Perforated Shear Panels for Seismic Rehabilitation of Existing Reinforced Concrete Buildings

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Perforated Shear Panels for Seismic Rehabilitation of Existing Reinforced Concrete 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 appropriate for the intervention of seismic retrofitting of existing reinforced concrete (RC) buildings designed for gravity loads only. Some past research has shown that, when traditional full SPSWs are used as bracing devices for framed buildings, they may induce excessive design forces to the surrounding frame members. Therefore, low yield steel could be a valuable option to overcome this applicability limit. Nevertheless, the scarce availability in the market of these steels suggests the employment of aluminium alloys and perforated steel plates, which have the benefit of incurring behaviour in the plastic range for low stress levels. In this paper, in order to conduct a parametric analysis concerning the use of full and perforated SPSWs for seismic upgrading of existing RC framed structures, first some experimental tests have been numerically calibrated using the SeismoStruct software. Subsequently, the proposed finite element model has been used to design the retrofitting systems with either full or perforated SPSWs of an existing RC residential five-storey building. Finally, the differences in the use of these solutions, in terms of both structural and economic viewpoints, have been demonstrated.

Keywords: steel plate shear walls, perforated plates, bracing devices, tension-field, reinforced concrete buildings, retrofitting, finite element, modelling, parametric analysis.

1 Introduction

Steel Plate Shear Walls (SPSWs) represent an effective passive control system. They are characterized by a very stable hysteretic response up to large deformations, by both high initial stiffness and strength. 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 with RC shear walls. In addition, by using shop-welded or bolted connection types, the erection process can be eased, allowing a considerable reduction in constructional costs. Application examples of such devices in new steel buildings, with either bracing or dissipative functions, are detected in Asia and America [1]. 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 on them a considerable increase in performance [2]. The beneficial contribution offered by shear panels is guaranteed by the development of a diagonal tensile band mechanism (called a *tension-field*), which is more effective the greater the plate area involved in the deformation process [3]. In particular, when traditional full systems, configured as simple steel panels without stiffeners, are employed, the optimal behaviour is guaranteed as with plates having width/height ratios between 0.8 and 2.5 [4].

Some studies have shown that when full SPSWs are used as bracing devices for framed buildings, they may introduce a problem in the capacity design application, so as to result in excessive design forces to the surrounding frame members, thus increasing their required size and costs [5]. The scarce availability in the market of Low Yield Steel (LYS), usually used to limit the forces transmitted by the plates on steel frame members, suggests the employment of aluminium alloys [6, 7] and perforated steel plates [8], which have the benefit of experiencing behaviour in the plastic range for low stress levels. A recent study by the authors has shown the suitability of such panels for seismic-resistant applications through the setup of an easy design tool useful for their application [9].

In this paper, in order to conduct a parametric analysis concerning the use of full and perforated SPSWs for seismic upgrading of an existing RC framed structure, the experimental test results conducted in Bagnoli (Naples) [6, 7] have been numerically calibrated on the basis of the finite element software SeismoStruct [10]. With this software, the SPSW model has been implemented with the equivalent tensile diagonal one proposed by Thorburn et al. [11]. The excellent experimental-to-numerical correspondence of the results, validating the proposed model, has allowed its application for a subsequent analysis on an existing residential five-storey RC building in Torre del Greco (Naples) equipped with SPSWs. Following the same design approach reported in [2], push-over analyses of the retrofitted structure with full and differently-perforated SPSWs have been performed. Finally, the structural and economic differences between these two solutions have been exposed and critically discussed.

2 Experimental study

In order to both study the behaviour of existing RC buildings retrofitted by perforated SPSWs and to validate the model proposed in SeismoStruct [10], the ILVA-IDEM project [12] has been considered. In this project, the retrofitting of an industrial building located in Bagnoli (Naples) by using various reinforcement systems, including the aforesaid panels, was carried out. The project was started by the availability of a real building in the ex Italsider area destined to be demolished (see Figure 1a). This building was representative of a construction type common in 1960s and 1970s designed for gravity loads only.

The construction is characterized by a rectangular plan with dimensions of 41.60x6.50 m and two floors with heights on the ground of 3.30 m and 6.55 m, respectively. It has a single bay in the transverse direction and twelve bays in the longitudinal one (see Figure 1c). Thanks to the regular configuration of the structure, once cladding and internal walls were eliminated, the building was divided in six test modules. Different upgrading techniques were tested; in particular, the module n°5 was chosen to test the metal shear panels system (see Figure 1b).

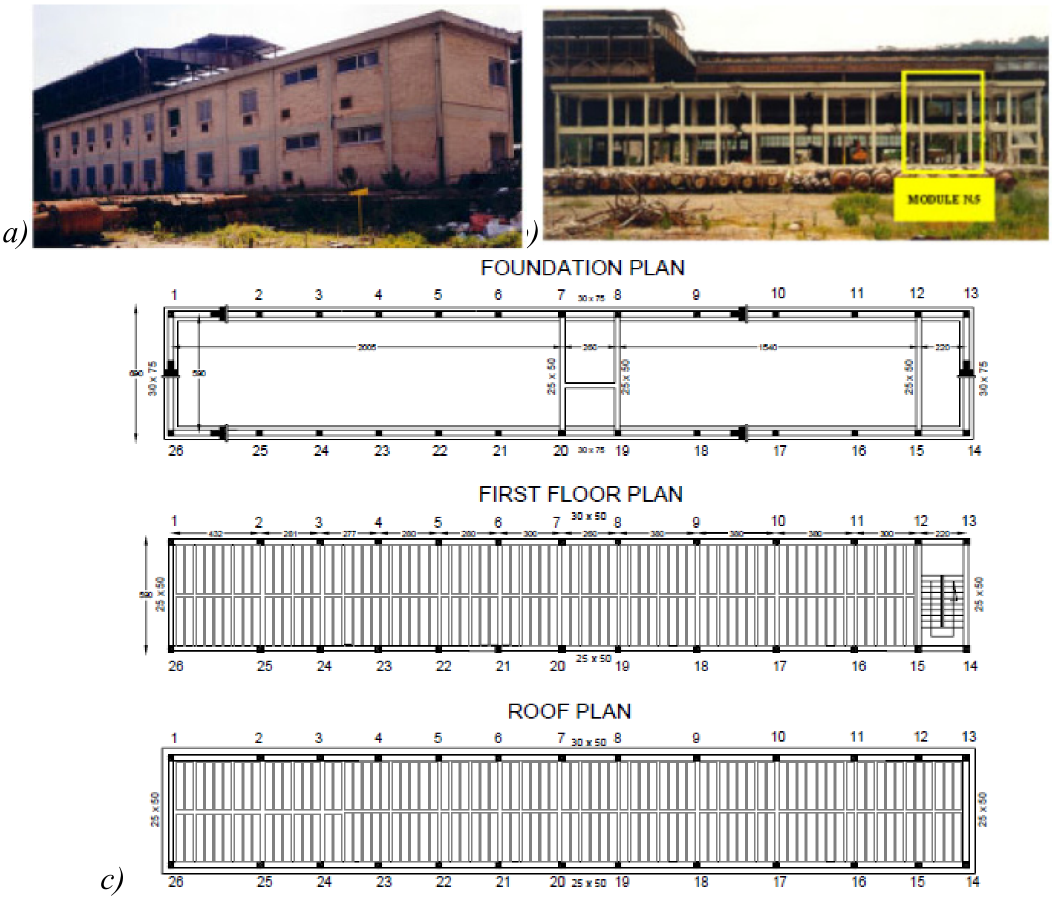


Figure 1: View of the original RC building (a), its division in sub-structures (b) and plan configurations at different levels (c).

The geometrical configuration of this sub-structure is characterized by a rectangular plan with dimensions of 6.30x5.90 m and two 3.30 m high levels (see Figure 2). The slab thickness is equal to 24 cm and 20 cm at the first and second floor, respectively. No complementary elements are located at the first floor, excluding a plaster layer, having a thickness of 4 cm, located on the inner side of the slab. At the roof floor, both a 5 cm thick slope slab, realised with sand and mortar, and waterproofing layers are located. Both slabs have a middle transversal floor beam and are supported by emergent rectangular beams with sections of 30x50 cm and 25x50 cm. These beams, placed along the longitudinal direction at the first level, are reinforced at the top with 2ϕ8 bars, at the bottom with 2ϕ8+3ϕ12 bars, laterally with 2ϕ8 bars and transversally with ϕ8/200 mm stirrups. The beams at the second level have a *T* cross-section with the same height, width and steel reinforcement of the first level one. In the transverse direction, the lateral resistance is mainly provided by the columns, which have square cross-sections of 30x30 cm. Columns are reinforced by 4ϕ12 longitudinal bars located in the corners and are transversally constrained by ϕ8/300 mm stirrups. The main mechanical properties of both concrete and steel rebars were determined by means of laboratory tests carried out on specimens directly extracted from the existing structural members. From compression tests, the concrete had an average compressive strength of 21 MPa, an average elastic normal modulus of 16829 MPa and an average specific weight equal to 2239 Kg/m^3. From tensile tests, the steel rebars had an average yield and ultimate strengths of 443 MPa and 693 MPa, respectively.

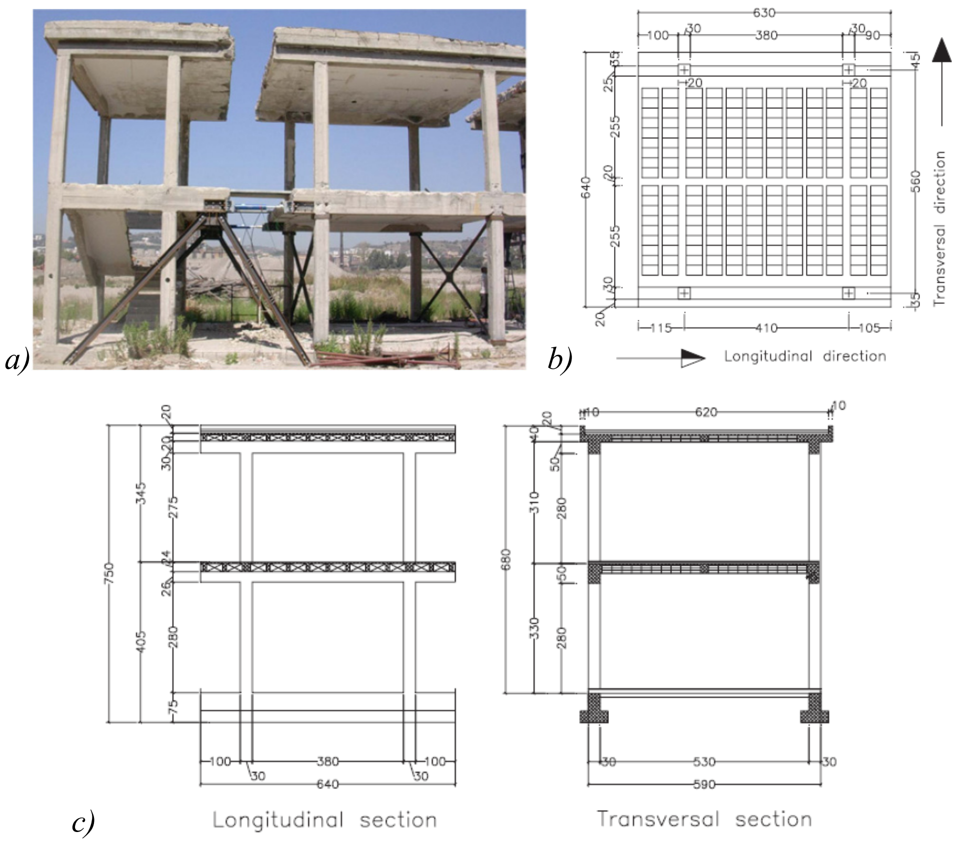


Figure 2: Test set-up of the module n°5 by using module n°6 as retaining structure (a), first floor carpentry (b) and vertical sections (c).

Before testing the structure, impacting hammer dynamic tests were made to measure its vibration frequencies. From these tests the parameters in Table 1 were achieved. This activity is essential to allow for the calibration of the finite element models. In fact, the results obtained from the structure theoretical models cannot interpret well the real behaviour when both structural damage phenomena and deterioration of the materials' mechanical features appear.

Mode	1	2	3	4	5	6
Period (s)	0.625	0.556	0.455	0.208	0.186	0.147

Table 1: Experimental periods of the module n°5 obtained from hammering tests.

A pull-out test in the transversal direction was performed to evaluate the structure stiffness. Before this test, in order to prevent possible structure torsional effects, two steel X-bracing were placed at the ground floor along the direction transversal to the applied load. After, the structure was cyclically tested at low displacements to derive its initial state properties (see Figure 3).

a) 

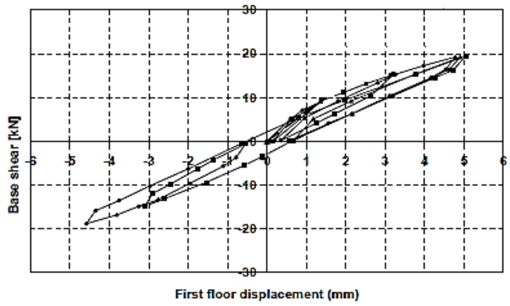
b) 

Figure 3: Pull-out test setup (a) and experimental load-displacement curve (b).

The seismic retrofitting of the RC module was designed following the performance-based design approach according to the US guidelines [13, 14]. Based on a non-linear analysis, the behaviour of the initial RC structure under lateral actions was evaluated according to the results of the performed preliminary test. The structure is located on a soil type B having a peak ground acceleration equal to 0.25g [15]. After fixing a target design displacement of the first level of the RC structure under collapse conditions, corresponding to an inter-storey drift of about 1%, and by assuming a viscous damping coefficient equal to 20%, the design parameters were deduced. With the same design data in terms of both stiffness and strength used for the retrofitted structure, two retrofitting solutions, the first with steel plates and the second with heat treated pure aluminium plates, were defined. The properties of the materials of the devices used obtained from experimental tests are reported in Table 2.

Material	f_y (MPa)	f_u (MPa)	ε_u	E (MPa)	Strain hardening factor
Steel	305	340	32%	200000	1.15
Pure aluminium EN-AW 1050A	*21	80	45%	70000	3.76

*conventional yielding strength at 0.2% strain level.

Table 2: Mechanical properties of the plates obtained from experimental tests.

Steel plates with dimensions of 600x2400x1.15 mm and aluminium plates with dimensions of 600x2400x5.00 mm were chosen for the experimental tests. Since the Canadian code [16] suggests the use of plate width/height ratios between 0.8 and 2.5, the use of intermediate stiffeners, composed of two steel plates connected through bolted connections, was foreseen. The thickness of the stiffener plates was determined according to the EC3 provisions [17]. Furthermore, an intermediate steel beam was considered to reduce the bending effects of the steel columns of the surrounding frame. This steel frame, having pinned-joints, was characterized by 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 to resist the effects induced by the plate tension-field mechanism. The plate-to-frame connections were realized 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. The Figure 4 shows one of the described retrofitting systems and the results obtained from the experimental tests.

a) *b)*

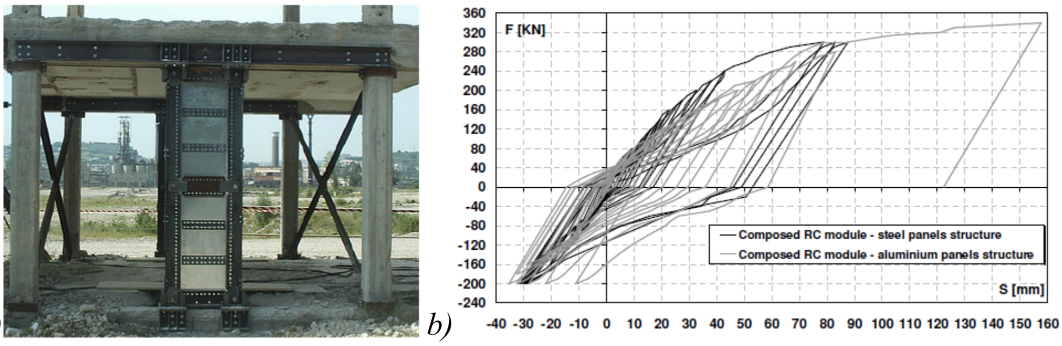


Figure 4: Retrofitted structure with steel plates (a) and comparison of the experimental load-displacement curves (b).

3 Setup and validation of the FE model

The choice of an appropriate and easily implementable FE model to both simulate the above experimental tests and perform supplementary numerical analyses is a crucially 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 FE software SeismoStruct [10] has been used. This software can predict the behaviour of three-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 [11] having a cross-section area equal to:

$$A_d = \frac{t\, b\, sin^2 2\alpha}{2\, sin\beta\; sin2\beta} \tag{1}$$

where t and b are the plate thickness and width, respectively, whereas α and β are the tension-field and diagonal angles of the steel plate measured from the vertical direction, respectively. The 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 software. In this model the tension-field angle α is given by:

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

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 [18]. The Canadian code [16] provides the following minimum second moment of area of columns adjoining SPSWs to prevent their excessive deformation, leading to premature buckling, under the pulling action of the plates:

$$I_c \geq \frac{0.00307\,t\,d^4}{b} \tag{3}$$

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

According to Sabouri-Ghomi et al. [19], the behaviour of thin plates in the pinned joint frame can be modeled 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 by as follows:

$$F_{py} = \frac{C_{m1}}{2}\,\sigma_{ty}\,b\,t\,sin2\vartheta \tag{4}$$

$$K_{pw} = \frac{\frac{C_{m1}}{2}\sigma_{ty}\,sin2\vartheta}{\frac{2\,C_{m2}\,\sigma_{ty}}{E\,sin2\vartheta}}\frac{b\,t}{d} \tag{5}$$

In the Equations (4) and (5), d, b and t are the terms already introduced, E and G are the normal and shear elasticity moduli of the metal plate, σ_{ty} is the tension-field stress in the plate yielding condition, ϑ is the tension-field angle, measured from the horizontal 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. These values can be obtained from the calibration of experimental tests. An example of a useful analytical tool for the estimation of these factors is proposed in [9], where the Authors provided appropriate design charts for designers, derived from experimental tests on the sold products of steel producers, to obtain the modification factors.

In order to setup a valuable FE model in SeismoStruct, the behaviour of the bare structure of Bagnoli [12] has been calibrated taking the above data. RC beams and columns have been modelled by using *infrmFB* elements, while the floor has been modelled using *elfrm* beams having same stiffness and weight of the effective floor. A three-dimensional view of the modelled sub-structure is illustrated in Figure 5.

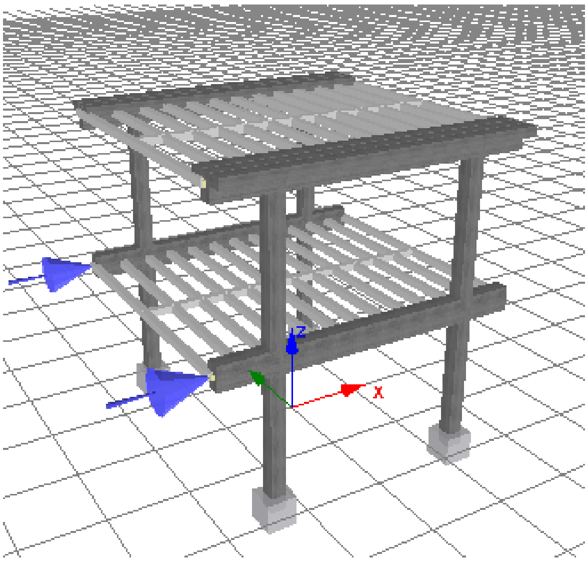


Figure 5: Numerical model of the sub-structure n°5 setup with the SeismoStruct software.

A reduced elastic normal modulus E_c of RC beams and columns has been adopted to take into account the degradation effect associated with the weather (see Figure 4). In particular, $0.5E_c$ and $0.4E_c$ have been adopted for beams and columns, respectively. Degradation zones extended at lengths of 35 cm and 65 cm for beams and columns, respectively, have been assumed. The experimental-to-numerical modal comparison achieved with these assumptions is shown in Table 3. Subsequently, $0.3E_c$ and a reduced strength have been assumed for the edge columns to consider the damage 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.

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

Table 3: Experimental-to-numerical comparison of the module n°5 vibration periods.

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

Once the initial structure has 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.

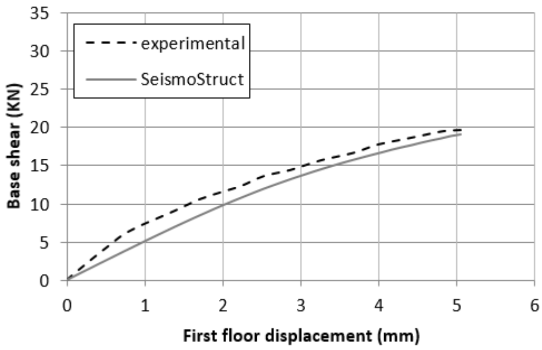


Figure 6: Comparison between the experimental curve and the final numerical one of the RC initial structure.

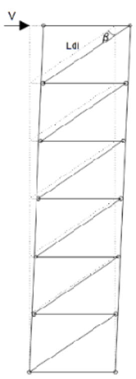
Each of six panel fields, having dimensions of 600x400 mm and being separated each other the horizontal stiffeners, has been numerically represented with an by equivalent diagonal, as previously described (see Figure 7). The equivalent tensile diagonal has been modelled by a *truss* element with elastic-plastic material, starting from the shear strength F_{wy} and initial stiffness K_w of the wall estimated as follows:

$$F_{wy} = \frac{C_{m1}}{2}\,\sigma_{ty}\; b\; t\; sin2\vartheta \tag{6}$$

$$K_w = \frac{\frac{C_{m1}}{2}\,\sigma_{ty}\; sin2\vartheta}{\frac{2\,C_p\,\sigma_{ty}}{E\; sin2\vartheta}}\,\frac{b\,t}{d} \tag{7}$$

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 [20].

a)



b)

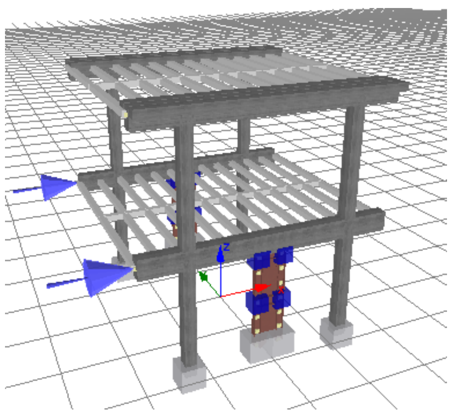


Figure 7: Calculation scheme of the SPSW (a) and SeismoStruct numerical model of the retrofitted sub-structure n°5 (b).

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 curve (see Figure 8). The same comparison could be done also for the aluminium solution but, with the damage that occurred after the test on steel panels, a further calibration is required.

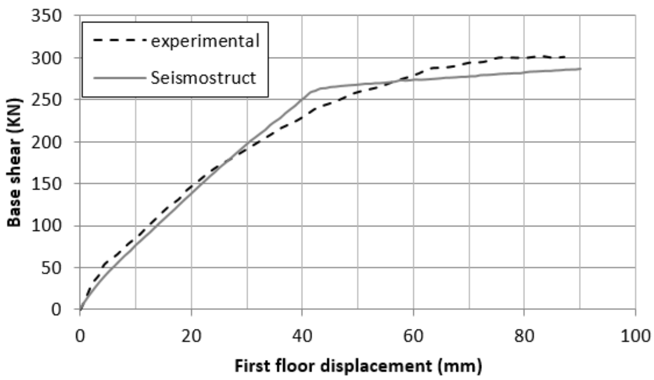


Figure 8: Comparison between the experimental curve and the calibrated numerical one for the structure retrofitted with steel panels.

4 Application to a case study

The benefits arising from the use of perforated steel panels instead of traditional full ones are already known [8]. However, few studies on existing RC buildings retrofitted with such devices are available. Therefore, in this paper, an existing building has been retrofitted with either traditional or perforated panels aiming at showing the different advantages deriving from their use [21]. The case study is a residential multi-storey 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 is on five storeys with rectangular shape of dimensions 30x12 m (see Figure 9). It has two bays in the transverse direction and seven bays in the longitudinal one. The ground floor, generally dedicated to commercial activities, has a height of 4.0 m, while the upper floors a height 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 each to other in the transversal direction only from both the slab and the edge beams. The staircase is located in the building central position and it is made of 30x60 cm knee beams. Floors are made of RC - hollow tiles mixed slabs having depth of 28 cm and 24 cm at the intermediate and top levels, respectively.

In absence of specific documentation on carpentry, the elements sizes have been determined from in-situ inspections, whereas the reinforcement details have been deduced from an appropriate simulated design [22]. 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 [15], a 50% reduced Young's modulus has been assumed for both beams and columns.

The building is located on a soil type C having a peak ground acceleration $a_g S$ equal to 0.28g and corresponding to a 975 years return-period [15].

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

From the modal analysis, whose results are depicted in Table 4, 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 (see Figure 11), it appears that in the longitudinal direction the demand is particularly focused between the 3rd and 4th floor, where the variation of in elevation stiffness is very high (see Table 5). On the other hand, in the transversal direction, the failure is essentially caused by the staircase column collapse.

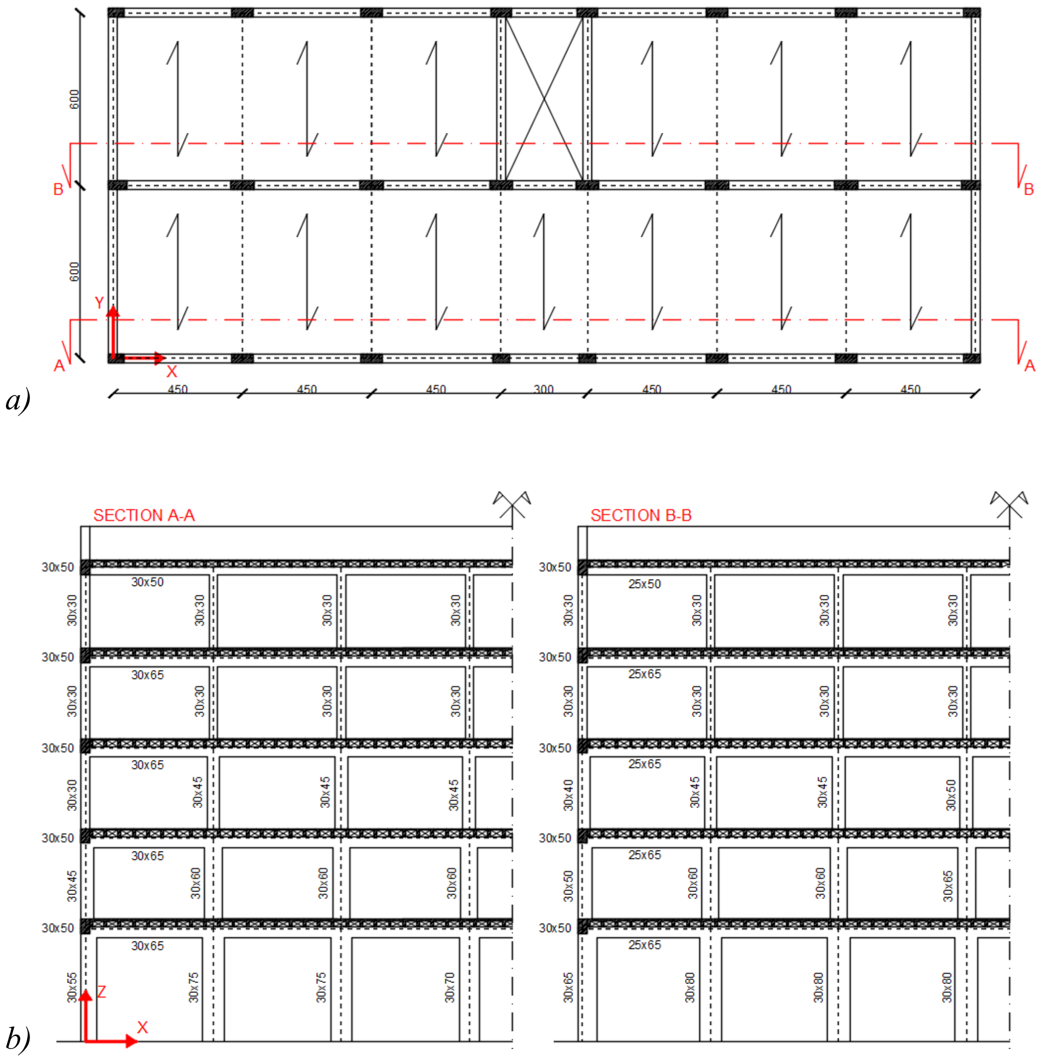


Figure 9: Existing building under investigation: typical plan layout (a) and vertical sections (b).

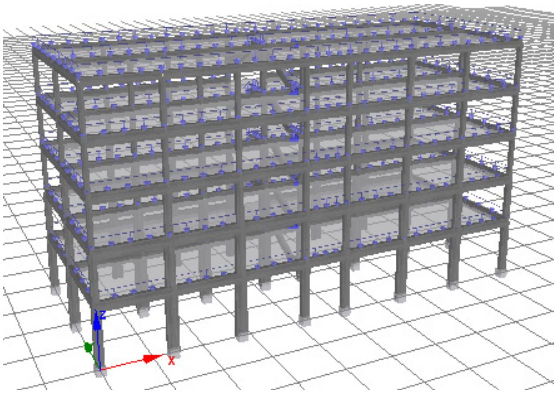


Figure 10: Numerical model of the investigated existing 5-storey building.

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

Table 4: Modal analysis results.

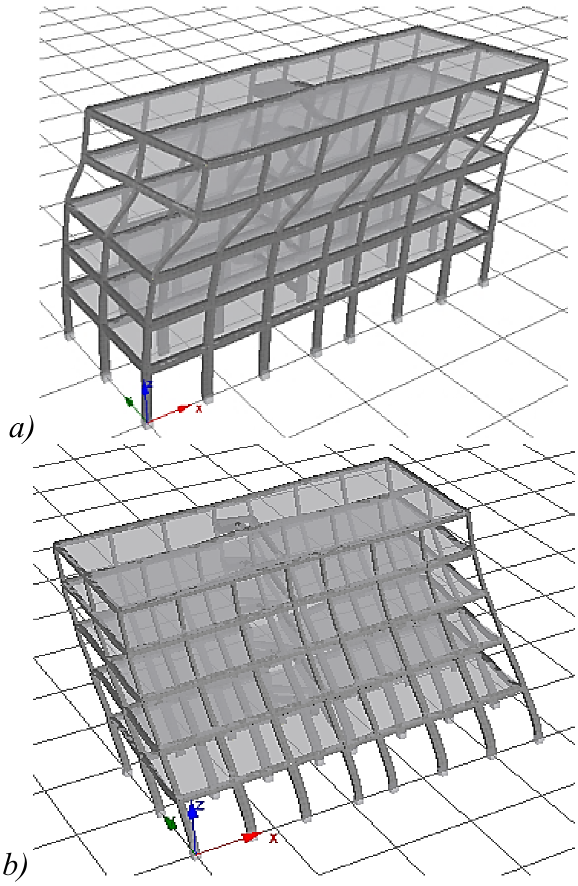


Figure 11: Deformed shape of the building under pushover analysis in directions X (a) and Y (b) (amplified deformation factor equal to 50).

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%	87877	0%	31419	-35%
4	473	0%	87760	-51%	48290	-21%
3	474	-1%	180779	-38%	60844	-14%
2	478	-4%	293620	-26%	70666	-21%
1	499	-	397521	-	89464	-

Table 5: Regularity analysis of the structure.

The seismic upgrading of the above RC building by means full SPSWs, which are known to provide the structure with a significant contribution in terms of initial stiffness, shear strength and dissipated energy, has been developed on the basis of the US procedures [13],[14]. 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 (8) and (9), respectively. In particular, in Equation (9), 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_p provided by the panels is determined from Equation (10), where the term K_{ini} is the initial structure stiffness.

$$T_{ret} = 2\pi\sqrt{\frac{S_{d,pp}}{Sae,pp}} \tag{8}$$

$$K_{ret} = K_{ini}\left(\frac{T_{ini}}{T_{ret}}\right)^2 \tag{9}$$

$$K_p = K_{ret} - K_{ini} \tag{10}$$

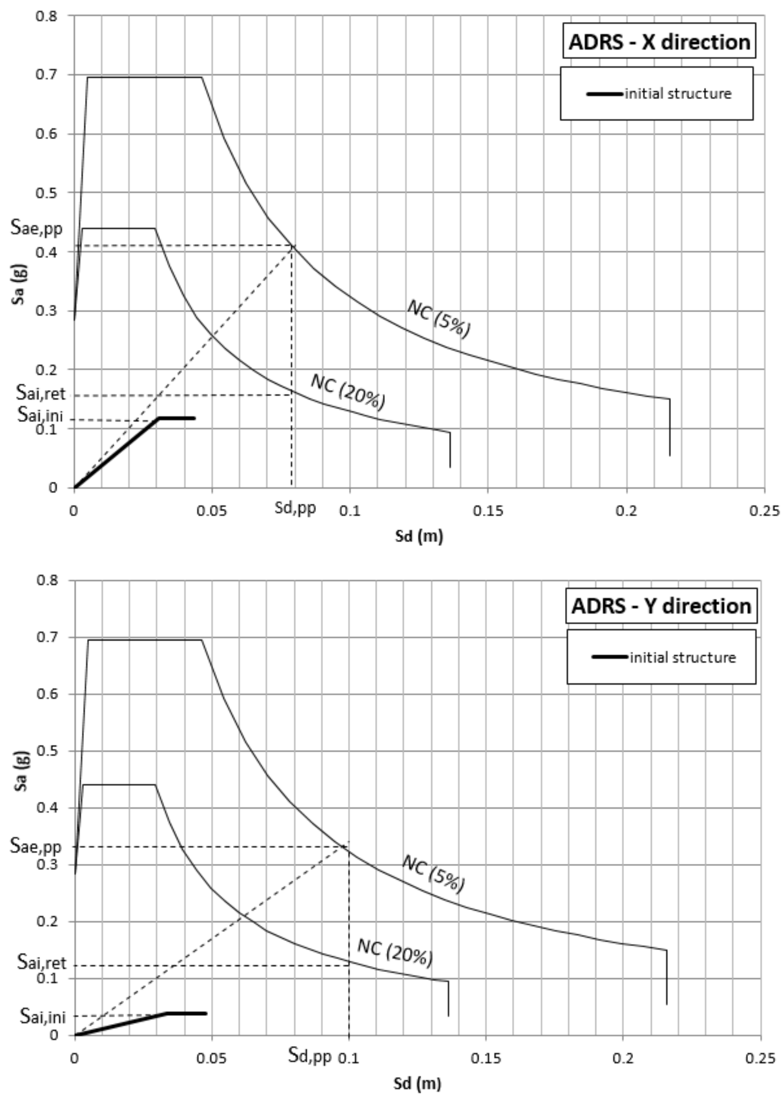
Considering that the retrofitted structure is able to provide at least the same damping level of the bare structure, the target shear strength of the retrofitted structure V_{ret} is obtained from Equation (11), where V_{ini} and $S_{ai,ini}$ are the shear strength and the inelastic spectral acceleration 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_p given by panels is evaluated through Equation (12).

$$V_{ret} = V_{ini} \frac{S_{ai,ret}}{S_{ai,ini}} \tag{11}$$

$$V_p = V_{ret} - V_{ini} \tag{12}$$

In Figure 12, 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. By analogy with [12], an upgraded system with partial-bay SPSWs, arranged in one and two pairs along directions X and Y, respectively, has been designed (see Figure 13). The SPSW disposition has been dictated from both the necessity to reduce as much as possible the interruption of building activities and to respect the architectural requirements.



a)

b)

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

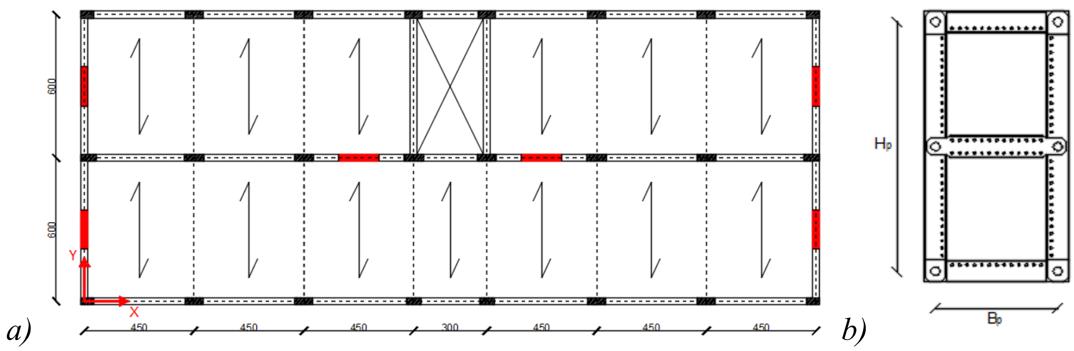


Figure 13: Location of SPSWs (a) and details of the external frame (b).

In order to respect the optimal panel shape ratio [4] and considering the building inter-story height, the SPSW width B_p 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. Full steel plates with yielding strength of 235 MPa have been chosen as seismic-resistant systems. The thicknesses of plates have been firstly derived by reversing Equations (6) and (7) and then converted into the most used commercial values. Tables 6 and 7 show the results of this first design phase.

Floor	f_y (MPa)	C_{m1}	B_p (mm)	$V_{pi,x}$ (KN)	$n_{p,x}$	$t_{p,x}$ (mm)	$V_{pi,y}$ (KN)	$n_{p,y}$	$t_{p,y}$ (mm)
5	235	1.0	1650	223	2	0.58	475	4	0.61
4			1650	465	2	1.20	989	4	1.27
3			1650	651	2	1.68	1383	4	1.78
2			1650	780	2	2.01	1658	4	2.14
1			1650	856	2	2.21	1818	4	2.34

Table 6: Thicknesses of SPSWs derived from the strength 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)	$C_{p,y}$	$H_{p,y}$ (mm)	$t_{p,y}$ (mm)
5	200000	14991	8.5	2300	1.78	31850	13.6	2400	3.15
4				2250	1.74			2400	3.15
3				2250	1.74			2400	3.15
2				2250	1.74			2400	3.15
1				3375	2.61			3450	4.53

Table 7: Thicknesses of SPSWs derived from the stiffness design.

Since the stiffness based design implies greater thicknesses than the strength based one, the values from the former design process have been considered, they being subsequently replaced by the most common commercial types (see Table 8). The steel frame surrounding SPSWs has been designed to both possess an adequate stiffness and remain in the elastic field. This outcome is achieved for full panels by both using Equation (3) and verifying the elements under the actions induced by the tension-field mechanism [23]. The coupled UPN profiles in Table 8 have been obtained from this procedure.

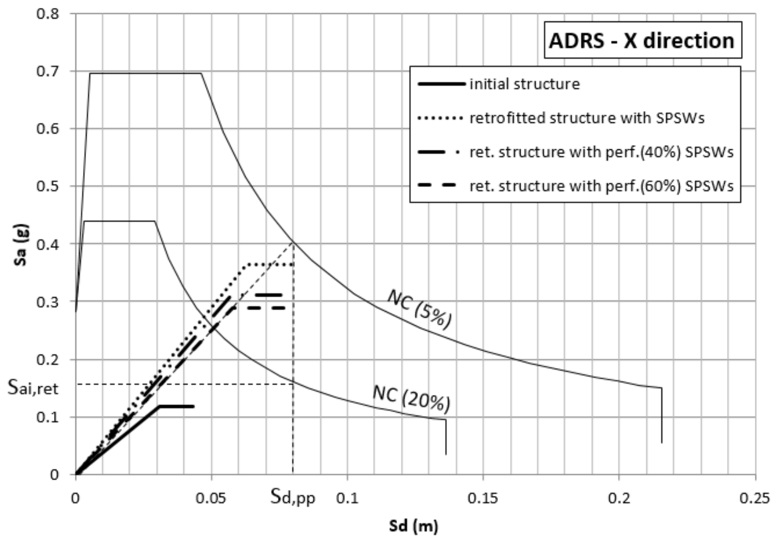
Floor	Steel plates (S235)		Steel frames	(S275)
	$t_{p,x}$ (mm)	$t_{p,y}$ (mm)	X dir.	Y dir.
5	1.80	4.00	2UPN160	2UPN240
4	1.80	4.00	2UPN160	2UPN240
3	1.80	4.00	2UPN160	2UPN240
2	1.80	4.00	2UPN160	2UPN240
1	3.00	5.00	2UPN260	2UPN320

Table 8: The assumed plate commercial thicknesses and steel frame members.

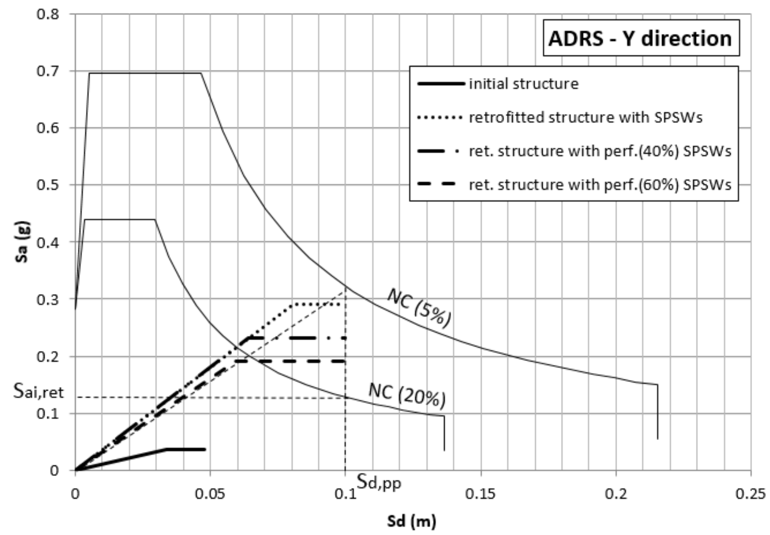
In order to transfer the actions to the walls, the RC beams have been reinforced, analogously to the intervention of Figure 4, by means of two UPN300 coupled profiles at the first floor and two UPN260 at the upper floors.

The analysis results have shown that only SPSWs are not sufficient to achieve the target displacement, due to the failure of existing columns. Therefore, the retrofit project has been completed with the RC jacketing of the longitudinal perimeter columns at the 3rd and 4th floors, of the transversal perimeter columns from the 2nd to 4th floors and of the stair case columns up to the 4th floor. Furthermore, jacketing with steel profiles has been considered for members incurring 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 displacement.

Successively, two retrofitting solutions by means of perforated SPSWs, having the same size and yielding strength than full SPSWs, have been proposed. The first solution is characterized by a 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 ρ = 60%. The behaviour of the perforated panels has been implemented in the FE model by adopting a linear reduction of the modification factors in comparison to those used for full panels [9]. Figure 14 shows the results obtained from the pushover analyses on the structure equipped with perforated SPSWs.



a)



b)

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

The results show that the shear strength of the structure retrofitted with full SPSWs is clearly higher than that of perforated panels. 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 perforated SPSWs 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, as RC beams reinforcement, for the less drilled solution, UPN280 coupled profiles at the first floor and two UPN240 at the upper floors have been adopted, while for the more drilled solution, UPN260 coupled profiles at the first floor and two UPN220 at the upper floors have been adopted. Additional saving occurs for the steel frame of the walls, since holes in the plates implicate a plastic deformation concentration around them, reducing the actions on the perimeter areas (see Table 9). Considering the current costs of both steel elements and local reinforcing interventions, a cost saving of at least 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.

Floor	Perforated SPSWs ($\boldsymbol{\rho}$ = 40%) Steel frames (S275)		Perforated SPSWs ($\boldsymbol{\rho}$ = 60%) Steel frames (S275)	
	X dir.	Y dir.	X dir.	Y dir.
5	2UPN120	2UPN180	2UPN120	2UPN120
4	2UPN120	2UPN180	2UPN120	2UPN120
3	2UPN120	2UPN180	2UPN120	2UPN120
2	2UPN120	2UPN180	2UPN120	2UPN120
1	2UPN160	2UPN220	2UPN120	2UPN160

Table 9: Steel frame members of the perforated SPSWs.

5 Conclusions

In this paper a study aimed showing the benefits of using perforated SPSWs has been presented. The use of such systems, already known in the literature for applications in new steel structures, can be particularly advantageous for retrofitting existing buildings designed without seismic criteria. In particular, 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 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 the calibration and validation of a simple FE model developed with the SeismoStruct software.

Subsequently, the case study of an existing multi-storey RC building retrofitted with either traditional 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 at least 16% and 27%, respectively, compared to the cost deriving from using full panels. In addition, by increasing the drilling configuration, a significant shear strength reduction is achieved without excessively compromising 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.

However, the main benefit deriving from the use of perforated plates is to select *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 data assigned for architectural requirements, which cannot be modified.

Finally, shear walls with perforated common steel plates can be a viable alternative to others stiffening and dissipative solutions based on metals more expensive (aluminium) and not available in the European market (low yield steel).

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