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CIVIL ENGINEERING | RESEARCH ARTICLE

Low yield metals and perforated steel shear walls for seismic protection of existing RC buildings

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Abstract: In the field of the seismic protection of buildings, the use of steel plate shear walls (SPSWs) may be particularly profitable in the seismic retrofitting interventions of existing RC buildings designed for gravity loads only. Some past researches have shown that when traditional full SPSWs are used as bracing devices of framed buildings, they may induce excessive design forces to the surrounding frame members. Therefore, low yield steels (LYS) could be a valuable option to overcome this applicability limit. Nevertheless, the scarce availability on the market of these steels suggests the employment of aluminium alloys and perforated steel plates, which have the benefit of incurring excursions in plastic range already for low stress levels. In this paper, a parametric analysis concerning the use of perforated metal plate shear walls (MPSWs) for seismic upgrading of existing RC framed structures represents a novelty of the research in the retrofitting interventions field. To this purpose, first, some experimental tests have been considered to calibrate a finite element model of the panel devices by using the SeismoStruct software. Subsequently, the proposed FEM model has been used to design the retrofitting systems with either full MPSWs or perforated SPSWs of an existing RC residential five-storey building, designed between the 1960s and 1970s of the last century. Finally, the different retrofitting panel systems examined have been compared to

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PUBLIC INTEREST STATEMENT

Seismic retrofitting of existing structures by means of innovative interventions is a topic of central interest in the field of Civil Engineering. Many existing buildings in seismic areas do not have proper requirements to resist earthquakes and the research interest is pointing towards the study of new techniques and development of advanced materials. Costs of retrofitting solutions always represent an important aspect to account for. Therefore, innovative retrofitting solutions not expensive for stakeholders and easy to be implemented in commercial software packages for designers represent an appealing topic. In this light, metal plate shear walls can be a useful alternative when significant upgrading of the seismic performance of existing buildings is sought. In this paper, the Authors presented a possible way to implement such systems in numerical models and showed a comparison in terms of performance and costs among different solutions of metal plate shear walls.

each other in terms of both structural and economic viewpoints, allowing to select the best intervention strategy.

Subjects: Environmental and Geotechnical Engineering; Structural Engineering; Design

Keywords: metal plate shear walls; perforated panels; bracing devices; existing RC buildings; retrofitting; FEM model; parametric analysis

1. Introduction

Steel plate shear walls (SPSWs) represent an effective passive control system, they being characterised by high initial stiffness, great strength, and a very stable hysteretic response up to large deformations. SPSWs are very effective in limiting the inter-storey drifts of framed buildings, also reducing the structure weight, as well the seismic forces, in comparison to RC shear walls. In addition, by using shop-welded or bolted connection types, the erection process can be easy, allowing a considerable reduction of constructional costs. Application examples of such devices in new steel buildings, with either bracing or dissipative functions, are detected in Asia and America (Astaneh-Asl, 2000). However, the use of SPSWs may be particularly profitable for seismic retrofitting of existing RC buildings designed for gravity loads only, since their inclusion in the existing structures confers them a considerable performance increase (Mistakidis, De Matteis, & Formisano, 2007). The beneficial contribution offered by shear panels is guaranteed by the development of a diagonal tensile bands mechanism (called tension field), which is more effective as greater is the plate area involved in the deformation process (Basler, 1961). In particular, when traditional full systems, configured as simple steel panels without stiffeners, are employed, the optimal behaviour is guaranteed with plates having width-to-height ratios between 0.8 and 2.5 (Formisano, Mazzolani, & De Matteis, 2007).

Some studies have shown that when full SPSWs are used as bracing devices of framed buildings, they may introduce some problems in the capacity design application, so to result in excessive design forces to the surrounding frame members, thus increasing their required size and costs (Vian & Bruneau, 2005). The scarce availability on the market of low-yield steel (LYS), usually employed to limit the forces transmitted by the plates to steel frame members, suggests the use of aluminium alloys (Brando, D'Agostino, & De Matteis, 2015; De Matteis, Formisano, & Mazzolani, 2009; Formisano, De Matteis, & Mazzolani, 2010; Formisano & Mazzolani, 2016; Foti, Diaferio, & Nobile, 2013) and perforated steel plates (Purba & Bruneau, 2007), which have the benefit of experiencing excursions in plastic range already for low stress levels. A recent study carried out by the Authors has shown the suitability of such panels for seismic-resistant applications through the set-up of an easy design tool useful for their use (Formisano, Lombardi, & Mazzolani, 2016).

In this paper, in order to perform a parametric analysis concerning the use of full metal plate shear walls (MPSWs) and perforated SPSWs for seismic upgrading of an existing RC-framed structures, first the experimental test results conducted in Bagnoli (district of Naples) (De Matteis et al., 2009; Formisano et al., 2010) have been considered to calibrate a finite element model in SeismoStruct (Seismosoft, 2014). In this software, shear walls have been implemented according to the equivalent tensile diagonal method proposed by Thorburn et al. (1983). The excellent experimental-to-numerical correspondence of results has validated the proposed modelling procedure for the application of full MPSWs and differently perforated SPSWs into an existing residential five-storey RC building in Torre del Greco, a town in the province of Naples. Following the same design approach reported in Mistakidis et al. (2007), pushover analyses on the retrofitted structure with the before mentioned metal shear walls have been performed. Finally, the structural and economic differences among these solutions have been exposed and critically discussed aiming at individuating the best solution for retrofitting the inspected building.

2. The experimental study

In order to both study the behaviour of existing RC buildings retrofitted by perforated SPSWs and validate the model proposed in SeismoStruct (Seismosoft, 2014), an experimental test performed

in the framework of the ILVA-IDEM project (Mazzolani, 2008) has been considered. In this project, the retrofitting of an industrial building located in Bagnoli (district of Naples) by using various reinforcement systems, including the aforesaid panels, was carried out.

Thanks to the regular configuration of the structure, once cladding and internal walls were eliminated, the building was divided in six test modules (Figure 1). Different upgrading techniques were tested; in particular, the module no. 5 was chosen to test the metal shear panels system.

The geometrical configuration of this substructure is characterized by a rectangular plan with dimensions of 6.30×5.90 m and two 3.30 m high levels (see Figure 2). More details about geometry, materials and steel reinforcement can be found in Formisano et al. (2010).

The seismic retrofitting of the RC module was designed following the performance-based design approach according to US guidelines (Applied Technology Council (ATC)-40, 1996, Federal Emergency Management Agency(FEMA)-273, 1997).

Steel panels with dimensions of $600 \times 2,400 \times 1.15$ mm and pure aluminium (AW-1050A) plates with dimensions of $600 \times 2,400 \times 5.00$ mm were chosen for experimental tests. Mechanical properties of these panels are reported in Table 1.

Since the Canadian code (Canadian Standards Association (CSA), 2001) suggests the use of plate width-to-height ratios between 0.8 and 2.5, the use of intermediate stiffeners, composed by two steel plates connected through bolted connections, was foreseen. The thickness of the stiffeners plate was determined according to the EC3 provisions (EN 1993-1-1, 2005). Furthermore, an intermediate steel beam was considered to reduce bending effects of the steel columns of the

Figure 1. View of the original RC building (a), its division in sub-structures (b) and plan configurations at different levels (c).

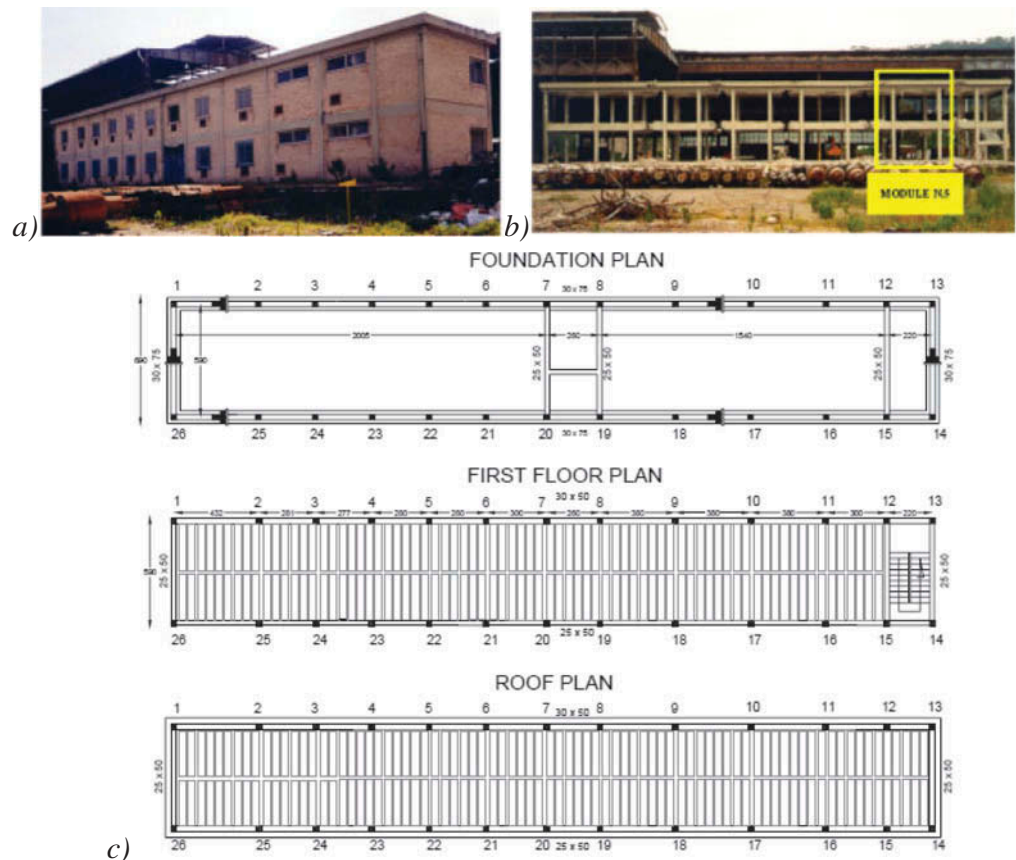


Figure 2. Test set-up of the module no. 5 by using module no. 6 as retaining structure (a), first floor carpentry (b) and vertical sections (c).

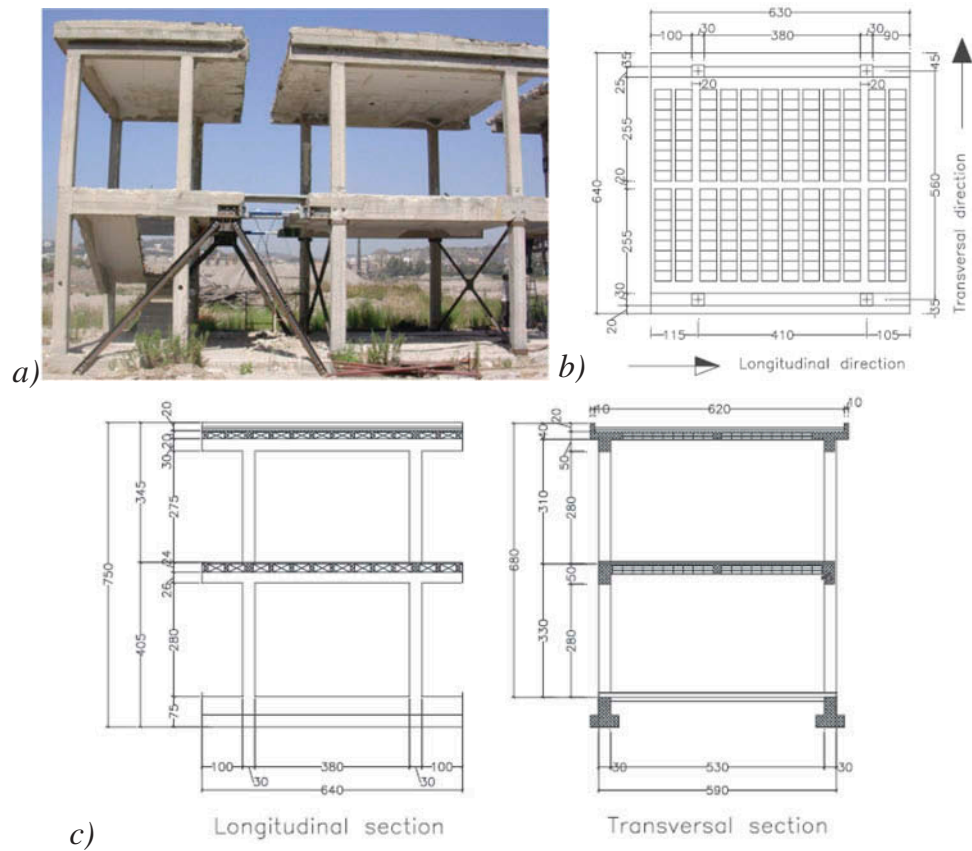


Table 1. Mechanical properties of panels obtained from experimental tensile tests

Material	f_y (MPa)	f_u (MPa)	ϵ_u	E (MPa)	Strain hardening factor
Steel	305	340	32	200,000	1.15
AW-1050A	21*	80	45	70,000	3.76

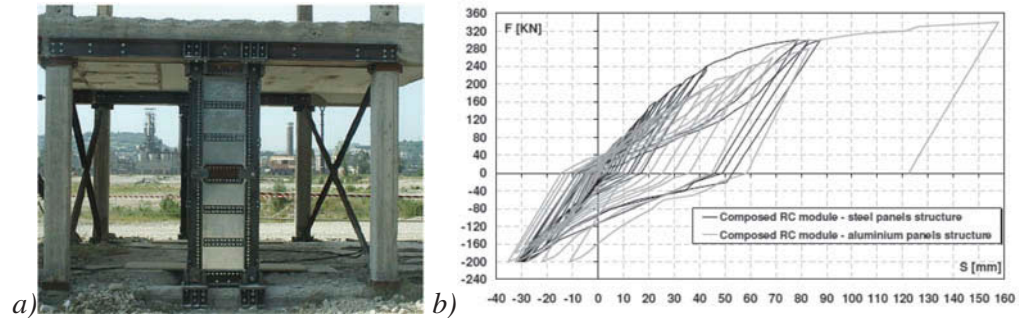
*Conventional yielding strength at 0.2% strain level.

surrounding frame. This steel frame, having pinned joints, was made of S275 UPN180-coupled profiles for perimeter beams and columns and by S275 UPN240-coupled sections for the intermediate beam. It was designed in order to both avoid any buckling phenomenon and resist to the effects induced by the tension-field mechanism of plates. The plate-to-frame connections were realised by means of bolted joints. Both the first level and foundation RC beams were reinforced by two UPN220-coupled profiles, opportunely stiffened, designed in order to absorb the maximum load transferred during test. Figure 3 shows one of the described retrofitting systems and the results obtained from experimental tests.

3. Set-up and validation of the FEM model

The choice of an appropriate and easily implementable FEM model to both simulate the above experimental tests and perform supplementary numerical analyses is a crucial importance task. In order to carry out a parametric analysis on the application of both full and perforated SPSWs inside existing RC framed structures, the FEM software SeismoStruct (Seismosoft, 2014) has been used. This software can predict the behaviour of three-

Figure 3. Retrofitted structure with steel plates (a) and comparison of the experimental load–displacement curves (b).



dimensional framed structures under static and dynamic loads by taking into account both geometric non-linearity and materials inelasticity. So, the explicit modelling of the inelasticity diffusion both along the element and through the section allows for an accurate estimate of the damage accumulation.

For monotonic analyses, metal shear panels can be simply schematised by a single equivalent tensile diagonal (Thorburn et al., 1983) having a cross-section area equal to

$$A_d = \frac{t b \sin^2 2\alpha}{2 \sin\beta \sin 2\beta} \quad (1)$$

where t and b are the plate thickness and width, respectively, whereas α and β are the tension-field angle and the diagonal angle of the steel plate measured from the vertical direction, respectively. Equation (1) is based on an elastic strain energy formulation. An alternative model is the strip one, which can be sometimes very difficult to be implemented in the used software. In this model, the tension-field angle α is given by

$$\tan^4 \alpha = \frac{1 + \frac{t b}{2 A_c}}{1 + t d \left(\frac{1}{A_b} + \frac{d^3}{360 I_c b} \right)} \quad (2)$$

where A_c and I_c are the cross-section area and the second moment of area of the surrounding columns, respectively; A_b is the beam cross-section area and d is the panel height (Timler & Kulak, 1983). The Canadian code (CSA, 2001) provides the following minimum second moment of area I_c of columns adjoining SPSWs to prevent their excessive deformation, leading to premature buckling, under the pulling action of the plates:

$$I_c \geq \frac{0.00307 t d^4}{b} \quad (3)$$

Any contribution offered from the plate buckled in compression can be neglected. In this condition, for width-to-height ratios between 0.8 and 2.5, the inclination of the generated tension field can be directly assumed to be 45°.

When the equivalent scheme is subject to an initial shear load V , a horizontal displacement δ_r occurs at the top (see Figure 4). By simple analytical steps, the elongation ΔL_d and tensile force N in the equivalent diagonal can be evaluated through the following relationships:

$$\Delta L_d = \delta_r \sin\beta \quad (4)$$

$$N = E_d A_d \Delta L_d / L_d = V / \sin\beta \quad (5)$$

where in Equation (5), the terms E_d and L_d are the Young modulus of the plate material and the diagonal length, respectively.

Figure 4. Schematization of the equivalent tensile diagonal model and nomenclature of geometrical parameters considered in Equations (4) and (5).

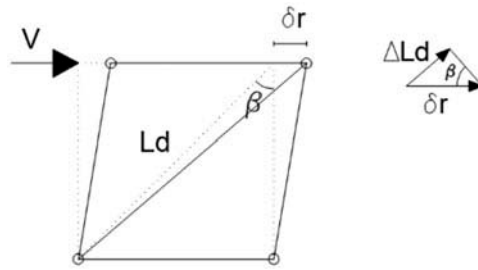
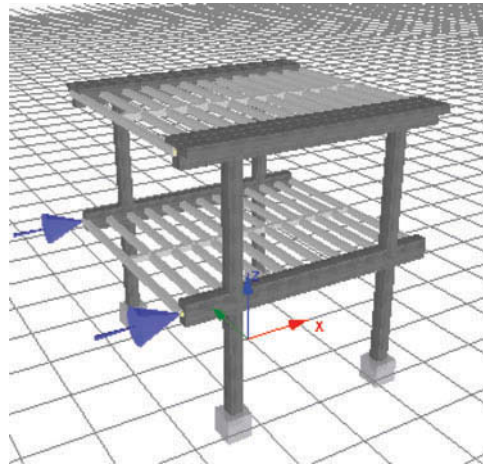


Figure 5. Numerical model of the substructure no. 5 set-up with the SeismoStruct software.



According to Sabouri-Ghomi et al. (2005), the behaviour of thin plates in pinned joint frames can be schematised through an elastic-perfectly plastic bilinear behaviour, where both the shear strength F_{py} and initial stiffness K_{pw} of the panel can be evaluated as follows:

$$F_{py} = \frac{C_{m1}}{2} \sigma_{ty} b t \sin 2\vartheta \quad (6)$$

$$K_{pw} = \frac{\frac{C_{m1}}{2} \sigma_{ty} \sin 2\vartheta b t}{\frac{2 C_{m2} \sigma_{ty}}{E \sin 2\vartheta}} \frac{1}{d} \quad (7)$$

In Equations (6) and (7), d , b and t are the terms already introduced, E and G are the normal and shear moduli of the metal plate, σ_{ty} is the tension-field stress in the yielded plate, ϑ is the tension-field angle, measured from the vertical direction, and C_{m1} and C_{m2} are modification factors, taking into account beam-to-column connections, plate-to-frame connections and the effect of both flexural behaviour and stiffness of boundary elements. Such modification factors are limited as follows: $0.8 < C_{m1} < 1.0$ and $1.0 < C_{m2} < 1.7$, but Sabouri-Ghomi et al. (2005) recognised that these values will need further refinement as more test results will become available in the future. These values can be obtained from the numerical calibration of experimental tests, as proposed by Formisano et al. (2016), where an example of a useful analytical tool for the estimation of these factors is proposed.

The forecast behaviour of the panel can be implemented by assigning at the fictitious material of the equivalent diagonal yielding strength $\sigma_{y,d}$ and Young modulus E_d evaluated, respectively, through the following relationships:

$$\sigma_{y,d} = \frac{F_{wy}}{A_d \sin \beta} \quad (8)$$

$$E_d = \frac{K_w L_d}{A_d \sin^2 \beta} \tag{9}$$

In order to set-up a valuable FEM model of metal panels in SeismoStruct, the behaviour of the bare RC structure of Bagnoli has been first calibrated. RC beams and columns have been modelled by *infrmFB* elements, while the floor has been modelled by *elfrm* beams having same stiffness and weight of the effective floor. A three-dimensional view of the modelled substructure is illustrated in Figure 5.

A reduced Young modulus E_c for RC beams and columns has been adopted for taking into account the environmental degradation effect associated to the construction age. In particular, $0.5 E_c$ and $0.4 E_c$ have been adopted for beams and columns, respectively. Degradation zones extended at lengths of 35 and 65 cm from the ends of beams and columns, respectively, have been assumed. The experimental-to-numerical modal comparison achieved with these assumptions is shown in Table 2.

Subsequently, $0.3 E_c$ and a reduced strength have been assumed for the edge columns to consider the damages in these zones caused by the experimental pull-out test previously carried out in the transversal direction on the same structure upgraded with shape memory alloy bracings (Mazzolani, 2008).

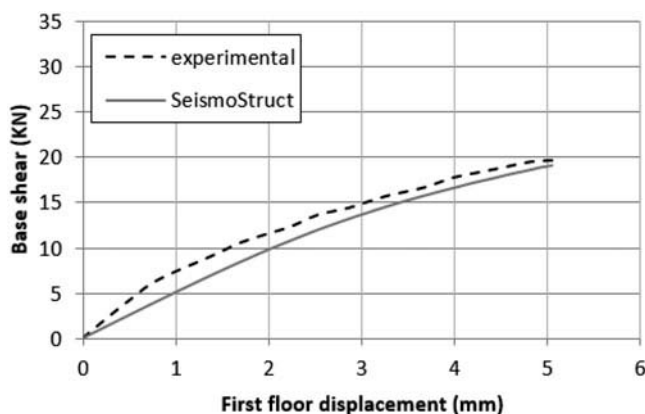
In Figure 6, both the experimental curve and the final numerical one based on the RC bare structure experimental stiffness, the latter introducing the previously mentioned reduction coefficients, are shown.

Once the initial structure behaviour has been calibrated, the steel shear walls have been modelled in SeismoStruct. The steel frame members have been modelled by *elfrm* elements to remain in the elastic range under the forces applied by SPSWs. The steel frame hinges have been modelled by *link* elements with translational stiffness infinitely greater than rotational one. Finally, the wall-to-RC beam connections have been modelled by means of rigid links.

Table 2. Experimental-to-numerical comparison of the module no. 5 vibration periods

Mode	1	2	3	4	5	6
Experimental period(s)	0.625	0.556	0.455	0.208	0.186	0.147
Numerical period(s)	0.639	0.505	0.428	0.201	0.191	0.152

Figure 6. Comparison between the experimental curve and the final numerical one of the RC bare structure.



Each of six panel fields, having dimensions of 600 × 400 mm and being separated to each other by means of horizontal stiffeners, has been numerically represented with an equivalent diagonal, as previously described (see Figure 7).

The equivalent tensile diagonal has been modelled by a *truss* element with an elastic–plastic material behaviour, starting from the shear strength F_{wy} and initial stiffness K_w of the wall estimated as follows:

$$F_{wy} = \frac{C_{m1}}{2} \sigma_{ty} b t \sin 2\vartheta \quad (10)$$

$$K_w = \frac{\frac{C_{m1}}{2} \sigma_{ty} \sin 2\vartheta b t}{\frac{2 C_p \sigma_{ty}}{E \sin 2\vartheta} d} \quad (11)$$

where C_{m1} and C_p are modification factors, taking into account both the plate behaviour and the wall flexural effect, that should be properly calibrated (Sabouri-Ghomi et al., 2005).

The calibration of the wall model in SeismoStruct has been performed by deriving the force–displacement curve of the only-walls contribution. In fact, knowing the force–displacement curve of the retrofitted structure and the calibrated curve of the initial structure, the latter furtherly pushed up to the same ultimate displacement of the experimental test, the only-walls contribution can be simply derived by subtraction. By adopting the values of 1.0 and 5.4 for C_{m1} and C_p , respectively, the experimental curve appears to be well simulated by the numerical one (see Figure 8). The same comparison could be also done for the aluminium panels solution but, with the damages occurred after the test on steel panels, a further calibration of results is needed, representing a future development of the study.

4. Application to a case study

The benefits arising from the use of perforated steel panels instead of traditional full ones are already known (Formisano et al., 2016; Purba & Bruneau, 2007). However, few studies on existing RC buildings retrofitted with such devices are available, whereas the current applications deal with either common SPSWs (De Matteis et al., 2009; Formisano et al., 2010) or Buckling Restrained Braces (Di Sarno & Manfredi, 2010, 2012). Therefore, in this paper, an existing building has been retrofitted with either traditional panels or perforated ones aiming at showing the different advantages deriving from their use (Formisano & Lombardi, 2015; Formisano & Sahoo, 2015). The case study is a residential multistorey RC building in Torre del Greco (district of Naples, Italy), representative of the typical 1960s and 1970s constructions designed for gravity loads only. The building under investigation develops on five storeys with rectangular shape of dimensions

Figure 7. Calculation scheme of the SPSW (a) and SeismoStruct numerical model of the retrofitted substructure no. 5 (b).

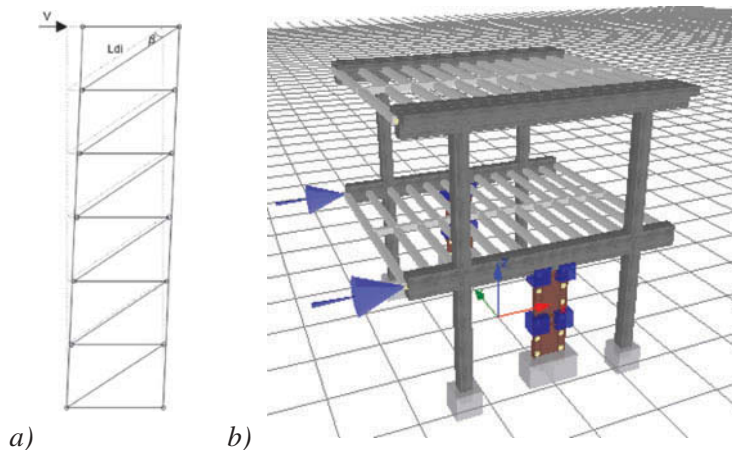
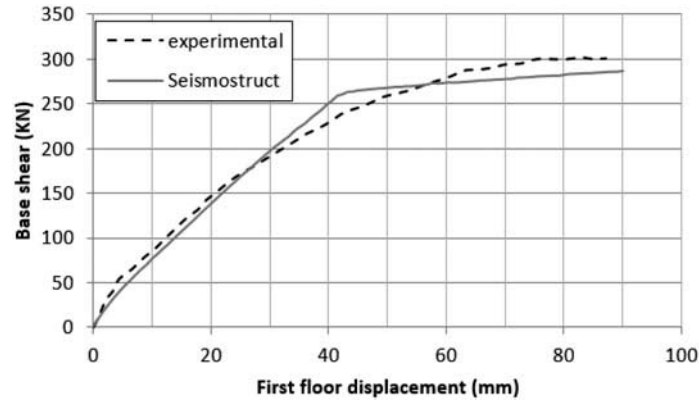


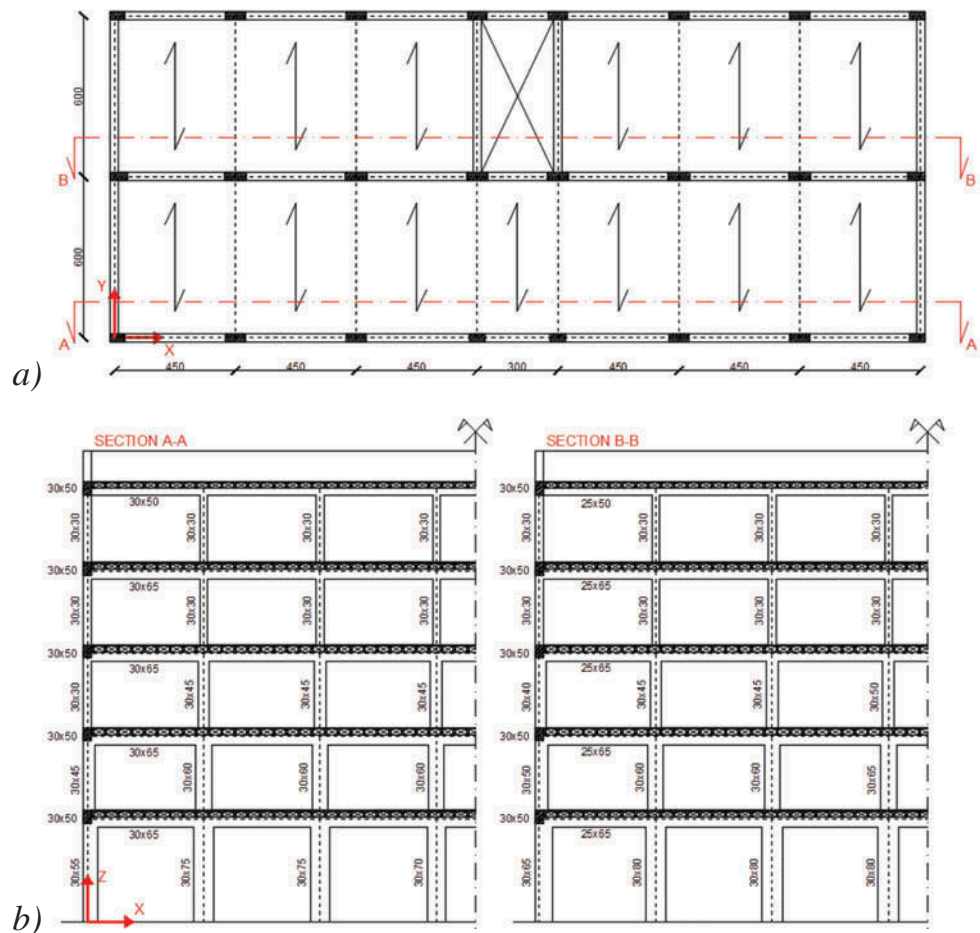
Figure 8. Comparison between the experimental curve and the calibrated numerical one for the structure retrofitted with steel panels.



30 × 12 m (see Figure 9). It has two bays in the transversal direction and seven bays in the longitudinal one. The ground floor, dedicated to commercial activities, has height of 4.0 m, while the height of other floors is 3.2 m. The building total height is 16.8 m, without considering the summit parapet.

Seismic-resistant frames are placed in the longitudinal direction only. They are connected to each other in the transversal direction from both the slab and the edge beams only. The staircase

Figure 9. Existing building under investigation: typical plan layout (a) and vertical sections (b).



is located in the building central position and it is made of 30 × 60-cm knee beams. Floors are made of RC—hollow tiles mixed slabs having depth of 28 and 24 cm at the intermediate and top levels, respectively.

In absence of a specific documentation on carpentries, the elements sizes have been detected from *in-situ* inspections, whereas the reinforcement details have been deduced from an appropriate simulated design (R.D.L. n. 2229, 1939). According to the materials used at that construction time, R_{cm}180 concrete and Aq50 Italian steel ($f_{ym} = 270$ MPa and $f_{um} = 550$ MPa) have been considered. In order to take into account the presence of a cracking state of the structural members, according to EC8, a 50% reduced Young modulus has been assumed for both beams and columns.

The building is located on a soil type C and it is subjected to a peak ground acceleration $a_g S$ of 0.28 g associated to an elastic response spectrum with a 975-year return period at the life safety limit state.

The three-dimensional view of the examined structure modelled with the SeismoStruct software is illustrated in Figure 10.

From the modal analysis, whose results are depicted in Table 3 and Figure 11, the building has shown a high deformability, especially in the transversal direction, due to the lack of frames.

From the pushover analyses on the initial structure, as shown in Figure 12 where also the performance points in both analysis directions are individuated, it appears that in the longitudinal direction, the demand is particularly focused between the third and the fourth floors, where the variation of elevation stiffness is very high (see Table 4). On the other hand, in the transversal direction, the failure is essentially caused by the staircase column collapse.

The seismic upgrading of the above RC building by means of full MPSWs, which are known to give to the structure a significant contribution in terms of initial stiffness, shear strength and dissipated energy (Formisano, De Matteis, Panico, Calderoni, & Mazzolani, 2006; Formisano, De Matteis, Panico, & Mazzolani, 2008), has been developed on the basis of the US procedures (Applied Technology Council (ATC)-40, 1996, Federal Emergency Management Agency(FEMA)-273, 1997).

Figure 10. Numerical model of the investigated 5-storey RC-existing building.

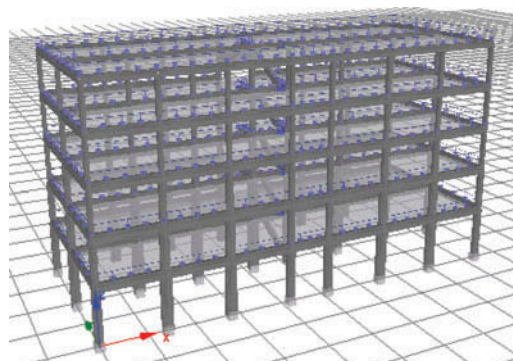


Table 3. Modal analysis results

Mode	1 (U_y)	2 (R_z)	3 (U_x)
Period(s)	1.70	1.40	0.95
Participating mass (%)	84	78	70

Figure 11. Deformed shape of the building under pushover analysis in directions X (a) and Y (b) (amplified deformation factor equal to 50).

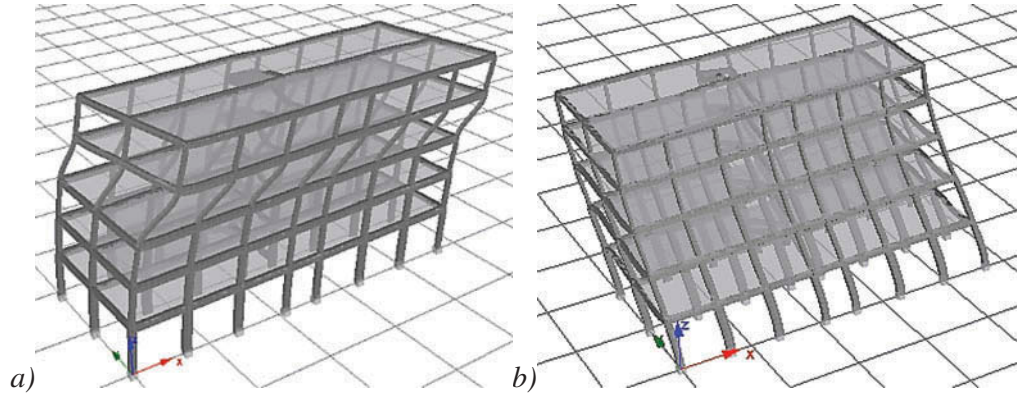


Figure 12. Capacity curves and performance points of the initial structure in directions X (a) and Y (b).

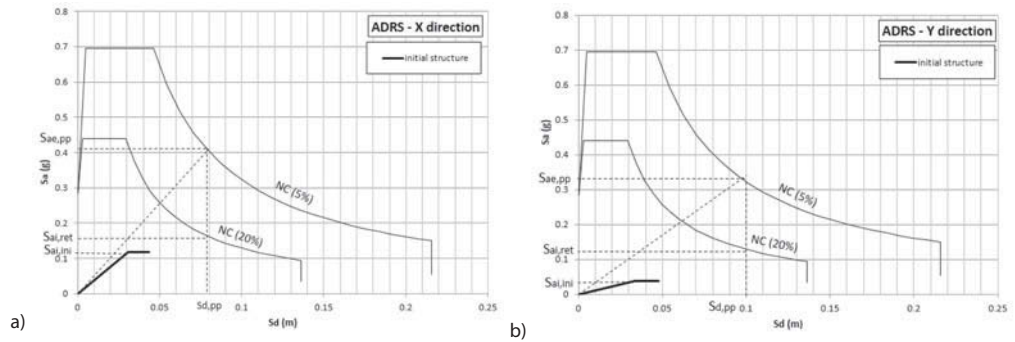


Table 4. Regularity analysis of the structure

Floor	Seismic mass (t)	Mass variation (%)	Direction X		Direction Y	
			Lateral stiffness (kN/m)	Lateral stiffness variation (%)	Lateral stiffness (kN/m)	Lateral stiffness variation (%)
5	353	-25	87,877	-	31,419	-35
4	473	-	87,760	-51	48,290	-21
3	474	-1	180,779	-38	60,844	-14
2	478	-4	293,620	-26	70,666	-21
1	499	-	397,521	-	89,464	-

Following a performance-based design approach, which aims at increasing the overall lateral stiffness of the initial structure, the procedure involves the choice of a target spectral displacement of the retrofitted structure, $S_{d,pp}$, corresponding to a given performance level (operational, immediate occupancy, life safety and near collapse). Once the seismic hazard parameters are known, the elastic spectral acceleration $S_{ae,pp}$ is determined from the ADRS (acceleration-displacement response spectrum) format. So, the target period T_{ret} and the target stiffness K_{ret} of the retrofitted structure are calculated from Equations (12) and (13), respectively. In particular, in Equation (13), the term T_{ini} is the fundamental period of the initial structure. After defining the performance points of the retrofitted structure, the stiffness contribution K_w provided by the walls is determined from Equation (14), where the term K_{ini} is the initial structure stiffness.

$$T_{ret} = 2\pi \sqrt{\frac{S_{d,pp}}{S_{ae,pp}}} \tag{12}$$

$$K_{ret} = K_{ini} \left(\frac{T_{ini}}{T_{ret}} \right)^2 \tag{13}$$

$$K_w = K_{ret} - K_{ini} \tag{14}$$

Considering that the retrofitted structure is able to provide at least the same damping level of the bare structure, the target shear strength of the retrofitted structure V_{ret} is obtained from Equation (15), where V_{ini} and $S_{ai,ini}$ are the shear strength and the inelastic spectral acceleration of the initial structure, respectively, and $S_{ai,ret}$ is the retrofitted structure inelastic spectral acceleration. Finally, the contribution in terms of shear strength V_w given by walls is evaluated through Equation (16).

$$V_{ret} = V_{ini} \frac{S_{ai,ret}}{S_{ai,ini}} \tag{15}$$

$$V_w = V_{ret} - V_{ini} \tag{16}$$

In Figure 13, the response spectrum is plotted in the ADRS plane, considering the spectral acceleration reduction obtained with a damping equal to 20%.

Once the required stiffness and strength of the panels have been determined, their preliminary design is developed. In analogy with Formisano and Sahoo (2015), an upgrading system with partial-bay MPSWs, arranged in one and two pairs along directions X and Y, respectively, has been designed (see Figure 13). The MPSWs disposition has been dictated from both the necessity to reduce as much as possible the interruption of building activities and to respect architectural requirements.

In order to respect the optimal panel shape ratio (Formisano et al., 2007) and considering the building inter-story height, the width B_w of MPSWs has been chosen equal to 1.65 m, while its depth has been divided in two equal parts by means of an intermediate steel beam within the external frame. In particular, the design of metal walls has been carried out initially considering full S235 steel plates (see Tables 5 and 6).

The plate thicknesses have been first derived by reversing Equations (10) and (11) and by assuming C_{m1} and C_{m2} equal to 1.0 and 1.7, respectively, and $C_{p,x}$ and $C_{p,y}$ equal to 8.5 and 13.6, respectively. These modification factors are obtained by iterations in order to fit the numerical behaviour of the retrofitted structure to the design requirements both in terms of global strength and stiffness. Logically speaking, these values should be obtained from experimental tests on designed shear walls. Since the stiffness-based design implies greater thicknesses than the

Figure 13. Location of MPSWs (a) and details of the external frame (b).

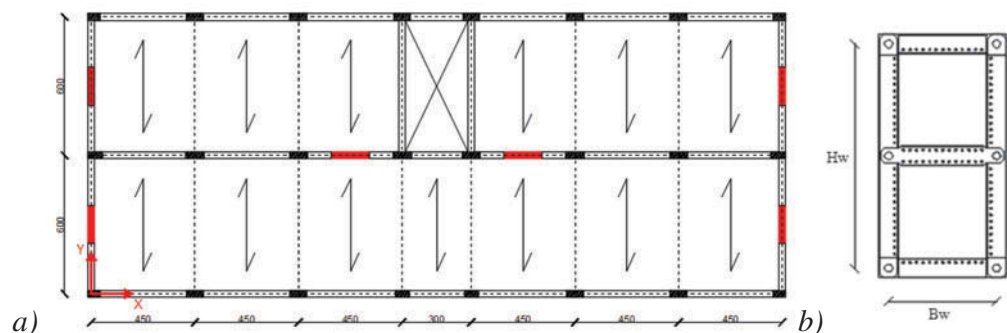


Table 5. Thicknesses of full SPSWs derived from the strength-based design

Floor	C_{m1}	B_p (mm)	$V_{pi,x}$ (kN)	$n_{p,x}$	$t_{p,x}$ (mm)	$V_{pi,y}$ (kN)		$t_{p,y}$ (mm)
5	1.0	1,650	223	2	0.58	475	4	0.61
4		1,650	465	2	1.20	989	4	1.27
3		1,650	651	2	1.68	1,383	4	1.78
2		1,650	780	2	2.01	1,658	4	2.14
1		1,650	856	2	2.21	1,818	4	2.34

Table 6. Thicknesses of full SPSWs derived from the stiffness-based design

Floor	E (MPa)	$K_{p,x}$ (kN/m)	$C_{p,x}$	$H_{p,x}$ (mm)	$t_{p,x}$ (mm)	$K_{p,y}$ (kN/m)	$n_{p,y}$	$H_{p,y}$ (mm)	$t_{p,y}$ (mm)
5	200,000	14,991	8.5	2,300	1.78	31,850	13.6	2,400	3.15
4				2,250	1.74			2,400	3.15
3				2,250	1.74			2,400	3.15
2				2,250	1.74			2,400	3.15
1				3,375	2.61			3,450	4.53

strength based one, the values from the former design process have been considered, they being subsequently replaced by the most common commercial thicknesses. Subsequently, the equivalent bracing behaviour has been obtained by means of Equations (8) and (9) for its implementation in numerical analyses.

Assuming to guarantee the same lateral stiffness level of the full SPSWs, the study has been extended by considering full LYS and aluminium plates, as already done in De Matteis, Formisano, Mazzolani and Panico (2005), as well as perforated S235 steel panels.

Table 7 shows the mechanical properties of all the metallic materials considered in the retrofit design.

Two drilling configurations have been proposed for perforated SPSWs. The first solution is characterized by plates with 36 holes having diameter of 160 mm and hole percentage ρ (intended as the ratio between the holes area A_{holes} and the panel one A_{sup}) equal to 40%, while the second solution has 36 holes having diameter of 190 mm and ρ equal to 60%. The behaviour of the perforated panels has been implemented in the FEM model by adopting a linear reduction of the modification factors in comparison to those used for full panels (Formisano et al., 2016). The modification factor values for C_{m1} and C_{m2} assumed in this paper are equal to 0.40 and 0.70, respectively, for $\rho = 40\%$, and 0.20 and 0.40, respectively, $\rho = 60\%$.

Table 7. Metallic materials considered for the shear walls design

Material	f_y (MPa)	f_u (MPa)	ϵ_u (%)	E (MPa)
Steel	235	360	35	200,000
LYS	86*	236	50	200,000
AW 1050A	21*	80	45	70,000

*Conventional yielding strength at 0.2% strain level.

Table 8 shows the commercial thicknesses of plates assumed in the following analyses. Due to the different Young moduli, aluminium plates thicker than steel ones have been considered in order to have comparable results in terms of global stiffness of the retrofitted structures.

The steel frame surrounding SPSWs has been designed to both possess an adequate stiffness and remain in the elastic field when the plates are subjected to significant plastic strains. This outcome is achieved for full panels by both using the Equation (3) and verifying the elements in terms of strength, under the actions induced by the tension-field mechanism (Bhowmick et al., 2014). The coupled UPN profiles made of S275 steel have been obtained from this procedure for the examined cases of shear walls (see Table 9).

Moreover, in order to transfer the actions to the walls, the RC beams have been reinforced, analogously to the intervention of Figure 3, by means of two S275 steel-coupled UPN profiles fixed to the RC beams by means of steel bolts (see Table 10).

The analysis results of the retrofitted structures have shown that due to the failure of existing columns, further interventions are necessary for other RC members in order to

Table 8. The used commercial thicknesses of panels

Floor	Steel plates		AW1050A plates	
	$t_{p,x}$ (mm)	$t_{p,y}$ (mm)	$t_{p,x}$ (mm)	$t_{p,y}$ (mm)
1	1.80	4.00	4.00	7.00
2	1.80	4.00	4.00	7.00
3	1.80	4.00	4.00	7.00
4	1.80	4.00	4.00	7.00
5	3.00	5.00	6.00	10.00

Table 9. The assumed steel frame members

Floor	Full SPSWs		Perf. (40%) SPSWs and LYS-PSWs		Perf. (60%) SPSWs and AW1050A-PSW	
	Dir. X	Dir. Y	Dir. X	Dir. Y	Dir. X	Dir. Y
5	2 × UPN160	2 × UPN240	2 × UPN120	2 × UPN180	2 × UPN120	2 × UPN120
4	2 × UPN160	2 × UPN240	2 × UPN120	2 × UPN180	2 × UPN120	2 × UPN120
3	2 × UPN160	2 × UPN240	2 × UPN120	2 × UPN180	2 × UPN120	2 × UPN120
2	2 × UPN160	2 × UPN240	2 × UPN120	2 × UPN180	2 × UPN120	2 × UPN120
1	2 × UPN260	2 × UPN320	2 × UPN160	2 × UPN220	2 × UPN120	2 × UPN160

Table 10. The assumed steel members for the strengthening of RC beams

Floor	Full SPSWs	Perf. (40%) SPSWs and LYS-PSWs	Perf. (60%) SPSWs and AW1050A-PSW
	Dir. X–Y	Dir. X–Y	Dir. X–Y
5	2 × UPN260	2 × UPN240	2 × UPN220
4	2 × UPN260	2 × UPN240	2 × UPN220
3	2 × UPN260	2 × UPN240	2 × UPN220
2	2 × UPN260	2 × UPN240	2 × UPN220
1	2 × UPN300	2 × UPN280	2 × UPN260

achieve the target displacements. Therefore, the retrofit project has been completed with RC jacketing of (1) longitudinal perimeter columns at the third and fourth floors, (2) transversal perimeter columns from the second floor to the fourth one and (3) the staircase columns up to the fourth floor. Furthermore, jacketing with steel profiles has been considered for members exhibiting brittle failure due to shear. These additional interventions on the existing members have been designed to ensure the expected performance of the structure up to the target displacements.

Figure 14 shows the results obtained from the pushover analyses on the structure equipped with the mentioned solutions.

The results show that the shear strength of the structure retrofitted with full SPSWs is clearly higher than the other solutions. As a negative consequence, the greater actions induced by the full SPSWs on the RC structure have requested the design of additional local retrofitting interventions.

Also for the other shear panel solutions, additional interventions on the main RC structure have been foreseen, but they have been more economic than those required by using full SPSWs. In particular, although the solutions based on plates with low yield strength materials

Figure 14. Capacity curves of initial and retrofitted structures in directions X (a) and Y (b).

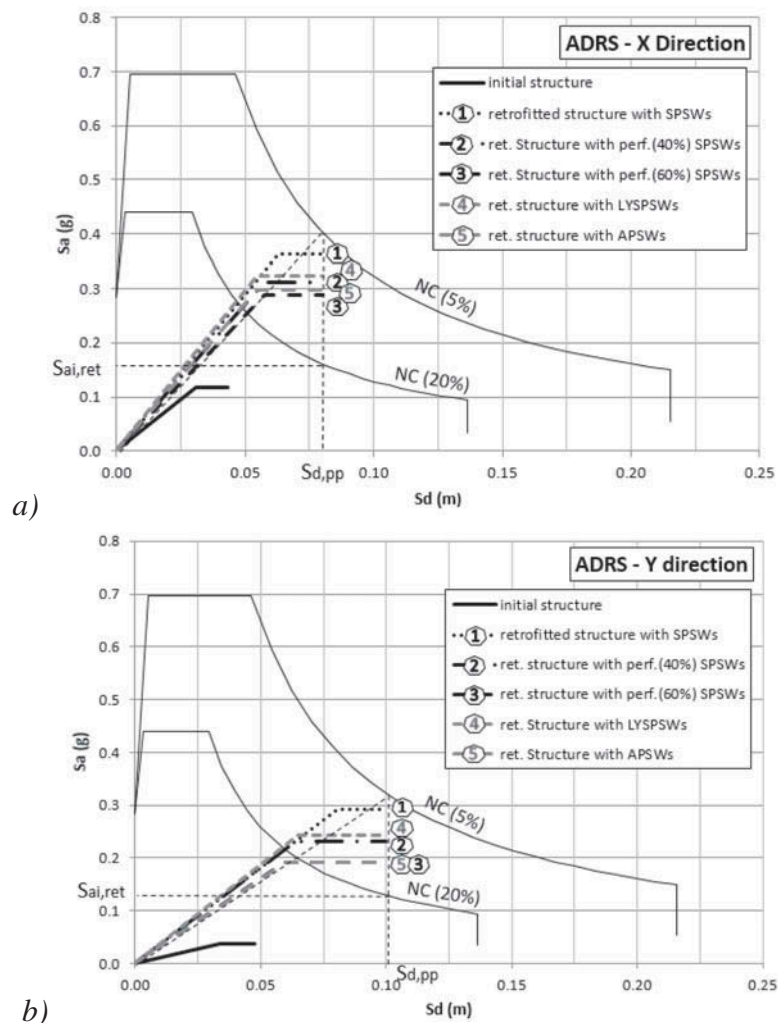


Table 11. Economic comparison among examined solutions

Wall type	Plates (€)	Perimetral steel frame (€)	Local interventions (€)	Total (€)
Full SPSWs	14,200	36,000	72,000	122,200
Perf. (40%) SPSWs	15,100	22,500	68,400	106,000
Perf. (60%) SPSWs	15,100	17,800	64,100	97,000
LYS-PSW	19,900	22,500	68,400	110,800
AW1050A-PSW	37,100	17,800	64,100	119,000

(i.e. low yield steel and pure aluminium) seem to be comparable with those based on perforated plates made of traditional steels in terms of performances, the differences are noticed from the economic point of view (see Table 11). In fact, considering the current Italian costs of both steel elements and local reinforcing interventions, a cost saving of about 16% and 27% has been, respectively, estimated for the less drilled and the more drilled perforated SPSWs with respect to the installation of full SPSWs. This confirms the benefits deriving from the use of perforated SPSWs.

5. Conclusions

In this paper, a study aimed to show the benefits of using perforated SPSWs has been carried out. The use of such systems, already known in literature for applications into new steel structures, can be particularly advantageous for retrofitting existing buildings designed without seismic criteria, although the current Eurocodes do not provide any indications about this issue.

When referred to existing RC structures, the use of traditional full SPSWs may involve the transfer of excessive stresses on the boundary members induced by the plate tension-field mechanism. Such stresses can lead to the design of massive local strengthening interventions, which are very often economically inconvenient.

Starting from these premises, in the first part of the paper, the availability of recent experimental test results on a real RC building retrofitted with SPSWs has allowed both to calibrate and validate a simple FEM model developed by the SeismoStruct software.

Subsequently, the case study of an existing multistorey RC building retrofitted either with full MPSWs (i.e. traditional steel, LYS and pure aluminium plates) or perforated SPSWs has been numerically analysed in the static non-linear field. The analysis results have shown that perforated SPSWs with drilling percentages of 40% and 60% provide cost savings in the retrofit design of about 16% and 27%, respectively, compared to the cost deriving from using full plates. By increasing properly the drilling configuration, a significant shear strength reduction is achieved without excessively compromise both the stiffness and the ductility of the retrofitted structure. In fact, by choosing an appropriate drilling pattern, it is possible to reach large drifts without fractures around the holes, which could decrease the shear capacity of panel devices. The main benefit deriving from the use of perforated plates is to choose a priori the shear strength they offer on the basis of a given drilling configuration, according to the design requirements, without changing the geometric dimensions of the walls, which sometimes represent architectural requirements impossible to be modified.

Although perforated SPSWs made of common steel plates can be a viable alternative to other stiffening solutions based on metals more expensive (aluminium) and not available on the European market (LYS), further experimental tests are necessary for the validation of modification factors to be used in the used retrofitting design formulas.

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