Evaluation of seismic damage to cladding panels in single-storey steel buildings through a multi-criteria approach

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Keywords: Damage limit state; Seismic analysis; Steel structures

ABSTRACT

The seismic design of steel constructions is commonly based on structural models where only the primary steel members and connections contribute to the strength and stiffness, i.e. cladding panels are considered in the seismic mass but are not included as components influencing the structural response. Few exceptions to this common approach can be found. In this context, the objectives of this article are: 1) the analysis of the influence of the cladding panels on the structural response under seismic conditions as compared to the results obtained when only beams, columns, and braces are considered in the model; 2) the evaluation of the seismic damage that cladding panels might undergo through explicit modelling of their response as component of the structural model. Recent studies on single-storey steel buildings made by moment-resisting portals in the transverse direction and concentrically-braced frames in the longitudinal direction have pointed out that claddings might have an important role on influencing the seismic response and proposed a multi-criteria scheme for the evaluation of the damage in claddings. Starting from these achievements, the current study presents the analysis of the quantification and extent of the damage in the cladding panels for increasing levels of the seismic input. The considered case study is a single-storey steel building made of moment-resisting portals in the transverse direction and concentrically-braced frames in the longitudinal direction. Specific attention is given to the investigation of the influence of cladding panels with non-symmetric distributions in plan introducing torsional effects even if the structural elements have a symmetric configuration.

1 INTRODUCTION

The analysis of steel constructions for their seismic design is commonly based on structural models where only beams, columns, and braces contribute to the seismic response, while cladding panels are considered in the overall seismic mass but are not included as components explicitly influencing the structural behaviour, e.g. (Ballio and Mazzolani 1983), (Landolfo et al. 2015). Exceptions to this common approach are few studies that can be found in the literature and dealing with studies involving the following aspects: (i) development and validation of nonlinear cyclic models for cladding panels to be included in advanced models incorporating structural and non-structural elements; (ii) design of cladding panels as structural elements providing bracing functions; (iii) analysis of the influence of the cladding panels on the structural response under seismic conditions as compared to the results obtained when only beams, columns, and braces are considered in the model definition; (iv) analysis of the seismic damage that cladding panels undergo through explicit modelling of their response as component of the structural model. A review of the state of the art on such topics can be found in (Scozzese et al. 2018a). The present study contributes on the latter two aspects, i.e. cladding panels are included in the structural model with two objectives: 1) evaluate their influence on the simulated seismic response, and 2) estimate their damage under seismic excitations. The starting point is the outcome provided by previous studies on single-storey steel buildings (Scozzese et al. 2018a, 2018b) pointing out that claddings have an important role on the seismic response and should not be neglected in a refined finite element model.

More specifically, in this study, the multi-criteria approach proposed by (Cardone and Perrone 2015) for reinforced concrete frames and used in (Scozzese et al. 2018a) for the evaluation of the damage to the cladding panels is reviewed and its applicability is discussed. Specific attention is given to the investigation of the influence of cladding panels with non-symmetric distributions in plan, not evaluated in (Scozzese et al. 2018a). Two different panel distributions are considered: 1) perfectly symmetric along both X and Y directions; 2) asymmetric distribution along the X direction. Moreover, the effect of the choice of the control node used to evaluate the drift ratio is analysed. This is an aspect of high relevance for the type of buildings investigated, being characterized by the absence of a rigid diaphragm at the roof level. This implies that differential movements are not restrained among the frames, and, thus, the choice of the nodes with respect to which the drifts are computed might affect the final outcomes of the analyses. The relevance of this matter is further amplified by the consideration of panels with asymmetric distribution introduce torsional effects even if the structural elements have a symmetric configuration.

2 NON-STRUCTURAL DAMAGE ASSESSMENT

2.1 *Methodology*

According to the European structural codes involving steel structures (Eurocode 3, Eurocode 8) as well as the Italian structural code (NTC2018), the assessment of non-structural seismic damage of buildings is conventionally pursued by monitoring global response quantities, such as the inter-storey drift. The main limit of this conventional approach is represented by the lack of an explicit relationship with the actual level of deformation of the non-structural elements, whose behaviour is not explicitly considered during seismic analyses, and the relevant lack of the description of the interactions between structural and non-structural elements.

In this study, the damage evaluation of the non-structural elements is pursued through their inclusion in the structural model, thus, allowing an explicit estimation of their local seismic demand. A direct comparison with the outcomes achieved by means of a global drift-based approach applied to the conventional analysis of bare-frame models allows comparing the results of the proposed methodology to the results of the conventional code-based damage verifications.

In order to assess the onset of the Damage Limit State (DLS), a multi-criteria approach using a three-level local damage criterion (Scozzese et al. 2018a; 2018b) was proposed to characterize the attainment of the DLS of buildings:

(1) low-damage level: a percentage of panels exceeds the elastic response limit (a 50% value is considered in this study);

(2) medium-damage level: the totality of panels is beyond the elastic field;

(3) strong-damage level: at least one panel exceeds its maximum shear resistance.

A global condition is added besides the three-level local damage criterion, i.e. attainment of the 95% of the maximum base shear force resistance of the whole structural system, in order to consider cases where significant structural damage might occur before the development of damage in non-structural elements (Scozzese et al. 2018a; 2018b).

2.2 *Non-structural element modelling approach*

Lightweight sandwich panels widely used as enclosure elements in both industrial and civil constructions are considered in the case study herein analyzed. In particular, the type A panels tested by (De Matteis and Landolfo 2000) are adopted. The panels consist of external embossed steel sheets (thickness of 0.6 mm) with slight stiffening ribs and insulating polyurethane core, for a total thickness of 40 mm. The connection of panels to the main structural frame is made through cladding rails (for vertical cladding panels) or purlins (for roof panels) by means of bolts (generally of 8 mm in diameter and 110-120 mm spacing). For more details the interested reader is referred to (De Matteis and Landolfo 2000).

Each panel was modelled by means of a couple of diagonal truss elements with nonlinear axial behaviour following the Pinching4 model, able to describe the main features of the shear-displacement experimental response of cladding panels typically used in steel constructions (De Matteis and Landolfo 1999).

The Pinching4 model was calibrated based on the experimental results available from cyclic shear loading tests performed on individual units of panel type A, having size of 1000 mm x 2500 mm x 40 mm, as illustrated and discussed in (Scozzese et al. 2018). Details on the Pinching4 implementation can be found in (Scozzese et al. 2018a, 2018b).

3 CASE STUDY

3.1 *Geometry and structural design*

The considered case study has five single span duo-pitch portal frames repeated in the longitudinal direction at a constant distance; the frames are connected in the longitudinal direction by hot-rolled beams at the apex, at the eaves and at the crane-supporting bracket level. A three-dimensional view of the structural system is presented in Figure 1. The transverse X-direction has its abscissa labelled with the letters A and B, the longitudinal Y-direction has its ordinates labelled with numbers.

Horizontal forces are withstood by two different structure typologies: in the X-direction the resistance to lateral forces is due to moment resisting action of the portal frames, i.e. moment resisting frames (MRFs); in the Y-direction the resistance is provided by vertical concentric bracings, i.e. concentrically braced frames (CBFs), placed in the outer spans. The same geometry was considered in (Scozzese et al. 2018a, 2018b) and a detailed description of its structural configuration as well as its loading conditions and general design approaches, can be found in (Scozzese et al. 2017).

The case study is located in Naples, soil type C, topography condition T1. The transverse bay width is Lx = 20.00 m, the longitudinal bay width is Ly = 6.00 m, the height at the eaves is H = 6.00 m, and the height of the crane-supporting bracket (measured at top sur-face of the bracket) is Hc = 4.50 m. The roof pitch is equal to 6°. Purlins are used to support the roof cladding and are positioned every 2.5 m. Roof bracings are arranged in the outer bays to transfer horizontal forces to the vertical bracings. Steel grade is S275. The presence of a travelling overhead crane was treated as illustrated in details in (Scozzese et al. 2017).

Variable environmental loads (snow, wind, thermal actions) and seismic loads were determined accordingly to the Italian code prescription for the considered site. Seismic designs were made by assuming a dissipative structural behaviour, i.e., according to the ductility class high of the 2018 Italian Code (indicated in the code as CD“A”), providing a behaviour factor $q \geq 4$ (assumed $q = 4$) for the moment resisting frames and $q = 4$ for the frames with concentric braces.

Serviceability limit state verifications for the horizontal displacements of the portal frame for the characteristic combination of loads was limited to $\Delta i/H=1/300$. For the damage limitation limit state (SLD), the horizontal drift was limited to 1/200. The difference of horizontal deflection be-tween two consecutive portal frames, Δij /Ly, was also checked to not exceed 1/200 (Scozzese et al. 2017). Cross-sections resulting from the design are summarised in Table 1.

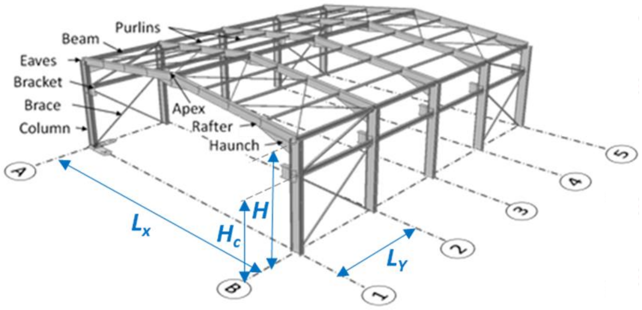


Figure 1. View of the steel structure of the considered case study

Table 1. Designed cross section for the considered case study.

Element	Cross-section
Column	HE 600 M
Rafter	HE 450 A
Vertical X braces	50x3
Vertical single brace	70x4
Longitudinal beam	IPE 270
Purlins	HE 160 A
Roof bracings	L 20x3

3.2 *Nonlinear finite element model and analysis*

The nonlinear response of the case study was investigated using the finite element open source software OpenSees (McKenna 2011) including both geometric and material nonlinearities. All the structural elements were modelled using nonlinear fibre sections with nonlinear behaviour assigned to each section fibre thorough the Steel02 constitutive law (Giuffré-Menegotto-Pinto steel material object with isotropic strain hardening) available within the OpenSees libraries. Attention was given to the following aspects of the vertical brace modelling: description of the buckling phenomenon in compression; proper modelling of the gusset plate connections that in real structures are neither pinned nor fixed joints. The method followed for modelling the bracings is that proposed in (Hsiao et al. 2012; Hsiao et al. 2013) and consisting in simulating the nonlinear out-of-plane rotational behaviour of the gusset plate connections by means of a rotational nonlinear spring located at the physical end of the brace. Details of the nonlinear finite element model of the structural

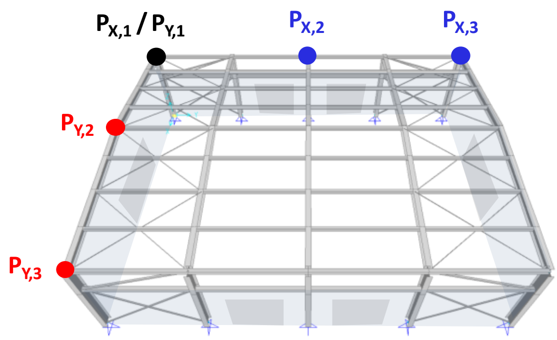
elements can be found in (Scozzese et al. 2018a, 2018b).
Material parameters are considered as deterministic, being the evaluation of the influence of model uncertainties beyond the scope of this study. Further information on model uncertainties can be found in (Scozzese et al. 2018b) where the methodology illustrated in (Franchin et al. 2018) was adopted.

Modelling of panels in OpenSees was made through couples of nonlinear truss elements adopted to reproduce the contribution of groups of assembled panels belonging to different structural fields. As already discussed, for each truss element, the Pinching4 uniaxial material available in the OpenSees library was adopted to simulate the nonlinear cyclic behaviour of the panel assembly, and the experimental results provided in (De Matteis and Landolfo 1999) and (De Matteis and Landolfo 2000) were used to calibrate the parameters required. The presence of concentrated openings was accounted by a reduction factor equal to 0.5 to both the stiffness and the strength of the truss elements. Details of the nonlinear finite element model of the non-structural elements can be found in (Scozzese et al. 2018a, 2018b).

3.3 Numerical results

Two different panel distributions are considered: 1) distribution with perfect symmetry along both X and Y directions (Figure 2a); 2) asymmetric distribution along the X direction, i.e., with cladding panels located within one extreme portal frame only (Figure 2b). Figures also show the opening locations, whose effects are numerically accounted by following the modelling strategy discussed. The influence of the cladding panels was studied by performing nonlinear static (pushover) analysis. It is worth noting that differential movements are possible among the portals (in X direction) and the braced frames (in Y direction), because of the lack of a rigid diaphragm at the roof level. Different control nodes were monitored in order to catch this phenomenon and consequently assess the impact on the results provided. Despite the relative movements are mainly expected along the portal direction (X), because of both the presence of a middle portal not restrained by roof braces and the asymmetric panel distribution, a set of three control nodes was also utilized to monitor the longitudinal (Y) direction. The assumed control nodes are identified in Figure 2. The analyses were performed in control of displacement, by pushing the building in each direction by means of sets of uniform horizontal forces applied at the nodes of the roof level.

(a)



(b)

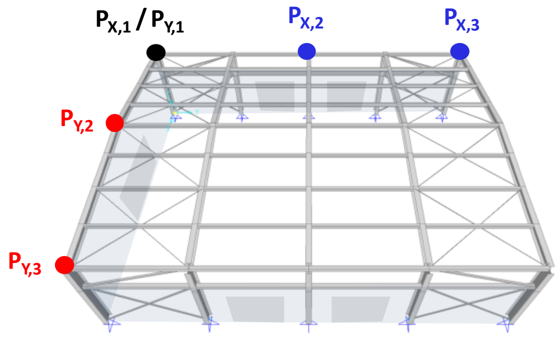


Figure 2. Scheme of panel distribution: (a) symmetric case; (b) asymmetric distribution along the X direction

The capacity curves for the case with symmetric panels are reported in Figure 3, while those related to the asymmetric panel distribution are shown in Figure 4. Curves related to different control nodes are superimposed in the charts by using different colours. It is worth noting that the choice of the control node has no influence for the response along the longitudinal direction, because of the symmetry of both structural and non-structural components. On the contrary, a higher sensitivity is observed in the X direction, due to the following reasons: (1) the presence of a middle portal not restrained by roof braces, which experiences higher displacements with respect to the lateral ones even for a symmetric panel assemblies' configuration (see Figure 3a); (2) the asymmetric panel distribution, which amplifies the effect said above on the central portal frame and, moreover, leads the extreme portal frame without panels to experience higher deformability, as witnessed by the pushover curves comparison in Figure 4a. It is also worth noting the asymmetric distribution of panels along X modifies the lateral response of the system, as can be observed by the different trends of the capacity curves in X of Fig. 3a (symmetric) and Figure 4a (asymmetric): hardening post-elastic behaviour with symmetric panels and softening branch with asymmetric ones. For what concerns the local damage criterion, the four performance states are highlighted on the capacity curves of Figure 3 and Figure 4 by coloured markers: a yellow square for the low-damage level; a red star for the medium-damage level; a red circle for the strong-damage level; a blue diamond for the base-shear limit.

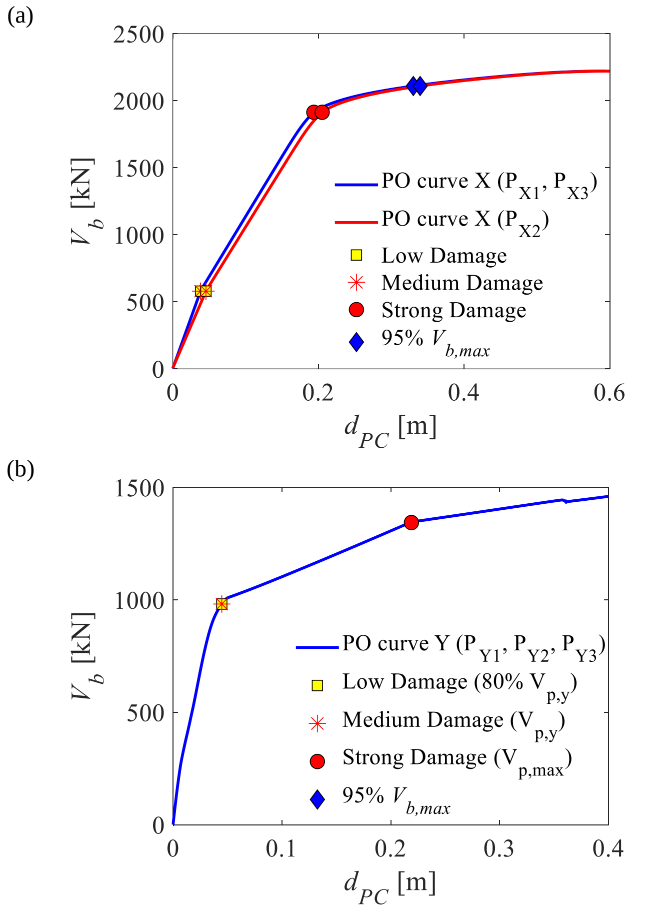


Figure 3. Pushover curves for symmetric panel distribution along X (MRF) (a) and Y (CBF) (b)

According to these results, the following comments can be done:

(a) low-damage and medium-damage levels are attained simultaneously, and this is due to the particular geometry of the building, with one storey only and panel assemblies of the same type and with the same heights along the two horizontal directions;

(b) the attainment of the limit on the base-shear (i.e., 95% $V_{b,max}$) is always posterior to the panel strong-damage state, except for the case with symmetric panels in X direction (Figure 4a), where a non-negligible structural damage occurs before the development of strong-damage level on panels; this testifies the worsening of the lateral structural response due to the presence of non-structural elements with asymmetric distribution in plan;

(c) as expected, no differences are observed in the longitudinal direction (Y) in terms of both capacity curves and panel performance points.

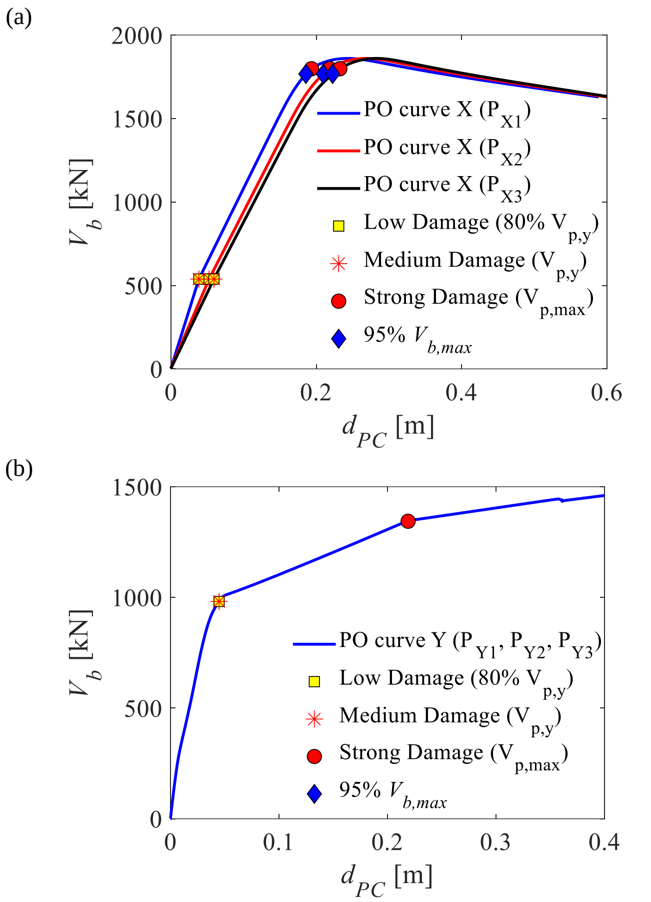


Figure 4. Pushover curves for asymmetric panels: direction X (MRF) (a) and Y (CBF) (b).

4 CONCLUSIONS

This paper focused on the influence of cladding panels on seismic response of non-residential single-storey steel buildings. Starting from a previous work carried out by the Authors, where cladding panels were explicitly included in the structural seismic analysis, the current study aimed at investigating the influence of cladding panels with non-symmetric distributions in plan. Moreover, the effect of the choice of the control node used to evaluate the drift ratio was analysed. In order to assess the onset of the Damage Limit State (DLS), a multi-criteria approach was adopted. According to the outcomes provided in this work, the following main conclusions can be drawn. Because of the specific building geometry, characterized by the absence of a rigid diaphragm at the roof level, the choice of the nodes for which the drift ratio is evaluated might affect the results, either underestimating or overestimating damage. An asymmetric distribution of panels could exacerbate the changes with respect to the control node.

For what concerns some possible future developments, these might be oriented to: (1) extend the current study to different typologies and different asymmetric configurations of cladding panels; (2) improve the local damage criterion and tailor it to better fit on single story buildings; (3) analyse the influence of structural and non-structural parameters on the obtained results through response sensitivity analysis; (4) evaluate the effect of model uncertainties both for structural and non-structural components. Regarding the second point, it is remarked that, due to the specific geometric features, all cladding panels experience almost simultaneously the same damage state, thus, reducing the usefulness of a criterion based on the attainment of a given damage condition by subsets of panel assemblies (such as, 50% of panels at a given damage condition).

ACKNOWLEDGMENT

The Authors would like to acknowledge the financial support of the Italian Civil Protection Department, ReLUIS project 2014-2018 (http://www.reluis.it/).

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