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COLD FORMED STEEL PANELS FOR THE SEISMIC STRENGTHENING OF EXISTING RC BUILDINGS

Simone Labò[1], Andrea Belleri[1], Michelle Gualdi[1], Atsushi Sato[2]

[1] University of Bergamo
Department of Engineering and Applied Sciences
Dalmine (BG), Italia
e-mail: simone.labo@unibg.it, andrea.belleri@unibg.it, m.gualdi5@studenti.unibg.it

[2] Nagoya Institute of Technology
Department of Architecture and Design
Nagoya, Japan
e-mail: sato.atsushi@nitech.ac.jp

Abstract

In the last years, structural retrofit solutions for RC buildings have been studied and developed to meet the urgent need to renovate the existing building stock under multiple perspectives: seismic safety, architectural and comfort renovation, and thermal envelope enhancement. Cold-formed steel (CFS) panels could be used for this purpose among other strengthening solutions.
This paper investigates the behavior of CFS panels used as seismic strengthening for existing RC buildings. Reduced complexity finite element models are used to numerically simulate the structural behavior of CFS panels; simplified analyses are carried out to derive useful indications about the applicability of these panels in strengthening RC buildings such as, for example, the panel stiffness and strength for varying the panel components.
The applicability of CFS panels as a seismic retrofit solution for existing buildings not designed to withstand horizontal loads is herein preliminarily addressed through the application to a reference case study.

Keywords: Renovation, Cold-Formed Steel, Seismic Retrofit System.

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1 INTRODUCTION

Sustainability has become a huge priority in global development in all fields, including construction. Most European buildings have overcome their "design" end-of-life and manifest both structural and energy efficiency issues. The obsolescence of the existing reinforced concrete (RC) building stock concerns structural and non-structural components and can result in partial or total collapse, particularly under seismic loads. This condition can be found in Italy, where more than 50% of the existing RC buildings were built before 1974 [1], [2], i.e., before the enforcement of modern anti-seismic regulations. In recent years, many initiatives have been promoted to increase building energy performances. However, structural safety is rarely considered.

The concept of sustainability can't be related only to environmental aspects but must incorporate structural safety other than resilience. An integrated and holistic approach including all sustainability aspects in building retrofit was proposed by [3], [4], [5], which define a strategy targeting building renovation to reduce the impact along the building life cycle. Subsequently, the effective implementation of interventions with restorable, versatile, maintainable elements made with reusable materials is the key to reducing the impact of the building itself during its life.

In this regard, using prefabricated and standardized retrofit components allows to speed up the installation procedures and dismantle and reuse of the components at the end of life to reduce the refuse. In this context, the use of Cold-Formed Steel (CFS) panels as seismic-resistant elements is a valuable solution.

The first experimental investigations on the seismic behavior of CFS panels were conducted in the 90s [6], [7]. A series of tests were carried out on shear wall specimens consisting of CFS framing sheathed with sheet steel attached to a gypsum board on one side and with a plain gypsum board on the opposite side by [8] at the University of California. Full-scale shake table testing was run also in "CFS-NEES" North American projects [8].

In Italy, shake-table tests on a two-story full-scale building were conducted [10], showing that even though the box-building behavior and the presence of non-structural elements can increase the lateral resistance of the construction, sudden and undesirable brittle failures can appear. The dynamic behavior of reduced-scale three-story two-bay prototypes was analyzed by [11] where the main outcomes showed that the response to seismic action can be considered adequate thanks to the ductility of the system in the case of strap-braced walls.

Two interesting applications of cold-formed steel panels were tested at the Nagoya Institute of Technology (Nagoya, Japan) ([12], [13], [14], [15], [16]). Based on the results of the experimental campaigns on a) steel trapezoidal sheet shear walls [16], and b) steel sheet shear walls with burring holes [12], these systems are investigated herein with the purpose of evaluating their suitability as seismic retrofit systems for existing RC buildings. With this aim, first, the compatibility of this system with post-World War II RC buildings was assessed. Then a simplified finite element (FE) modeling strategy was developed to numerically investigate the effectiveness of the panel as a seismic force-resistant system (SFRS).

2 EXPERIMENTAL TESTS AND PRELIMINARY CONSIDERATIONS

2.1 Results of experimental campaigns and simplified backbone curves

For the sake of clarity, the trapezoidal sheet shear walls [16] and the steel sheet shear wall with burring holes [12], are called Panel 1 and Panel 2 in the following, respectively. The main geometries and layout are summarized in **Figure 1**; further details on the tested panels may be found in [16] and [12].

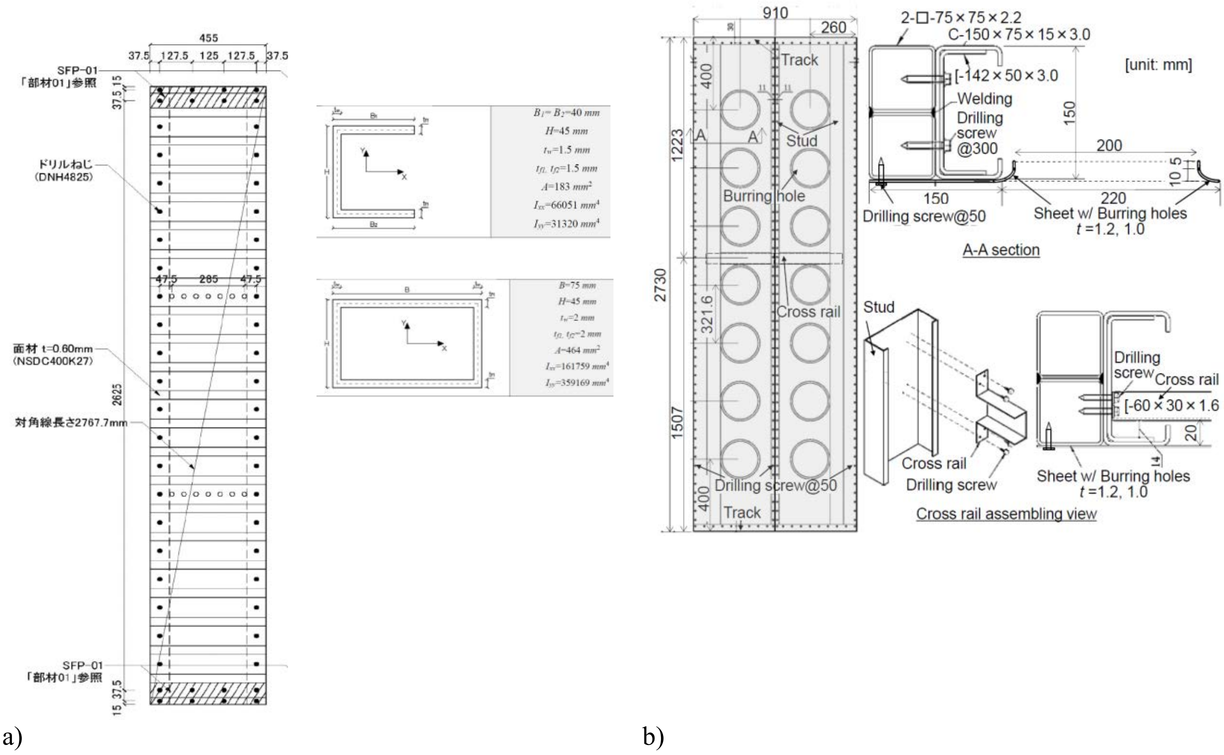


Figure 1: Experimental campaigns analyzed: a) the corrugated shear walls. Figure from [16], and b) the steel sheet shear wall with burring holes. Figure from [12].

Both the panels were tested under cyclic loadings. Given the preliminary assessment purpose of this work, only the backbone curve was considered to preliminarily evaluate the panel applicability as seismic strengthening solutions for RC buildings. Experimental results and backbone curves are plotted in **Figure 2**. Since Panel 2 was two times larger than Panel 1, the backbone curve of Panel 1 was doubled (blue in **Figure 2a**).

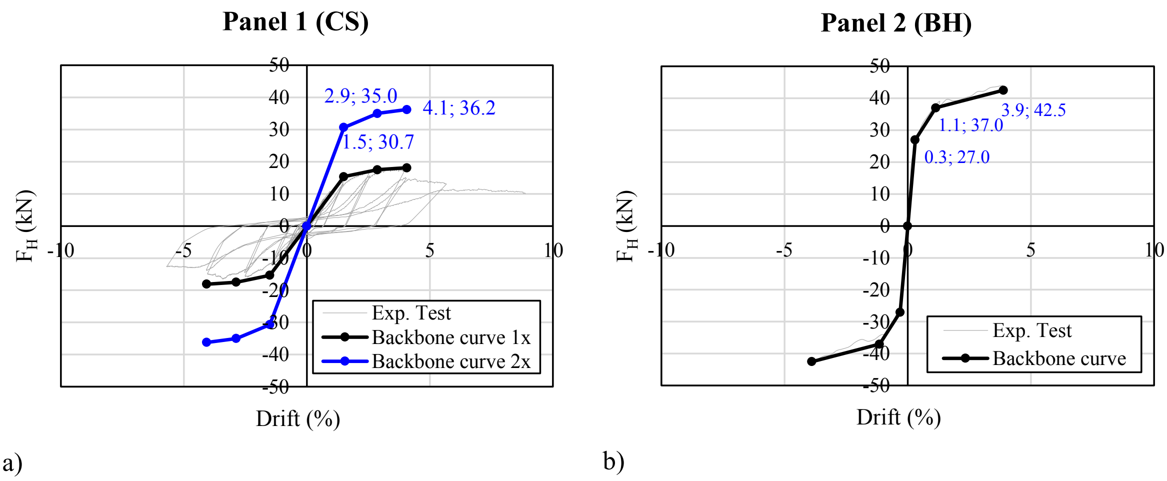


Figure 2: Experimental results and backbone curves of the panels: a) Panel 1 – corrugated sheet (CS), and b) Panel 2 – burring holes (BS).

The main parameters of the backbone curves are summarized in **Table 1**.

	Panel 1 (CS)	2xPanel 1 (CS)	Panel 2 (BH)	Unit
k_{El}	0.4	0.8	3.2	kN/mm
F_y	15	31	27	kN
δ_y	1.5	1.5	0.3	(%)
F_u	18	36	42	kN
δ_u	4.1	4.1	3.9	(%)

Table 1: Main parameters of the backbone curves derived from the experimental results.

2.2 CFS panels as retrofit systems: preliminary considerations

To draw some preliminary considerations, CFS panels structural features were compared with those related to a post-World War II RC frame. The reference frame was 4.67 m width (l) and 3.15 m height (h); both the cases with (W) and without (W/O) infills were considered. As for the frame, (50x31.5) cm^2 horizontal beams, and (30x30) cm^2 RC columns reinforced with 2+2Φ12 mm at the corners and Φ6/120 mm stirrups were considered. For the sake of simplicity, a shear-type behavior was considered. According to the regulation code at the time, concrete C20/25 (f_{ck}=25 MPa) and steel Feb32k (f_{ym}=315 MPa, f_{tk}=490 MPa) were considered.

As for the infills, weak, medium, and strong masonry infills were considered according to [17]. Their contribution was accounted for by means of two compression-only diagonal struts converging in the beam-column joints as adopted by [18], among others. The infills behavior was described by means of a trilinear curve in which cracking and peak forces were evaluated according to Decanini et al. [18], while the cracking and the peak displacement capacities were set as 0.3 % for minor cracking and 0.5 % for the infill failure [19].

Infills trilinear curves are plotted in **Figure 3** in terms of horizontal force (kN) as a function of the inter-story drift ratio (%). The infill panel stiffness (k_I) depends on the infill typology considered (weak, medium, and strong) and can be derived by dividing the peak force by the peak displacement to obtain an equivalent stiffness that accounts for cracking.

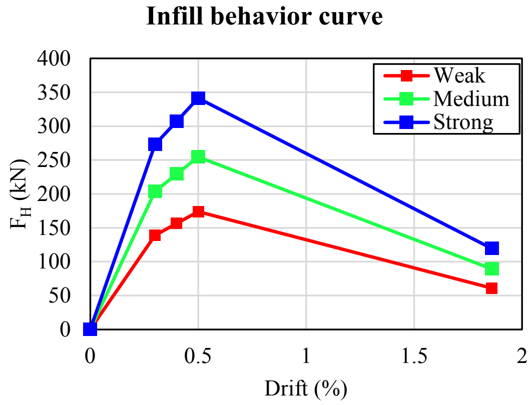


Figure 3: Trilinear curves of weak, medium, and strong infill panels.

The considered cases and the static schemes adopted to derive the stiffness and strength of the existing frame are depicted in **Figure 4**. Accordingly, the stiffness of the bare frame (k_{BF}) and of the infilled frame (k_{IF}) can be considered equal to $k_{BF} = 2 \cdot \left(\frac{12E^* I}{h^3} \right)$, and $k_{IF} = k_{BF} + k_I$, respectively where, E^* and I are the reduced elastic modulus (e.g., 0.5E, to account for cracking) and the inertia momentum of the RC columns, and h is the frame height. The strength of the bare frame was calculated according to the column dimensions, reinforcing bars, and material and accounting for a vertical load equal to 200 kN thus resembling the axial force of a residential three-floor building [15]. In the case of the infilled frame, the infill peak force was added.

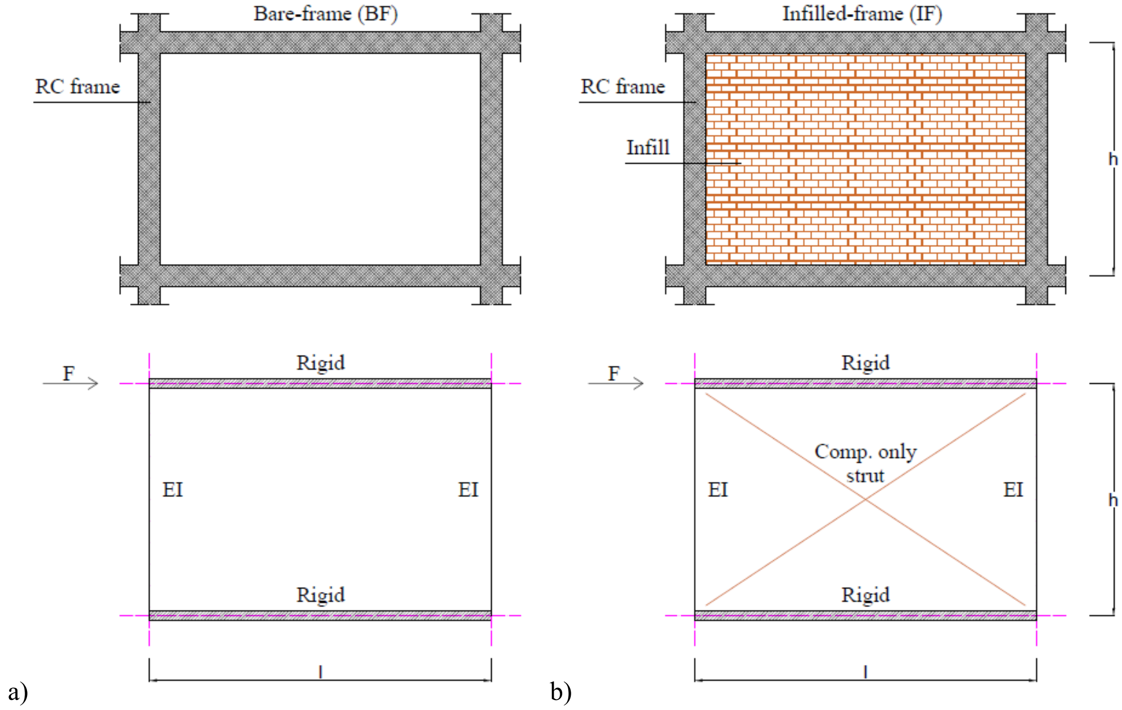


Figure 4: Sketches and static schemes of the reference frames considered: a) bare-frame, b) infilled-frame.

In this work, comparison is made on stiffness and strength considering that steel panels fully cover the frame surface (lxh); backbone curves were proportioned accordingly. Results are summarized in **Figure 5**. The stiffness and strength are plotted in **Figure 5a** and **Figure 5b**, respectively, for the bare frame (BF), the infilled frame with weak, medium, and strong infills (IF_w, IF_m, IF_s, respectively), Panel 1 (P1), Panel 2 (P2), and the cases in which Panel 1 and Panel 2 were considered as double (2xP1 and 2xP2) to resemble, for instance, sandwich-type panels.

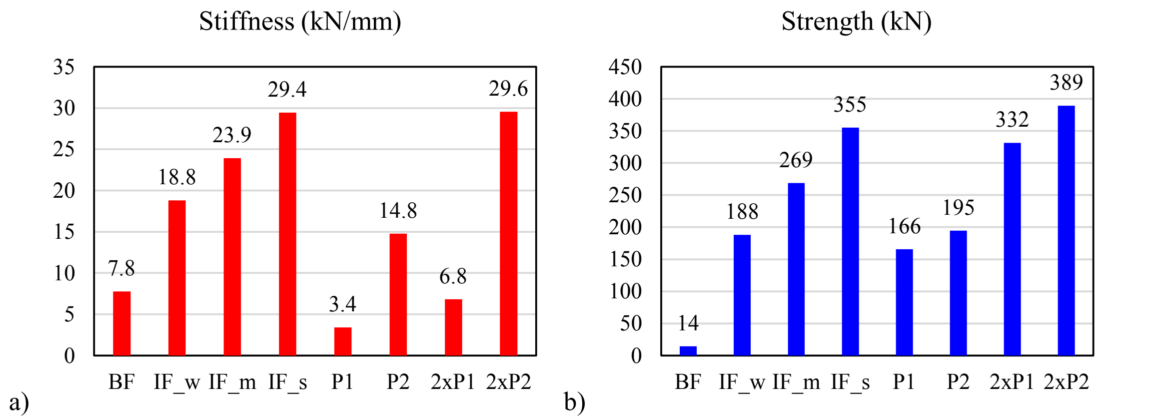


Figure 5: Stiffness and strength of the bare frame (BF), the infilled frame with weak, medium and strong infills (IF_w, IF_m, IF_s, respectively), the Panel 1 (P1), the Panel 2 (P2), and double P1, and P2 (2xP1 and 2xP2, respectively).

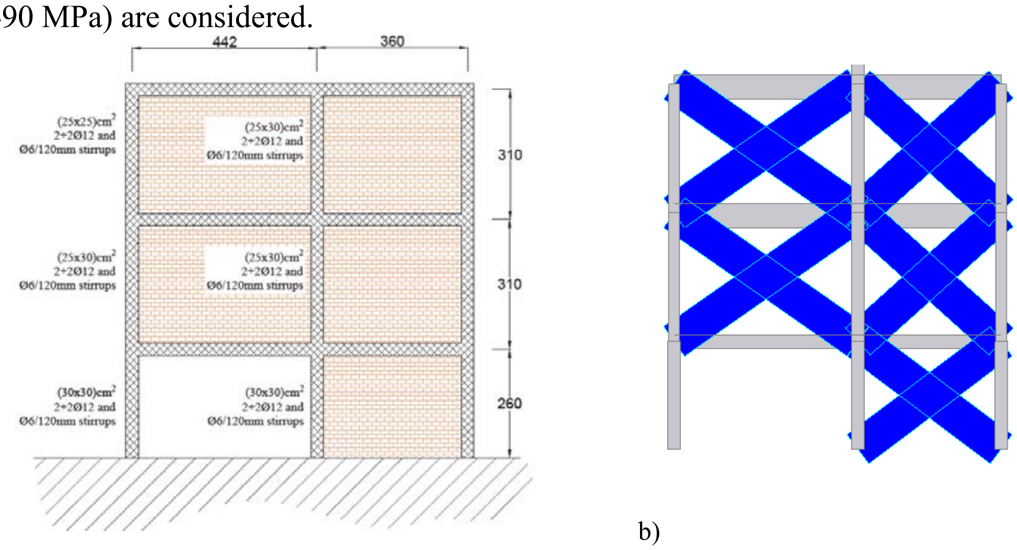
From **Figure 5** emerges that the major drawback in applying CFS panel as strengthening solution for RC existing buildings lays in the stiffness; to effectively preserve the existing structure (i.e., avoiding damages on both structural and nonstructural components) and transfer the seismic actions to the external CFS exoskeleton, the two systems stiffnesses must be comparable. Considering the CFS panel and existing frame as two springs in parallel, the seismic forces would be distributed according to their stiffnesses. Then, if the CFS panel stiffness is quite lower than that of the existing frame, even if the panel may provide equal or even greater strength than the existing wall, such a resource cannot be exploited unless extensive damage to the existing wall would allow for reducing its stiffness or selective weakening strategies are applied to the existing infills such as vertical cuts.

From **Figure 5a** emerges that Panel 1 stiffness is not even comparable to the BF case; only by considering a double panel (e.g., in a sandwich solution), its stiffness can be compared to the BF case. On the other hand, Panel 2 stiffness is comparable to a medium-capacity infilled frame (IF_m) and, if used as double (e.g., sandwich panel type), its stiffness may be compared to a strong-capacity infilled frame.

3 APPLICATION TO A REFERENCE FRAME

A reference 2D perimetral frame was designed to resemble a typical reinforced concrete (RC) post-World War II building (**Figure 6**). The bearing structure is made of RC beams and columns designed for vertical loads only; column dimensions and steel reinforcement are summarized in **Figure 6a**. To define the static loads, a residential building with a one-way RC beam-and-block flooring system featuring a 3 cm RC overlay for a total thickness of 19 cm was considered; the transversal span of the frame was 3.40 m. As for the non-structural elements, masonry infill panels are made of two layers of hollow bricks with two outer layers of plaster (corresponding to the medium infill in **Section 2**). According to the regulation code at the time of construction, concrete C20/25 (f_{ck}=25 MPa) and steel Feb32k (f_{ym}=315 MPa, f_{tk}=490 MPa) are considered.

a)



b)

Figure 6: 2D reference frame. a) sketch of the reference 2D frame; b) sketch of the FEM.

The reference 2D frame was modeled with the software MidasGen [20]. *Beam*-type elements were implemented for the RC frame, while compression-only diagonal struts converging in the beam-column joints were adopted for the infills. The inelastic flexural and axial behaviors were accounted for by means of lumped plastic hinges. Strength, and deformation capacity of beams and columns were modelled according to the formulation suggested in the

European building code [21]; the non-linear behavior of the infills was modeled by means of the FEMA infill strut axial plastic hinge [22].

CFS panels were modeled by means of nonlinear *general links* acting in the frame plane. *General links* were calibrated according to the results discussed in **Section 2** and to the frame geometry. Given the infill irregularity, steel panels were supposed to be applied only to the fully infilled portion of the façade (right span in **Figure 6a**). Panels were rigidly connected to the frame.

Nonlinear static analyses were conducted, and the results are summarized in **Figure 7**; points 1, 2, and 3 represent the cracking, peak, and ultimate points of the infill panels, respectively.

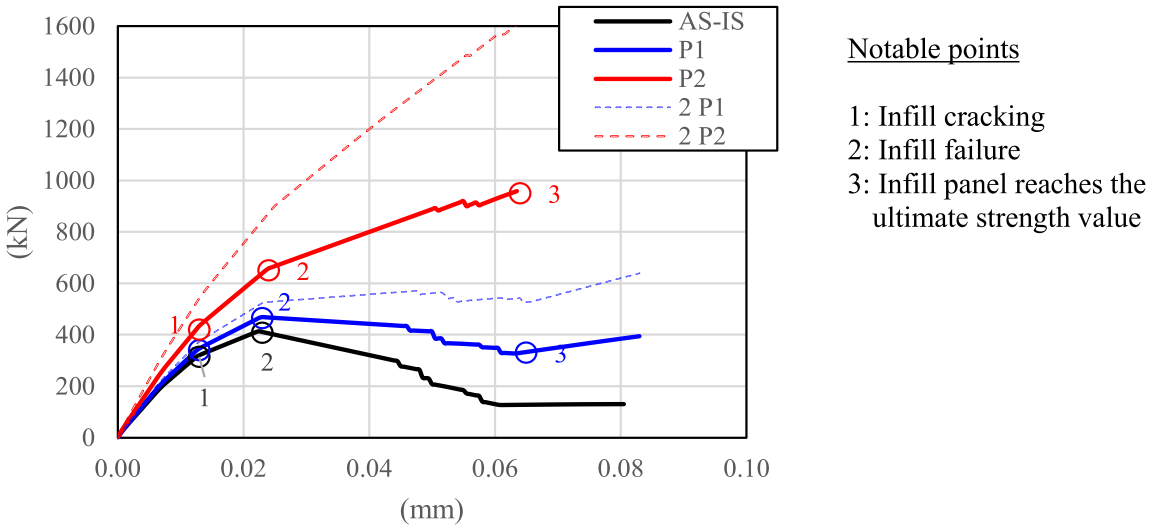


Figure 7: Capacity curves in the AS-IS condition (black) versus those obtained after introducing Panel 1 (blue) and Panel 2 (red) strengthening solutions. The double panels are plotted with dashed lines.

The capacity curve in the AS-IS condition is plotted in black line, Panel 1 solution in blue (P1), and Panel 2 solution in red (P2). As expected, by introducing Panel 1, stiffness and strength slightly increase (less than 30%). It is worth noting that when the infill panel at the base reaches its ultimate capacity (point 3) the capacity curve slightly increases due to the activation of the CFS panel. Although a further drop in the capacity curve was avoided thanks to the Panel 1 loading; however, this condition is associated with extensive damage in the infill panels. The results significantly change when Panel 2 is introduced (red line in **Figure 7)**. The stiffness almost doubled (1.7 times higher) and the global stiffness does not significantly vary at the infill panel failure. When double panels were introduced (dashed lines in **Figure 7**), the results do not significantly vary for the case of Panel 1, while stiffness and strength significantly increased (almost 3 times) in the case of Panel 2.

4 CONCLUSIVE REMARKS AND FUTURE DEVELOPMENTS

The paper aims to draw some preliminary consideration about the applicability of CFS panels as a seismic retrofit solution for existing post-World War II buildings. Moving from the results of two experimental campaigns, two solutions were investigated: in the first solution, corrugated steel panels were adopted, and in the second, steel sheet panels with burring holes were used. Preliminary considerations were drawn based on hand calculations and nonlinear analysis results. From the analyses emerged that:

- trapezoidal sheet panels are suitable in the case of bare-frame (without infill) or infilled frame with weak infill panels due to the limited stiffness of this strengthening system;

- sheet panels with burring holes may be an interesting retrofit solution since they allow for higher stiffness and strength with respect to the trapezoidal sheet panels;
- sheet panels with burring holes may develop stiffness and strength compared with those related to an ordinary infilled frame.

The effectiveness of such a retrofit solution deserves to be further evaluated. Moreover, the effectiveness of *sandwich*-type panels should be further investigated from a structural and technological point of view.

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