectrum Method. The iterations are firstly based on the definition of the performance-point and, subsequently, on the corresponding definition of both the energy dissipation and the associated damping up to the final result, where the convergence is attained. The operations are carried out in the ADRS (*Acceleration–Displacement Response Spectrum*) plane, which requires that a Multi Degree of Freedom (MDOF) system is transformed into a Single Degree of Freedom (SDOF). The representation in bilinear form of the structure capacity curve is required to estimate the effective (equivalent) damping and the consequent reduction in the spectral demand. The equivalent damping is defined as the sum of viscous and hysteretic contributions.

More specifically, the bilinear representation requires the definition of the previously recalled performance point, having coordinates (Sd_u, Sa_u), which are used to estimate both the

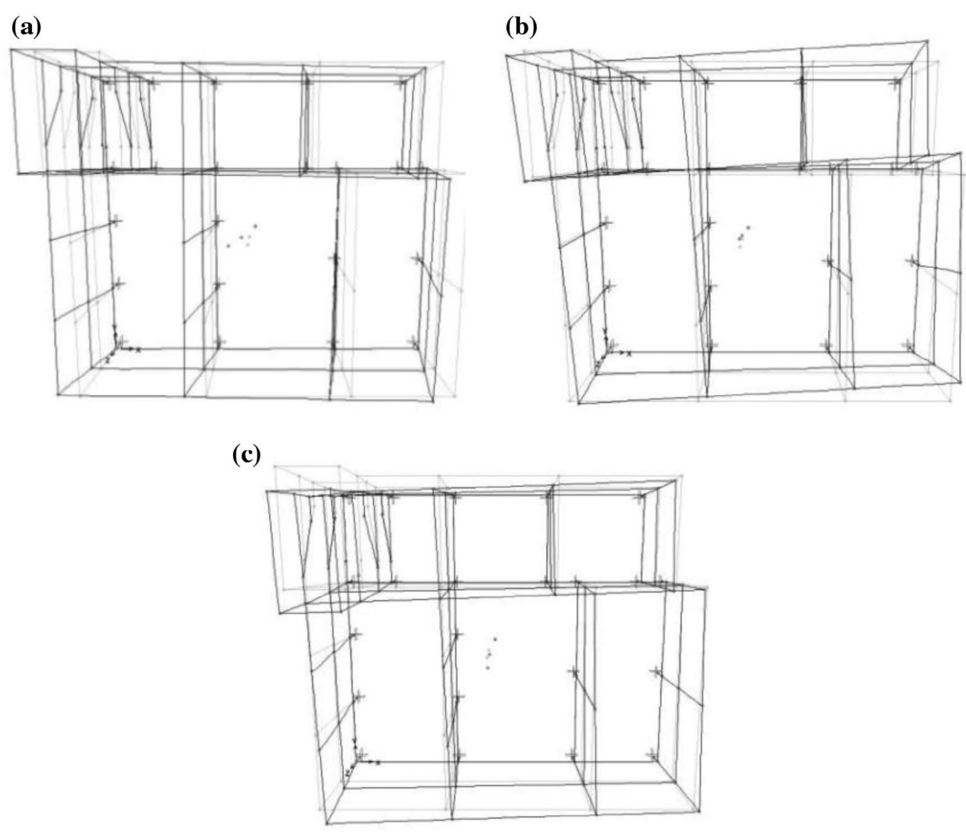


Figure 3. Modal vibration shapes: (a) first mode (T = 0,61 – mass participation factor = 0.58), (b) second mode (T = 0,43 – mass participation factor = 0.40) (c) third mode (T = 0,34 – mass participation factor = 0.43).

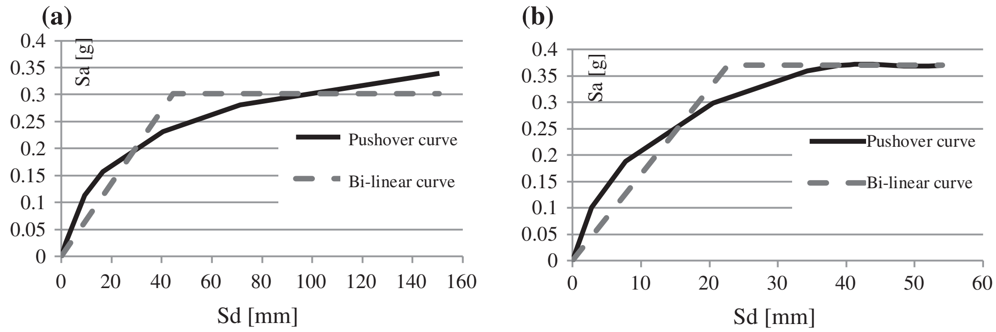


Figure 4. The structure SDOF equivalent bilinear curve: direction *x* (a) and direction *y* (b).

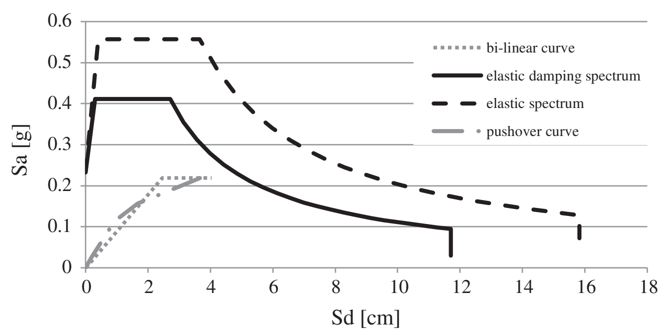


Figure 5. Pushover curve related to the formation of the first local damage in direction *x*.

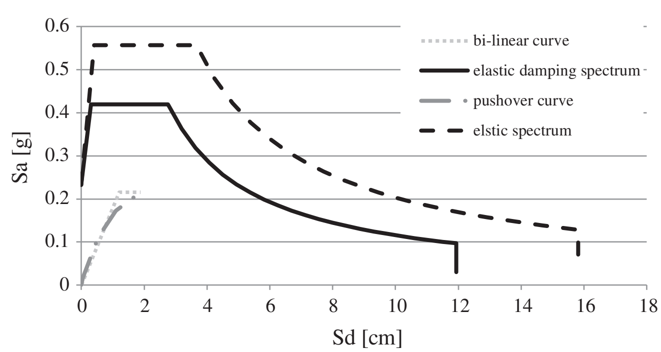


Figure 6. Pushover curve related to the formation of the first local damage in direction *y*.

expected maximum displacement and the maximum spectral acceleration. In Figure 4 the capacity curves of the examined structure are plotted. The results of the iterations (presented in Figures 5 and 6) lead to the conclusion that the demand is greater than the available structural capacity in seismic direction *y* only. Actually, in that direction, the performance points are characterised by a maximum displacement of 2.3 cm, which is greater than the corresponding structure ultimate displacement, which is equal to 1.9 cm. For this reason, an upgrading intervention is necessary so that the structure will meet the requirements of regularity and structural safety.

3. Design and application of the proposed rehabilitation systems

The upgrading design of RC buildings typically aims at increasing their stiffness and resistance with respect to horizontal actions. Various upgrading systems have been applied in the proposed case study to achieve the proposed target: Concentric Bracing Frames (CBF), EBF, BRB, SPSW and reinforced concrete (RC) infill shear walls. Taking into account the location of the staircase, the upgrading systems have been placed in an eccentric manner with respect to the school barycentre (dashed red lines in Figure 1(b)), so as to guarantee an improvement in its seismic behaviour. Therefore, the existing structural parts, i.e. the RC frames hosting the examined upgrading systems and the foundations below, have been verified under the new stress states.

The definition of the design stiffness of the upgrading system takes place through the equivalent bilinear curve in the ADRS representation, using either the 'principle of equality of displacements' or the 'principle of equality of areas'. The choice of one method over the other depends on the building first vibration period. In the examined case, according to the principle of equality of areas, the design stiffness of the upgraded structures have been determined on the basis of the target displacements at Life Safety limit state, that have been assumed equal to 3.25 and 1.55 cm in directions *x* and *y*, respectively. These displacements correspond to the displacement average values of the plateau of the original structure pushover curves.

Table 1 presents the data related to the existing structure (stiffness K_c, ultimate shear V_c, damping β_{eq}), to the upgraded structure (stiffness K_f and shear V_f), as well as to the relative difference between the upgraded structural parameters and the

Table 1. Stiffness and strength required to the upgrading systems.

	Existing structure			Upgraded structure			Relative difference	
	K_c	V_c	β_{eq}	K_f	V_F	β_{eq}	ΔK	ΔV
Direction of the analysis	N/mm	N	%	N/mm	N	%	N/mm	N
x	59900.47	1852076	18.49	143294.6	1958030	13.26	83394.2	105954
y	111417.7	3432910	12	138910.9	3432910	12.17	27493.2	385673

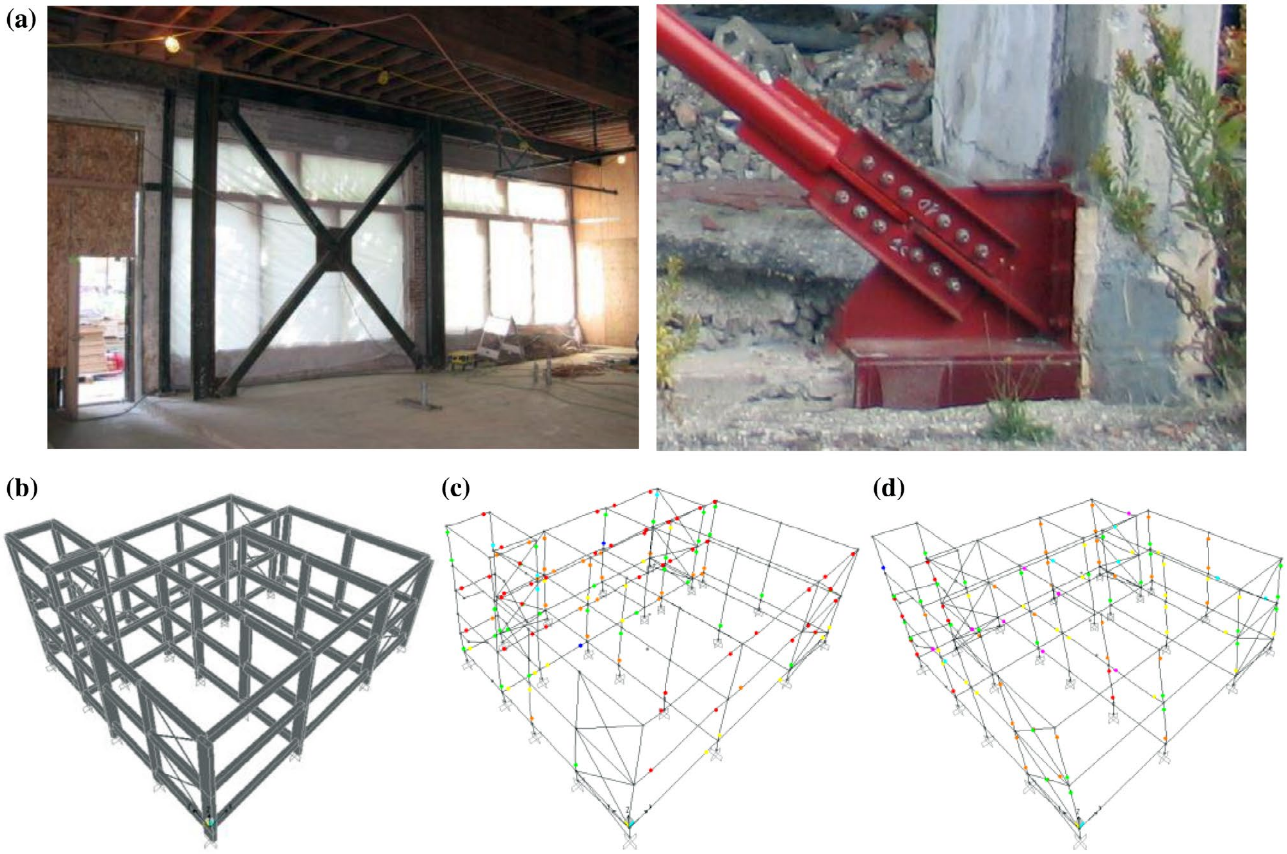


Figure 7. The St. Andrew's school with cross bracings (a), FEM model of the school retrofitted with CBF (b), collapse mechanisms in directions *x* (c) and *y* (d).

Table 2. Design of CBF systems.

Analysis direction	f_{yk} N/mm²	Cross-section	$A_{inf} = A_{sup}$ mm²	Second moment of area mm⁴	W_{pl} mm³	K N/mm	V N
x – side opposite to the staircase	275	139.7 × 5	2120	4810000	90800	71194.9	966003.2
x – staircase side		101.6 × 6.3	1890	2150000	57300	71247.7	776358
y – side opposite to the staircase		139.7 × 6.3	2640	5890000	112000	67884.1	1226041
y – staircase side		76.1 × 2.6	600	406000	14100	22411.5	250485.7

original structure ones, that is the design data for reinforcement systems (stiffness variation ΔK and strength variation ΔV). The upgraded structure has been then verified against the additional stress induced by any added retrofitting system and, whether necessary, appropriate non-dissipative reinforcement systems have been used to reinforce locally the deficient existing members.

The use of CBF for seismic strengthening existing RC buildings has been done in several applications during last 20 years (Caterino, Iervolino, Manfredi, & Cosenza, 2008; Perera, Gómez, & Alarcón, 2004; Pincheira & Jirsa, 1995) (Figure 7(a)). First, the design of the CBF system has been performed considering the data shown in Table 1, from which the stiffness and strength needs can be derived. The sizing of diagonals, made of S275 steel, has been based on the Italian code (MD, 2008) prescriptions, which foresee bracing elements either belonging to the first or to the second class (for double *T* profiles), or having a diameter/thickness ratio not greater than 36 (for circular hollow sections, Faggiano et al., 2014).

CBF have been inserted in the structural meshes in a form of circular hollow profiles arranged according to the classic St. Andrew's cross configuration (Figure 7(b)). The system has been designed in such a way so that the yielding of tensile diagonals, which have load bearing function under earthquakes, precedes both the failure of connections and the collapse of existing beams and columns. The stiffness provided by CBF is obtained by the following expression:

$$k = \frac{2E}{\frac{l_1}{A_1 \cos\alpha_1} + \frac{l_2\gamma_2}{A_2 \cos\alpha_2}} \tag{4}$$

where E is the steel elastic modulus; A_i is the brace cross-section area at the *i-th* floor; α_i is the angle formed by the *i-th* floor brace with the horizontal plane; l_i is the brace length and γ_2 is a partition coefficient of horizontal forces deriving from linear static analysis. The system resistance has been determined as follows:

$$V_y < V_{pl} = N_{pl} \cdot (\cos\alpha_1 + \cos\alpha_2) \tag{5}$$

where N_{pl} is the plastic strength of diagonals.

Regarding the non-linear modelling of dissipative elements, i.e. the diagonals, the axial hinge model according to

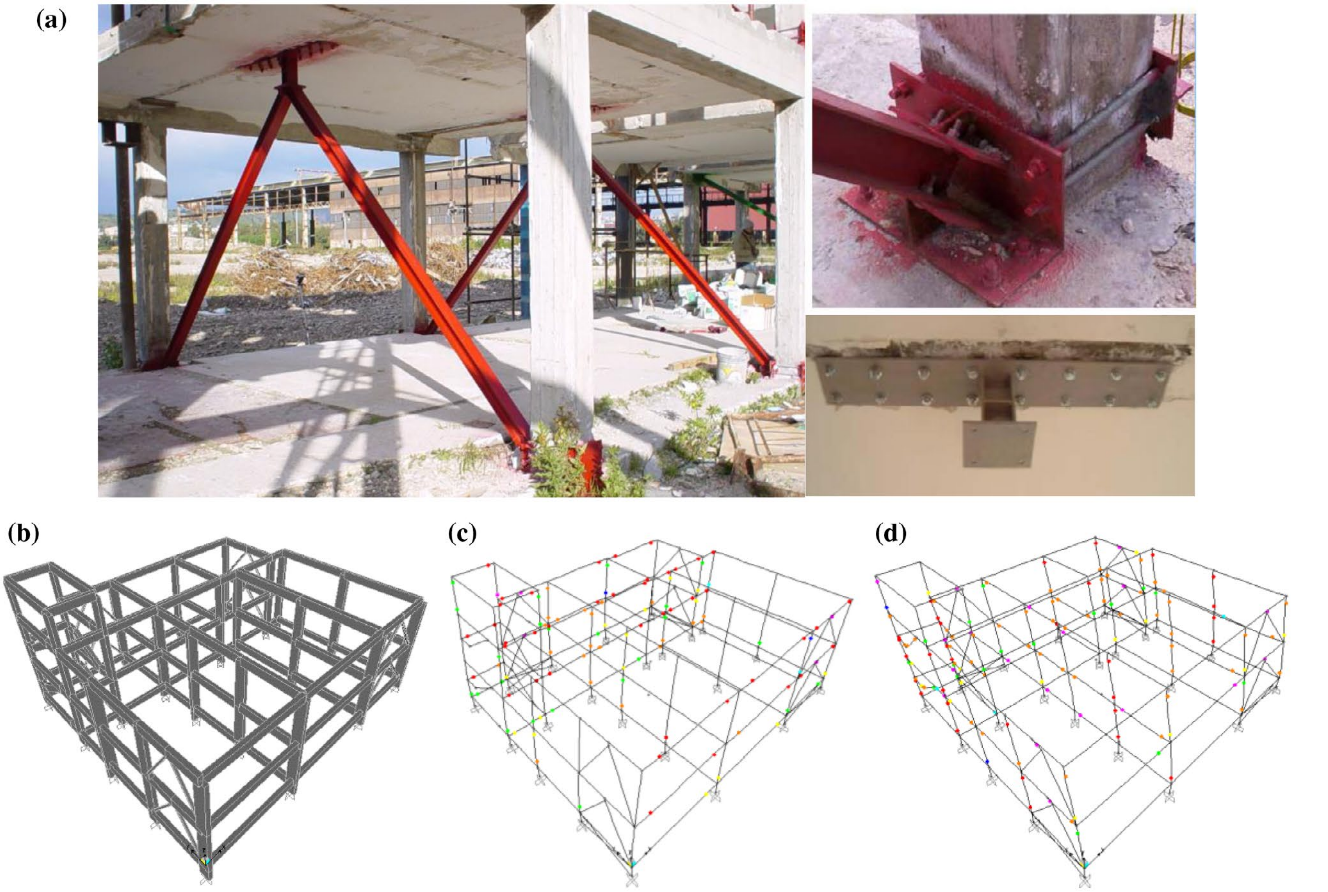


Figure 8. Application of EBF systems for seismic retrofitting (a), FEM model of the school retrofitted with EBF (b), collapse mechanisms in directions *x* (c) and *y* (d).

Table 3. Geometrical data of employed links.

Analysis direction	f_{yk} N/mm²	A_{link} mm²	Cross-section	b mm	h mm	t_w mm	t_h mm	e_{max} mm	L_{link} mm
x –side opposite to the staircase – ground floor	275	520	HEA100	100	96	5	8	221.7	200
x –side opposite to the staircase – first floor		520	HEA100	100	96	5	8	221.7	200
x – staircase side – ground floor		1490	HEB180	180	180	8.5	14	410.8	200
x – staircase side – first floor		1490	HEB180	180	180	8.5	14	410.8	200
y – side opposite to the staircase – ground floor		1490	HEB180	180	180	8.5	14	410.8	200
y – side opposite to the staircase – first floor		760	HEA140	140	133	5.5	8.5	229.8	200
y – staircase side – ground floor		1490	HEB180	180	180	8.5	14	410.8	200
y – staircase side – first floor		760	HEA140	140	133	5.5	8.5	229.8	200

the Georgescu's model (1996) has been considered, taking into account the strength degradation of the compressed brace due to instability, adjusted using the ductility limitation given by Tremblay (2002). The structural collapse mechanisms in the building's main directions show a dissipative behaviour mainly concentrated in CBF, as a confirmation of the project validity (Figure 7(c) and (d)). Table 2 presents the geometrical and mechanical data of the reinforcement system.

The EBF system transfers the axial stresses to the diagonals through the link, that is the element used for the energy dissipation (Della Corte, D'Aniello, & Landolfo, 2013; Durucan & Dicleli, 2010; Ghobarah & Abou Elfath, 2001; Mazzolani, Della Corte, & D'Aniello, 2009). This energy must be dissipated in order to avoid both the instability of diagonals and the collapse of other non-dissipative elements. The particular geometrical and structural configuration of the existing building requires the use of links placed in vertical position (Figure 8(a)). Each link is fully constrained to the beam and hinged to the bracings. The substitutability of the link is ensured by bolted connections at both its ends. This allows its easy removal when it is damaged after a severe seismic event.

The link inelastic behaviour is significantly affected by its length. In fact, the least the length, the most the influence of the shear forces over the inelastic performance. Depending on the length *e*, three different types of links can be used:

$$
\begin{aligned}
&\text{Short Link:} e \leq 0,8 \cdot (1+\alpha)\frac{M_{l,\mathrm{Rd}}}{V_{l,\mathrm{Rd}}}\\
&\text{Intermediate Link: } 0,8 \cdot (1+\alpha)\frac{M_{l,\mathrm{Rd}}}{V_{l,\mathrm{Rd}}} < e < 1,5 \cdot (1+\alpha)\frac{M_{textl,\mathrm{Rd}}}{V_{l,\mathrm{Rd}}}\\
&\text{Long Link: } e > 1,5 \cdot (1+\alpha)\frac{M_{l,\mathrm{Rd}}}{V_{l,\mathrm{Rd}}}
\end{aligned}
\tag{6}
$$

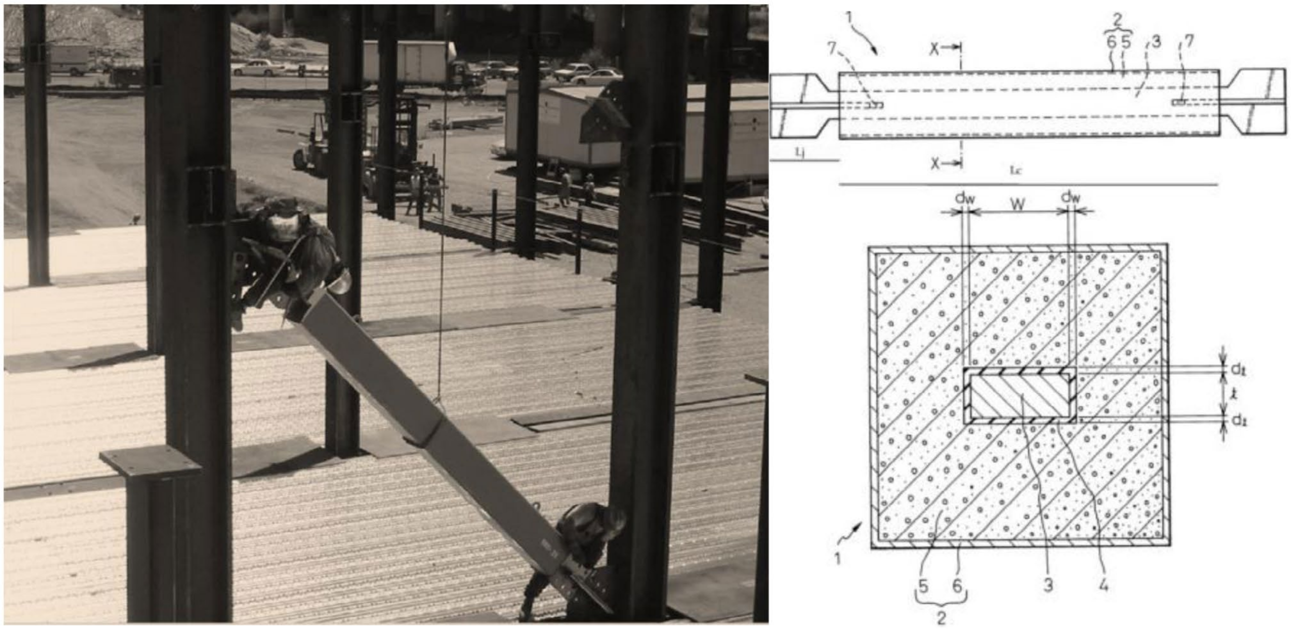


Figure 9. The BRB steel–concrete composite system.

where $M_{l,Rd}$ and $V_{l,Rd}$ are, respectively, the bending design resistance and the shear design strength of the link and α is the ratio between the lowest bending moment and the highest one expected at the two ends of the connecting element.

In the first case, the plasticisation takes place by shear, in the second by the combined effect of bending and shear, while in the last case flexural plastic hinges will appear at the link ends. The shear check is evaluated using the following formula:

$$V_y < V_{l,Rd} = \frac{f_y}{\sqrt{3}} \cdot t_w \cdot \left(h - t_f\right) \tag{7}$$

where $V_{l,Rd}$ is the applied shear, $V_{l,Rd}$ is the shear strength of the link, t_w, h and t_f represent the web thickness, the height and the thickness of the flanges link, respectively, and f_y is the steel yielding stress, equal to 275 MPa. The EBF stiffness K is evaluated assuming a system of springs arranged in series, function of the link and the diagonals properties, solicited by a system of horizontal forces:

$$K = \left(\frac{1}{K_1} + \frac{\gamma_2}{K_2}\right)^{-1} \tag{8}$$

where K_i is the stiffness of the i-th floor that takes into account the flexural deformability of link and diagonals, to be calculated according to the following relationship:

$$K_i = \left(\frac{L^3_{link,i}}{3 \cdot E \cdot I_i} + \frac{1}{2 \cdot K_{a,i} \cdot \cos^2 \alpha_i}\right)^{-1} \tag{9}$$

where L_{link} is the link length, E is the Young modulus, I is the link second moment of area, α is the inclination of the diagonal with respect to the horizontal direction and K_a is the axial stiffness of diagonals. Therefore, on the basis of the above formula, the design of the system links has been performed (Table 3).

The non-linear model foresees the presence of plastic shear hinges in the link (Figure 8(b)). With respect to the diagonals, their non-linear behaviour is based on the Georgescu–Tremblay model (Georgescu, 1996; Tremblay, 2002) already used for CBF. The energy dissipation is concentrated in the links, which avoid the plasticity of non-dissipative elements (Figure 8). The BRB are made of steel elements designed to avoid instability in compression and to ensure a stable and completely dissipative hysteretic response under tensile loads (Bosco & Marino, 2013; Della Corte, D'Aniello, & Landolfo, 2015; Tsai, Lai, Hwang, Lin, & Weng, 2004; Wada & Nakashima, 2004). The system stability is guaranteed through a core made of a steel pipe, which is either empty or filled with concrete, able to dissipate the input energy, in order to prevent local and global buckling (Figure 9).

The applied BRB system has an inverted V configuration with the intersection point of diagonals on the existing RC beam, which has been duly verified from the effect of additional stresses induced by the bracing members. The design of the BRB central plate, also called *core*, with a rectangular cross-section, has been performed using the following equation:

$$V_y < V_{pl} = 2A_c f_{yd} \cos \alpha \tag{10}$$

where A_c is the *core* cross-section, α is the bracing slope with respect to the horizontal direction and f_{yd} is the steel characteristic yielding stress, equal to 275 MPa.

Regarding the evaluation of the bracing stiffness, as in the previous case, an in series spring system (see Equation (8)) has been adopted, while the stiffness of the single braced field k_i has been determined using a system of parallel springs through the following relationship:

$$k_i = 2k_{i,eq} \cos \alpha_i \tag{11}$$

where $k_{i,eq}$ is the equivalent stiffness taking into account the variability of the core geometry calculated according to the following formula:

Table 4. Design of BRB systems.

Analysis direction	f_{yk} N/mm²	A_{core} mm²	b mm	h_{core} mm	L_{contr} mm	V_{prog} N	V_{contr} N
x –side opposite to the staircase – ground floor	275	1400	20	70	4800	26488.5	485.062
x –side opposite to the staircase – first floor		1000	20	50	5107	26488.5	32.565
x – staircase side – ground floor		1800	20	90	4259	26488.5	503.687
x – staircase side – first floor		1600	20	80	4602	26488.5	414,320
y – side opposite to the staircase – ground floor		2000	20	100	2359	23950	677,199
y – side opposite to the staircase – first floor		1800	20	90	2515	23950	571,694
y – staircase side – ground floor		400	10	40	2156	23950	115,381
y – staircase side – first floor		400	10	40	2326	23950	106,970

$$k_{i,eq} = \left(2\frac{1}{k_j} + \frac{1}{k_c}\right)^{-1} \tag{12}$$

where k_c and k_j, respectively, represent the stiffness of the central area (core) and of the larger extremities (joints) of the plate.

In the non-linear modelling phase of the BRB, the variability in the core cross-section is taken into account by defining an equivalent section, to restore the k_{eq} of a constant section plate. Such known stiffness, the core equivalent height h_{eq}, after the base b and the length L of the same core have been established, is determined by:

$$h_{eq} = \frac{k_{eq} \cdot L}{E \cdot b} \tag{13}$$

The design results of BRB systems are summarised in Table 4. Furthermore in this case, the bracing members dissipated input seismic energy, in order to preserve the existing structure from serious damages (Figure 10).

The seismic protection systems based on the use of SPSW consist of a series of metal plates realising a stiffening core able to absorb the effects of horizontal forces (De Matteis, Formisano, Mazzolani, & Panico, 2005; Formisano, De Matteis, Panico, Calderoni, & Mazzolani, 2006; Formisano, Mazzolani, & De Matteis, 2007; Görgülü, Tama, Yilmaz, Kaplan, & Ay, 2012). These systems, with small thicknesses, have to be considered as bidimensional elements, having depths and widths of the same dimension order. Generally, they are placed either in the perimeter frames of the structure or around staircases and elevators and offer high ductility, stable hysteretic characteristics and a good initial stiffness to the buildings equipped with these systems.

SPSW, which have a low erection cost and high speed of installation, are formed by connecting through welding or bolted connections a metallic panel inside a steel frame (Figure 11(a)). Shear walls realised with steel plates can be subdivided into two main types: (1) systems able to improve the strength and stiffness properties of the primary structure; (2) systems able to dissipate mainly the energy introduced by the horizontal actions in the primary structure. The first typology is related to the so-called slender panels, made of thin steel plates. Moreover, the second category is referred to the employment of the so-called compact panels that, in order to avoid shear buckling phenomena, are either characterised by the use of low yield metallic materials or supplied with ribs having opportune flexural stiffness.

According to previous studies (Formisano, De Matteis, Panico, & Mazzolani, 2008; Formisano, Mazzolani, Brando & De Matteis, 2006), the modelling of SPSW has been herein performed by the tensile diagonal bands technique, the so-called strip model theory, in which each plate is represented by a series of strip elements, hinged to the external frame, having the cross-section determined via the following formula:

$$A_s = \frac{L \cdot \cos\alpha + h_s \cdot \sin\alpha}{n} t \tag{14}$$

where n is the number of strip elements in which the plate is divided; α is the strip element inclination angle with respect to the horizontal direction; t is the slab thickness, h_s is the inter-storey height and L is the span width.

On the basis of the retrofitting design already performed on a full-scale RC structure (De Matteis, Formisano, & Mazzolani, 2008; Formisano, De Matteis, & Mazzolani, 2010), the plate stiffness is evaluated using the following relationship:

$$k = \frac{E}{4} \cdot \frac{B \cdot t}{C_{m2} \cdot n_f \cdot d} \tag{15}$$

where B, d and t are the width, height and thickness of the shear panel; n_f is the number of floors and C_{m2} is a correction coefficient, that takes into account the difference between the theoretical and numerical stiffness of the system, due to the flexibility of the columns. The system global stiffness, as in the previous cases, refers to the Equation (8). Then, the ultimate strength of the steel shear panels has been evaluated by the following relationship:

$$V = 1/2 f_{yd} tL \sin 2\alpha \tag{16}$$

In the examined case, 10 strip elements have been used, having an inclination angle equal to 45° (Figure 11(b)). Each member has a cross-section equal to the plate thickness multiplied by a width assumed as the distance between two consecutive strip

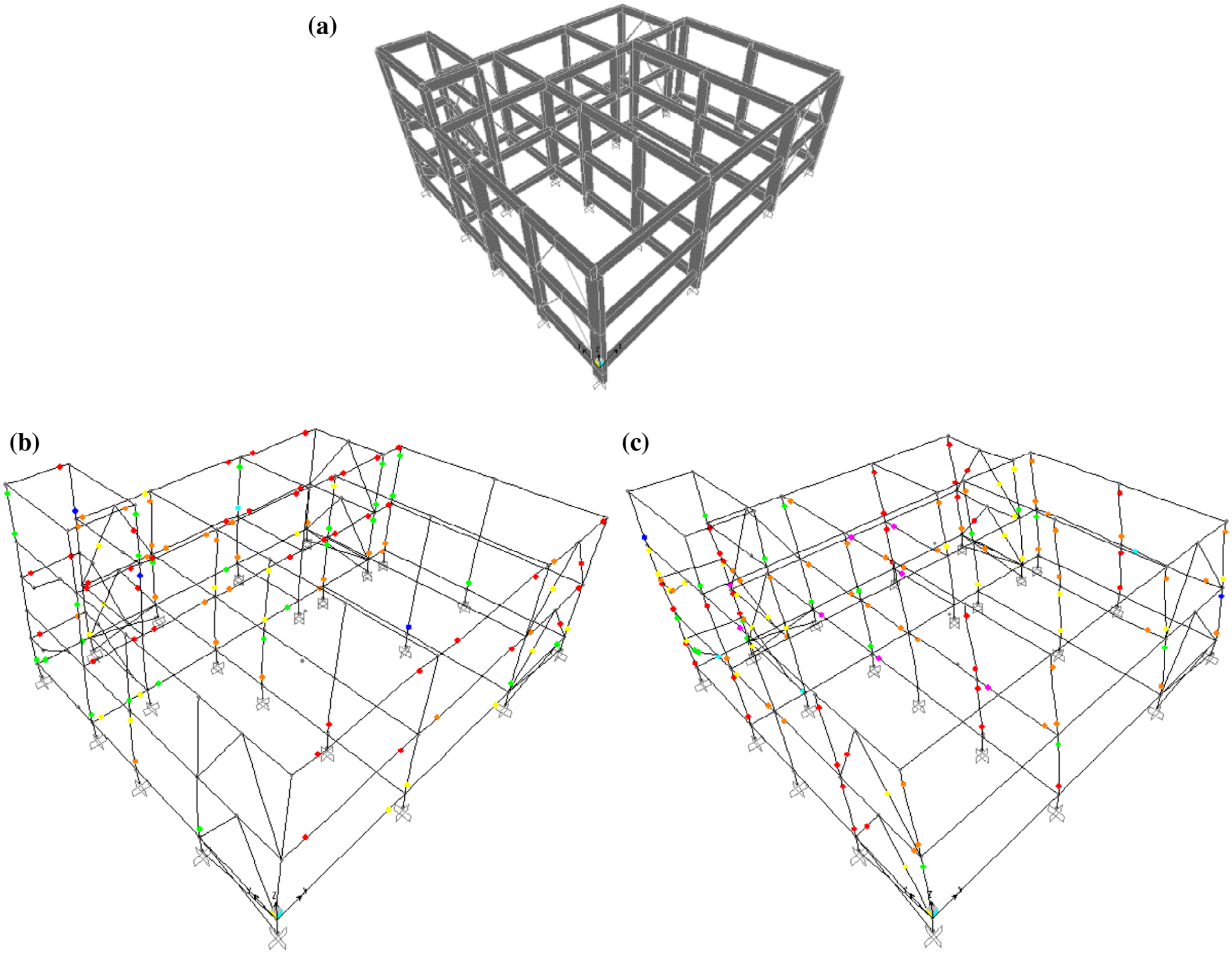


Figure 10. FEM model of the school equipped with BRB systems (a), collapse mechanisms in directions *x* (b) and *y* (c).

elements. The steel frame, in which the plate is anchored, consisting of steel double channel profiles, has been modelled with beam elements. Regarding the non-linear modelling phase, lumped plasticity hinges with axial behaviour have been placed in the middle of each truss. The introduction of SPSW has led to the regularisation of structure dynamic behaviour, also conferring, at the same time, a high energy dissipation capacity with the main plastic excursions concentrated into panels (Figure 11(c) and (d)). The commercially available thicknesses used in the model are listed in Table 5. In the direction *y*, these values have been adjusted to allow for the improvement in the structure dynamic behaviour.

Reinforced concrete infill shear walls represent a traditional upgrading system (Kaltakci, Arslan, Yilmaz, & Arslan, 2008; Kaplan, Yilmaz, Cetinkaya, & Atimtay, 2011; Karadogan et al., 2009) (Figure 12(a)). The introduction of RC shear walls in the existing structure has been implemented by increasing the size of allocated columns to satisfy the geometrical requirements provided by the Italian code for the definition of concrete walls. The dimensioning of the wall stiffness has been performed by considering a cantilever model on which two forces are applied, which represent the design forces dimensionless with respect to the base shear. Therefore, the stiffness is achieved by evaluating the top displacement due to the overlapping effect of the two forces according to the following expression:

$$k = \frac{V_y}{\delta} = \frac{EI}{\frac{h_1^3}{3} + \frac{h_1^2 h_2}{2} + \frac{\gamma_2 \left(h_1+h_2\right)^3}{3}} \tag{17}$$

where V_y is the base shear; δ is the total top displacement; h_1 and h_2 are the inter-storey heights and EI is the flexural stiffness of the wall. On the other hand, the wall resistance has been assessed by considering the bending moment at the base of the walls equal to the resistant bending moment M_{rd} of the respective cross-sections. Hence, RC infill shear walls have been formed with the geometrical configuration illustrated in Table 6.

The non-linear model of the retrofitted building involves the insertion of compression-bending hinges into the walls (Figure 12(b)). Also in this case, as in the previous ones, the walls lead to the avoidance of a localisation of the damage in the existing elements (Figure 12(c) and (d)). The application of the five seismic upgrading systems has been investigated to improve the dynamic behaviour of the existing structure, due to torsional rotation of the floors caused by the inhomogeneous location in the plane of seismic-resistant systems. Table 7 presents the values and directions of the main vibration periods for both the original

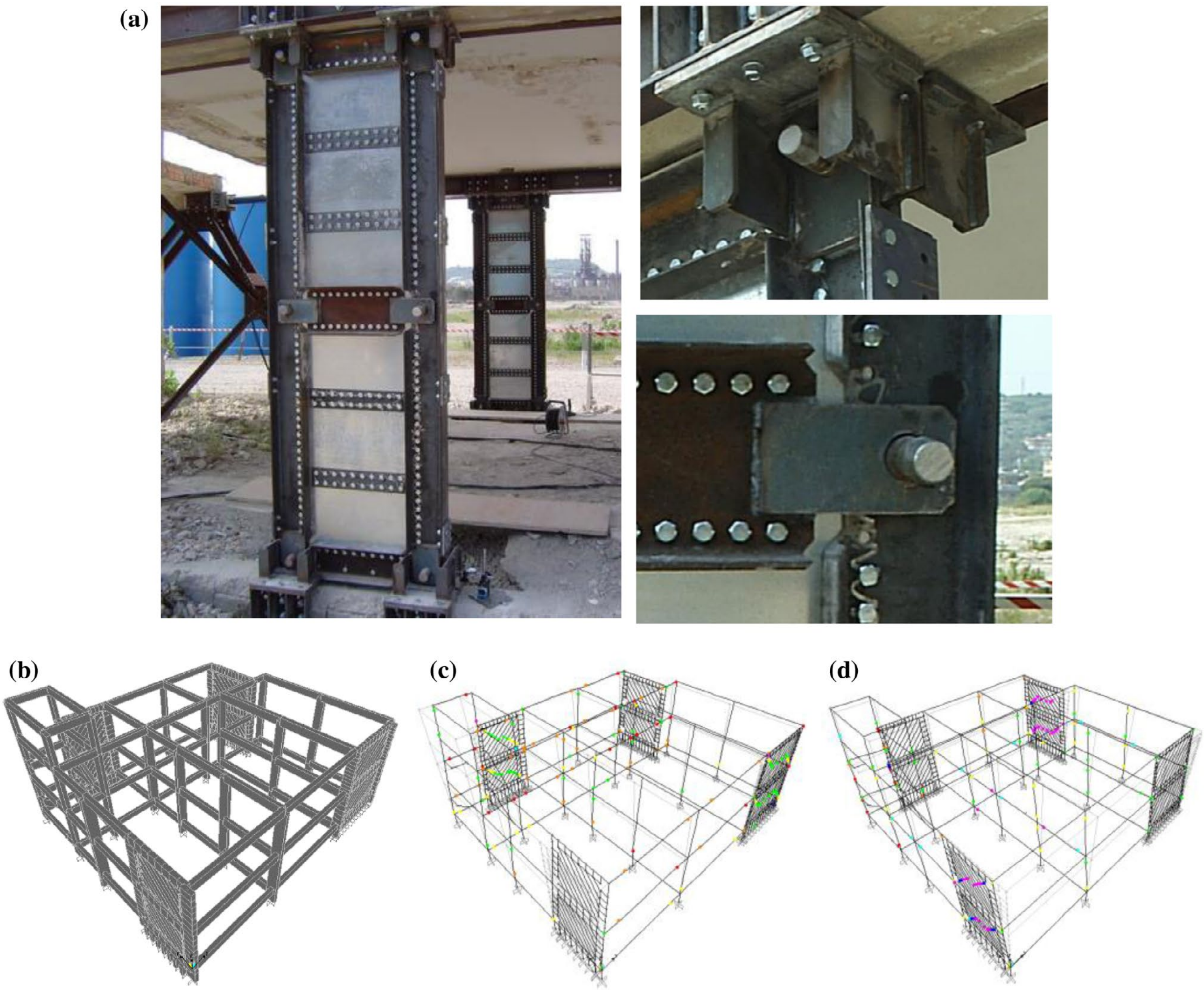


Figure 11. Application of SPSW retrofitting (a), FEM model of the school equipped with these devices (b), collapse mechanisms in directions *x* (c) and *y* (d).

Table 5. Design of SPSW systems.

	Thickness of panels (mm)			
	direction *x*		direction *y*	
Floor	staircase side	side opposite to the staircase	staircase side	side opposite to the staircase
1	0.8	1.1	0.4	1.4
2	0.9	1.2	0.4	1.4

building and the retrofitted building with the above-mentioned techniques.

From this table, it is noticed that the structural performance of the retrofitted building is improved in all cases. Actually, unlike the case of the bare structure, with all upgrading systems the first two modes are translational, while the third one is torsional. Figures 13 and 14 display the pushover curves of the structure upgraded with the different techniques considering the distributions of forces proportional to the first vibration mode and related to the structural masses, respectively.

Moreover, in all cases, the energy dissipation was concentrated in the special elements designed with dissipative function that, exhibiting extensive damage due to a better non-linear response, preserves the existing structure from brittle collapse. Referring to the examined innovative retrofitting systems, the highest increase in performance – compared to the behaviour of the bare structure – is obtained with EBF (in terms of stiffness) and SPSW (in terms of strength). Even in terms of seismic safety index, i.e. the ratio between the building capacity acceleration and the PGA, substantial performance improvements are shown. Actually, by transforming the base shear force achieved from the different retrofitted structures into acceleration values, the ratios between these values and the site PGA provide values ranging between 1.72 (SPSW) and 1.90 (BRB).

4. Selection of the optimal seismic upgrading system

In the previous section, various retrofitting techniques have been designed and applied to the examined case study of a RC school. In this section, the best technique will be chosen using a multi-criteria decision-making (MCDM) method. The MCDM analysis methods are comparison procedures based on multiple criteria aiming at contributing to the development of a learning procedure, which feeds the same decision-making process. In

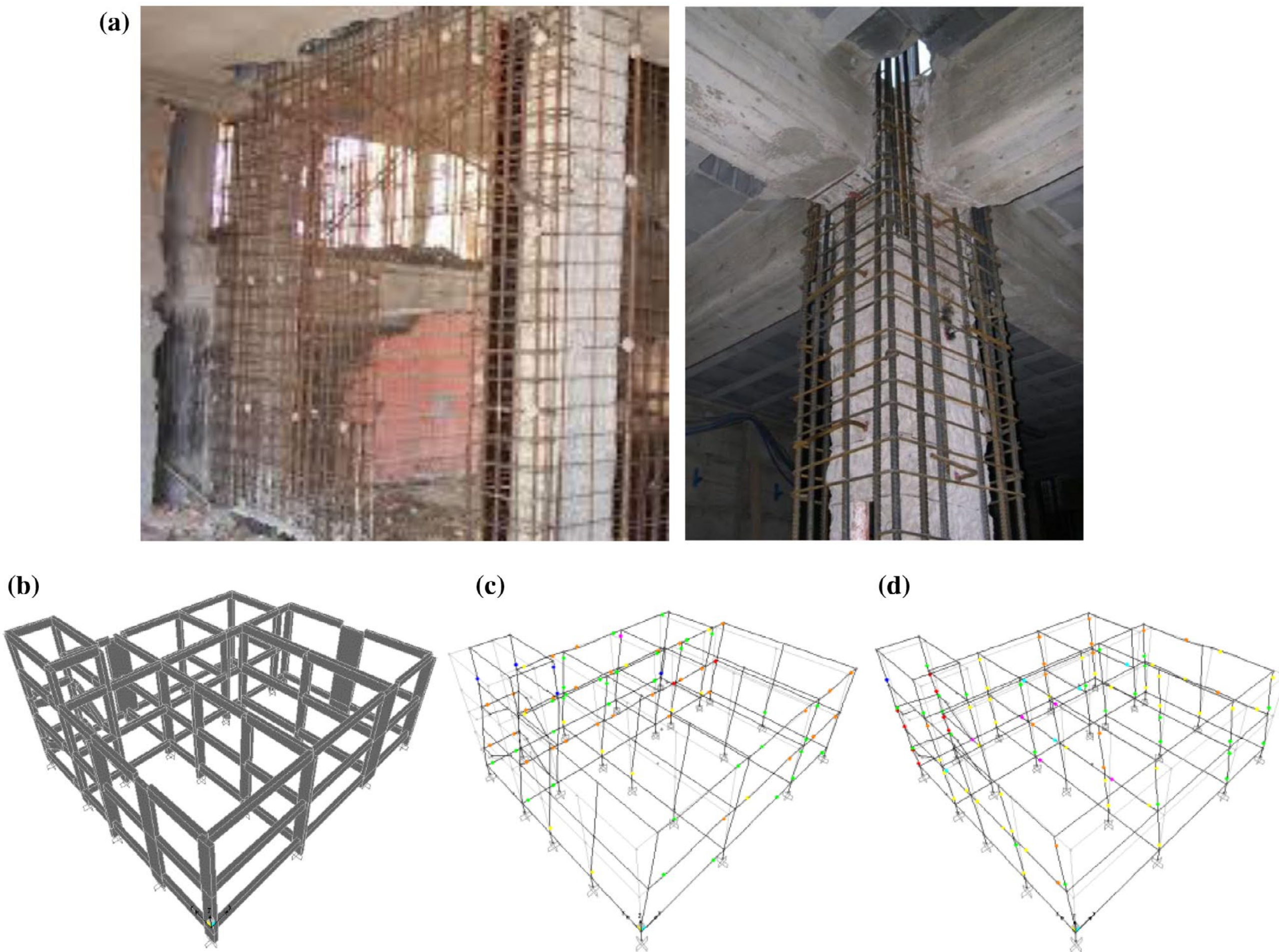


Figure 12. Application of RC shear walls (a), the FEM model (b) and collapse mechanisms in directions *x* (c) and *y* (d).

Table 6. Geometrical dimensions of reinforced concrete shear walls.

Analysis direction	B	h	h_{wall}	A	Second moment of area	A_{steel}	Number of bars
	mm	mm	mm	mm²	mm⁴	mm²	
x – staircase side	2000	7600	300	$60{\cdot}10^4$	$2000{\cdot}10^8$	3600	12ϕ20
x – side opposite to the staircase	2000	7600	300	$60{\cdot}10^4$	$2000{\cdot}10^8$	3600	12ϕ20
y – staircase side	1200	7600	300	$36{\cdot}10^4$	$4320{\cdot}10^6$	1200	4ϕ20
y – side opposite to the staircase	2200	7600	300	$66{\cdot}10^4$	$2662{\cdot}10^8$	4200	14ϕ20

particular, they can be considered as mathematical tools used to solve a decision problem by identifying the best alternative that fulfils a given number of criteria. All multi-criteria problems, regardless of their different nature, have common features (Hwang & Yoon, 1981), which can be summarised as follows:

- multiple goals/attributes: the decision-maker has to identify objectives and/or attributes relevant to the problem focus;
- conflicts between criteria;
- immeasurable measurement units;
- selection of the most satisfying alternative.

All multi-criteria decision problems are analysed by considering the following elements (Triantaphyllou, 2000):

- A 'goal' or a set of 'goals', which represents the general aim to be achieved.
- A decision-maker (DM) or a group of decision-makers (DMs) involved in the selection process, which are responsible of the evaluation procedure.
- A set of decisional alternatives, which are the fundamental elements of the evaluation and selection processes.
- A set of evaluation criteria, used by DMs to evaluate the performance of alternatives.
- The preferences of DMs, which are typically expressed in terms of weights assigned to the evaluation criteria.
- A set of scores, expressing the value of the alternative i with respect to the criterion j.

Table 7. Comparison among different retrofitting techniques.

	Existing structure		CBF		EBF		BRB		SPSW		RC walls	
	X	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*
Δ*k* (%)	–	–	106	39	170	72	91	76	124	54	57	36
Δ*V* (%)	–	–	35	25	17	13	51	16	74	48	23	9
Seismic safety index	–		1.78		1.75		1.90		1.72		1.70	
T_1 [sec]	0.61 (*x*)		0.43 (*x*)		0.39 (*x*)		0.45 (*x*)		0.44 (*x*)		0.50 (*x*)	
T_2 [sec]	0.43 (φ)		0.32 (*y*)		0.33 (*y*)		0.32 (*y*)		0.32 (*y*)		0.33 (*y*)	
T_3 [sec]	0.34 (*y*)		0.31 (φ)		0.31 (φ)		0.31 (φ)		0.32 (φ)		0.31 (φ)	

Based on the performance of the alternatives with respect to the criteria considered and, consequently, to the weights that decision-makers assign to the criteria, the different alternatives have been evaluated and sorted.

The TOPSIS (Technique for Order Preference by Similarity to Ideal Solution) method, one of the most popular MCDM techniques, has the aim to provide a ranking among several alternatives by defining two ideal (best and worst) solutions. It represents the various alternatives as points of a vector space having dimensions equal to the criteria number, so that the performance of different solutions become the coordinates in the assumed vector space. As a basis of the method, the TOPSIS technique creates two additional ideal alternatives that guide the DM to choose the optimal alternative among those considered. These two ideal alternatives are the optimal solution (A^+), having the superior performance over all criteria, and the worst one (A^-). Subsequently, the decision problem solution is represented by the alternative having, at the same time, the minimum distance from A^+ and the maximum distance from A^-.

Table 8. Values of criteria selected for the LCA analysis.

		Non-renewable energy source PEInr	Green-house effect GWP	Acidification AP	Photochem-ical smog POCP
		–	–	–	–
		C1	C2	C3	C4
Concrete	A1	0.67	0.13	0.220	0.002
Steel	A2	24	1.7	5.75	1.22

The first practical step of the method requires that the decision matrix D should be made of dimensionless elements in order to be able to compare criteria with different units. This produces the matrix R, which is made of parameters r_{ij} calculated as follows:

$$r_{ij} = \frac{a_{ij}}{\sqrt{\sum_{k=1}^{n} a_{kj}^2}} \tag{18}$$

where a_{ij} are the decision matrix elements. In this manner, all matrix elements are dimensionless. In the sequence, the weighted decision matrix V, composed of elements v_{ij}, is achieved by multiplying r_{ij} elements for the criteria weight vector ω_j according to the following relationship:

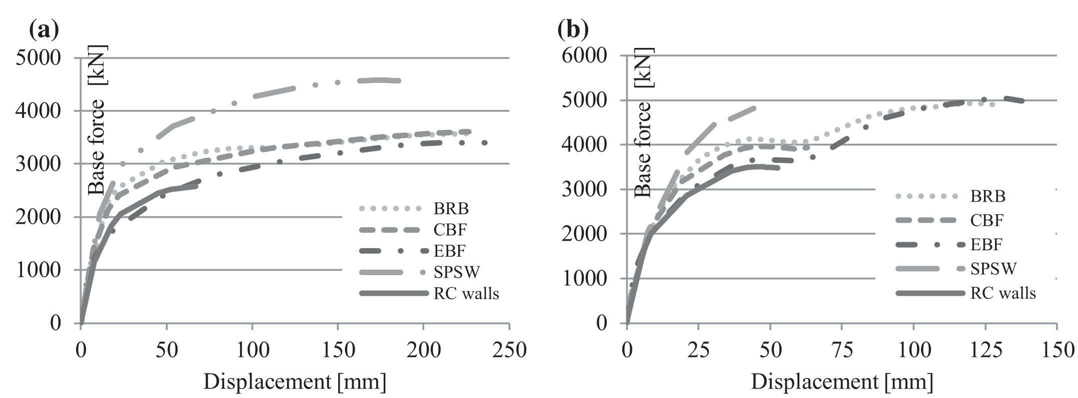


Figure 13. Pushover curves of the retrofitted structure with forces proportional to the first vibration mode in directions *x* (a) and *y* (b).

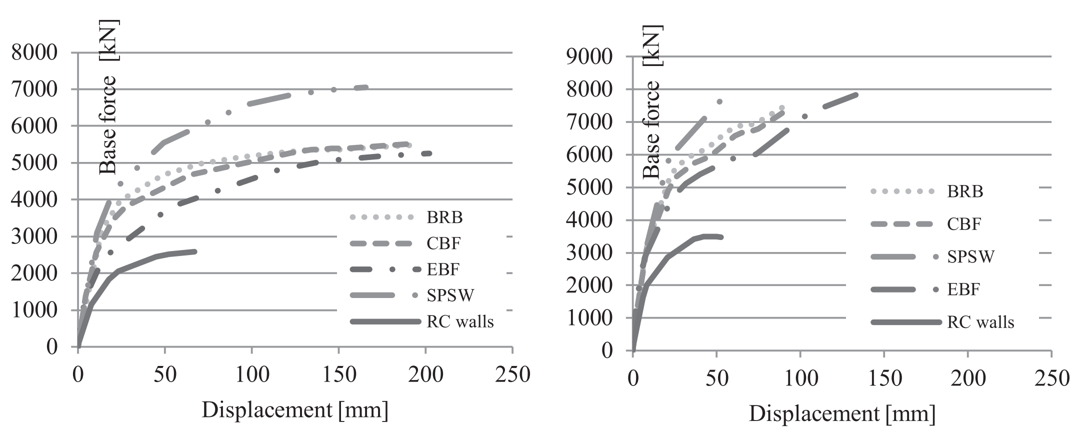


Figure 14. Pushover curves of the retrofitted structure with forces proportional to the masses in directions *x* (a) and *y* (b).

Table 9. The decision matrix.

		Seismic safety index	Continuation of the building use	Reversibility	Production energy	Cost
		–	–	–	–	€
		C1	C2	C3	C4	C5
CBF	A1	1.78	0.24	0.25	0.001	3885.72
EBF	A2	1.75	0.24	0.25	0.001	10041.85
BRB	A3	1.90	0.24	0.25	0.001	7071.32
SPSW	A4	1.76	0.24	0.23	0.001	32714.06
RC shear walls	A5	1.70	0.04	0.02	1	1943.15

$$v_{ij} = \omega_j \times r_{ij} \tag{19}$$

Subsequently, the method requires to fully assess the above two ideal alternatives. All alternatives, together with the two ideal ones, are considered as virtual points in a vector space, whose coordinates are their performance against the established criteria. Next, the distances among each alternative and the ideal ones are calculated. Hence, the preference list among alternatives can be generated by considering the following parameters:

$$S_{i+} = \sqrt{\sum_{1}^{m} j\left(v_{ij} - v_{ij+}\right)^2} \quad per \quad i = 1, 2, \ldots \ldots, n \tag{20}$$

$$S_{i-} = \sqrt{\sum_{1}^{m} j\left(v_{ij} - v_{ij-}\right)^2 \text{per } i = 1, 2, \ldots \ldots, n} \tag{21}$$

and taking as optimal solution the alternative having the minimum value of the factor C_{i+} is calculated as follows:

$$C_{i+} = \frac{S_{i-}}{S_{i-} + S_{i+}} \tag{22}$$

In general, MCDM methods are used to help the DM or a group of DMs to make objective choices not influenced by the evaluation process. In order to test the validity of the achieved results, the weight of each single criterion, taken one by one, is varied from 0 to 1, leaving all others unchanged, aiming at verifying if the ranking is changed or not. If the ranking does not change, then the stability of the obtained solution is confirmed. The generic weight absolute change able to reach a solution different from the identified one with the starting weights is indicated with *Absolute Top* (*AT*), where 'absolute' means that there is a value absolute change and 'top' indicates that this change modifies the alternative ranking top. Then, for each criterion, by dividing *AT* for the criterion weight, the *Percentage Top* (*PT*) is obtained, representing the weight change altering the ranking first solution.

Finally, the stability check of the solution is made by calculating the sensitivity parameter, achieved as reciprocal of the corresponding *PT* value. The solution will be more stable when the *PT* values are higher. In this context, robust criteria are defined when the *AT* values change does not provoke a decision problem solution alteration. Therefore, robust criteria are not sensitive to the final solution definition, since their weight variations do not change the ranking. Accordingly, when both a large number of criteria are stable and *PT* values are high, it is possible to declare with adequate certainty that the decision problem outcome is not influenced by the DM personal choices.

In the examined case, according to previous studies performed on the same topic (Caterino, 2006; Caterino, Iervolino, Manfredi, & Cosenza, 2009; Caterino et al., 2008), the purpose is to identify the best solution among the various retrofitting techniques applied to the inspected RC school building. In particular, the application of the TOPSIS MCDM method will allow to select the optimal retrofitting system towards a series of established performance criteria.

The various seismic upgrading systems, representing the alternatives for the application of the TOPSIS method and being already considered in previous applications developed by the first author (Formisano, De Lucia, & Mazzolani, 2011; Formisano & Mazzolani, 2015) are:

(1) Concentric Bracing Frames (CBF);
(2) Eccentric Bracing Frames;
(3) Buckling Restrained Braces;
(4) Steel Plate Shear Walls;
(5) Reinforced concrete infill shear walls (RC walls).

On the other hand, for comparing the above systems, different criteria have been considered. They can be of quantitative or qualitative type, with the former that can be expressed with a numerical value and the latter specified only through a verbal judgement. In addition, benefit or cost criteria can be considered. A benefit criterion should be maximised as much as possible, whereas the cost should be possibly minimised. The considered criteria are:

- *Seismic safety index:* it is a quantitative benefit criterion, based on the ratio between the building capacity acceleration and the PGA.
- *Continuation of the school activities:* it is an essential qualitative benefit criterion, which is based on a verbal judgement, to be taken into account when the building seismic

Table 10. Cost comparison among used seismic upgrading systems.

	Weight of steel		Volume of concrete		Cost		Total cost
	kg		m^3		€		
Seismic upgrading system	*x*	*y*	*x*	*y*	*x*	*y*	€
CBF	842,12	591,73	–	–	2.282,15	1.603,58	3.885,72
EBF	1.718,75	1.986,73	–	–	4.657,82	5.384,03	10.041,85
BRB	1.077,60	917,78	5,35	4,73	3.784,70	3.286,62	7.071,32
SPSW	6.049,81	6.021,80	–	–	16.394,97	16.319,09	32.714,06
RC shear walls	42,12	31,59	9,00	7,65	1.056,88	886,26	1.943,15

upgrading is made. In fact, the techniques which allow avoiding both the evacuation of the building from each occupants and the interruption of the daily activity should be preferred.

- *Reversibility:* it is the capacity of a given alternative to be easily removed from the building when other interventions are requested. It is a beneficial qualitative criterion and, therefore, the preference is devoted to techniques having this prerequisite.
- *Cost of the intervention:* it is a cost criterion that generally represents the major obstacle to overcome when a seismic upgrading intervention is examined. Obviously, the lowest possible value of this criterion is preferable. Moreover, it is a quantitative criterion which can be directly related to the analysis results.
- *Production energy:* this parameter indicates the pollution due to the intervention. For assessing this criterion another method has to be used, taking into account life cycle assessment (LCA) parameters, i.e. considering CO_2 emissions, etc.

The LCA is an analysis method evaluating functions and interactions that either a product or a service has with the environment during its entire life cycle. It includes the steps of pre-production, also considering the extraction and the production of primary materials, production of the finite element, distribution, use, also taking into account any reuse of the product as well as materials used for normal maintenance, recycling and final disposal. In the present work, only general information on the life cycle of each product are given, that is only a partial assessment of the processes of pre-production and fabrication of the products has been done, whereas a comprehensive study of all the processes occurred during their whole life cycle has not been performed. In Table 8, as already adopted in previous studies (Di Lorenzo, Formisano, Landolfo, Mazzolani, & Terracciano, 2010; Terracciano, Di Lorenzo, Formisano, & Landolfo, 2015), the data related to different LCA criteria for concrete and steel have been reported (Lavagna, 2008).

In the examined case study, as in any MCDM method, two basic elements are needed: the first is the decision matrix (rectangular $n \times m$, where n is the number of alternatives and m is the number of criteria), in which the performance of each alternative is examined taking into account all criteria; the second is the criteria weight vector. In Table 9 the decision matrix based on five alternatives against the five criteria previously described are listed. In this table, the intervention costs are determined on the basis of the detailed analysis of the upgrading system elements cost reported in Table 10. In Table 11 the matrix of preferences that will be later used for the consistency verification is reported. Moreover, in Table 12 the criteria weight vector, which has been determined using the AHP method developed by Saaty (1980) is presented.

Once these matrices are known, it is possible to calculate the matrix R (Table 13). Subsequently, the matrix R elements multiplied by the criteria weight vector allow defining matrix V (Table 14). Matrix V is used to define the virtual alternatives, namely the best A^+ and the worst A^-, representing, respectively, the best and the worst performance of the alternatives with respect to the selected criteria (Table 15). Finally, the distances between the virtual alternatives and the real ones have been calculated aiming at solving the decision problem, so as to generate

Table 11. The matrix of preferences.

	C1	C2	C3	C4	C5
C1	1	3	4	5	6
C2	0.33	1	2	3	4
C3	0.25	0.50	1	2	3
C4	0.20	0.33	0.50	1	1
C5	0.17	0.25	0.33	1.00	1

Table 12. The criteria weight vector.

	Criterion weight	
Seismic safety index	w_{C1}	0.489
Continuation of the building use	w_{C2}	0.228
Reversibility	w_{C3}	0.142
Production energy	w_{C4}	0.076
Cost	w_{C5}	0.064

Table 13. The matrix R.

	C1	C2	C3	C4	C5
A1	0.4474	0.4983	0.5095	0.0010	0.1104
A2	0.4399	0.4983	0.5095	0.0010	0.2852
A3	0.4776	0.4983	0.5095	0.0010	0.2008
A4	0.4424	0.4983	0.4687	0.0010	0.9290
A5	0.4273	0.0830	0.0408	1.0000	0.0552

Table 14. The matrix V.

	C1	C2	C3	C4	C5
A1	0.2188	0.1138	0.0725	0,0001	0.0071
A2	0.2151	0.1138	0.0725	0.0001	0.0183
A3	0.2335	0.1138	0.0725	0.0001	0.0129
A4	0.2163	0.1138	0.0667	0.0001	0.0595
A5	0.2089	0.0190	0.0058	0.0763	0.0035

a ranking among alternatives (Table 15). From Table 16 it can be noticed that the application of the TOPSIS method nominates BRB as the best seismic retrofitting system of the inspected school building.

As a conclusive step of this investigation, only the sensitivity of the solution found should be ascertained. The procedure is expected to vary the weight of each criterion for assessing whether there is a change in the top of the alternative ranking. If the ranking does not change, then the stability of the solution found is confirmed. For sensitivity analysis, first the parameter 'Absolute Top' (AT) has to be determined as the absolute value difference between the criterion weight and the minimum weight variation altering the ranking. Subsequently, the following parameters are calculated:

$$PT = \frac{AT}{\text{weight}} \tag{23}$$

$$\text{Sensitivity} = \frac{1}{PT} \tag{24}$$

where the '*Percentage Top*' (PT) represents the weight change altering the ranking first solution and the sensitivity is the reciprocal of PT. The solution will be more stable for higher PT values.

The sensitivity analysis conducted for choosing the best seismic upgrading system has provided the results shown in Table 16.

Table 15. The virtual solutions.

	C1	C2	C3	C4	C5
A+	0.2345	0.1627	0.0800	0.0000	0.0035
A–	0.2098	0.0177	0.0095	0.0763	0.0595

Table 16. Ranking of alternatives.

		C+
First	A3	0.941
Second	A1	0.907
Third	A2	0.860
Fourth	A4	0.698
Fifth	A5	0.284

Table 17. Sensitivity analysis.

Weight	AT	PT	Sensitivity
0.489	0.209	42.74%	2.34
0.228	–	–	–
0.142	–	–	–
0.076	–	–	–
0.064	–	–	–

In this context, the four criteria are robust, that is the *AT* values change does not provoke a decision problem solution alteration, and one criterion has a quite high *PT* value. This means that the decision problem outcome is sufficiently stable, i.e. it is not influenced by the personal choices of the DM. In conclusion, the applied MCDM method is a reliable technique to guide the DM when problems of seismic retrofitting with different solutions of existing buildings are concerned. This method is able to provide, thanks to the final sensitivity analysis, the best retrofitting technique from different viewpoints, namely structural, environmental and economical (Table 17).

5. Conclusions

In the present paper, the problem of seismic upgrading of a RC school building by means of a number of retrofitting techniques has been examined. Non-linear static analyses have shown that the seismic upgrading systems designed allow to increase stiffness and strength of the existing building, also providing an improvement in its dynamic behaviour, i.e. by achieving a regularisation of the structure dynamic behaviour, with a third vibration period always of torsional type.

Among the various upgrading systems shown, those involving limited reinforcement of existing elements are the BRB and reinforced concrete shear walls, which therefore require less intervention costs. The increase in the resistance of the existing structure is particularly high when intervention schemes have been conditioned by improving structural stiffness. In some cases, it is possible to notice a large stiffness increase, which is attributed in the direction x to the limitations dictated by the NTC08 code and in the direction y to the regularisation of the structural behaviour due to interventions.

In all analysis cases, the energy dissipation has been always concentrated in the upgrading dissipative systems, which have preserved the existing structure from damage. The comparison between the bare structure behaviour and the upgraded structures has shown that the greatest performance increases in terms of stiffness and strength have been achieved with EBF and SPSW, respectively. Noteworthy performance improvements have been found even in terms of seismic safety index, with values ranging between 1.72 (SPSW) and 1.90 (BRB). As a conclusion, the results obtained from the analyses conducted show the effectiveness of all the devices tested for the upgrading of RC school building investigated.

Finally, in order to detect the best upgrading solution, the MCDM TOPSIS method has been used. With this method, the five upgrading techniques examined (alternatives) have been assessed against certain cost, structural and environmental criteria. The analysis results have provided BRB as the best seismic retrofitting system of the school building inspected, followed by CBF and EBF. In contrast, the seismic-resistant systems based on shear walls made of either steel or reinforced concrete represent the worst retrofitting solutions. Moreover, as a final step, a sensitivity analysis has been conducted in order to prove the reliability of the obtained solution. It has been found that, since the four criteria were robust and the last one had a sufficiently high PT value, the achieved result is sufficiently stable, without being significantly affected by the personal choices of the DM.

As a final outcome of the study, the applied TOPSIS method can be considered as a reliable technique to solve problems of seismic retrofitting with different solutions for existing buildings. This method is able to provide, thanks to the final sensitivity analysis, the best retrofitting technique from different viewpoints, namely: structural, environmental and economical, and under an unquestionably objective manner. Conclusively, the novelty of the proposed analysis approach is related to the capacity of DM, through MCDM methods, of addressing the economic resources to identify, among different steel-based intervention techniques not always applied in practice, the best solution for upgrading the examined building. This can be applied to low-height structures as the one examined and, in order to have more general applicability, further analysis including multi-storey buildings should be performed.

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