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Analysis of seismic non-structural damage in single-storey industrial steel buildings

Fabrizio Scozzese[a], Giusy Terracciano[b], Alessandro Zona[a,*], Gaetano Della Corte[b], Andrea Dall'Asta[a], Raffaele Landolfo[b]

[a] School of Architecture and Design, University of Camerino, Italy
[b] Department of Structures for Engineering and Architecture, University of Naples Federico II, Italy

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ABSTRACT

This study illustrates an approach that allows evaluating the intensity and extent of non-structural damage in steel buildings. Different levels of cladding panel damage are introduced and a specific cladding panel hysteretic model is included in the structural model and calibrated on available data from experimental tests. Numerical results from nonlinear dynamic analyses are elaborated in the form of fragility curves for the damage limit state and used to compare the consequences of different cladding behaviours on prototype single-storey industrial steel buildings used as case studies. The outcomes are also compared to a damage limit state derived from structural analysis adopting bare-frame models and conventional drift-based damage criteria.

1. Introduction

The seismic design and the structural analysis of steel constructions are commonly based on models where only the structural elements (e.g. beams, columns, braces) contribute to the definition of the strength and stiffness, while non-structural elements (e.g. cladding panels) are considered in the overall seismic mass but are not included as components explicitly influencing the structural response, e.g. [1,2]. Exceptions are few studies published in the past two decades that incorporated cladding panels in the structural model. Earlier investigations made in Italy [3–5] focused on lightweight sandwich panels in single-storey steel industrial building and developed hysteretic models for such panels based on experimental tests. Starting from the evaluation of the effect of cladding panels on the seismic response, the final goal was to propose a design approach where non-structural elements are designed to contribute bracing functions [6]. The same goal was pursued in studies made in Canada [7–11] that investigated the effect of cladding panels on the behaviour of the roof diaphragms and developed procedures for their seismic design. Similarly, researches made in Japan [12,13] studied the influence of precast lightweight concrete panels in steel frame structures and evidenced an enhanced seismic behaviour compensating the increment of structural mass.

A different perspective is tackled in the study here presented: (1) the seismic design is based on structural models where panels contribute only to the definition of permanent loads and mass while their stiffness and strength are neglected, according to the common practice (bare-frame models); (2) structural models that explicitly include the nonlinear hysteretic behaviour of panels are afterwards implemented and used to analyse the seismic response of the designed structures; (3) the results of the numerical simulations are adopted to evaluate the damage in the non-structural elements for increasing seismic intensities as a measure of the performance at the damage limit state (DLS) of steel buildings, e.g. [1,2], and how the results obtained compare to conventional drift-based code limits for damage verifications using bare-frame models, e.g. [14,15]. This last aspect is very important to properly address the seismic performance not only in terms of collapse under the most severe earthquakes, but also in terms of functionality and economic loss, as for example examined in recent studies [16,17] following the 2012 Emilia earthquakes in Italy and highlighting the important consequences of more frequent low-to-moderate seismic events. Within this context, numerical applications are presented using as case studies single-storey industrial steel buildings, a structural solution very common in many regions of Southern Europe with moderate to high seismic hazard. The case studies are designed according to the current version of the Eurocodes [14,15], neglecting the stiffness and strength of the claddings. Afterwards, a geometric and material nonlinear model that includes the hysteretic behaviour of vertical and roof claddings is described and adopted to perform nonlinear static and dynamic analyses, aiming at an examination of the damage level attained in each panel and the damage extent in the buildings.

* Correspondence to: University of Camerino, Viale della Rimembranza, Ascoli Piceno 63100, Italy.
E-mail address: alessandro.zona@unicam.it (A. Zona).

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2. Criteria for non-structural damage evaluation

The seismic performance of a building in terms of non-structural damage can be based on either local response quantities, describing the behaviour of the non-structural elements and attained damage, or global response quantities such as the interstorey drift. In general, attention should be paid to the relations between the accuracy of evaluation criteria based on global response parameters with respect to those based on local response parameters, as for examples investigated in [18]. This makes comparisons between the two approaches (local- and global-based) very useful to gain more insight into the possible limitation of damage criteria based solely on global response parameters.

In this study the damage evaluation of the non-structural elements is pursued through their inclusion in the structural model. Being the engagement of the non-structural elements in the seismic response explicitly described, two outcomes are achieved: (1) estimation of the local seismic demand on the non-structural elements; (2) estimation of the influence of the non-structural elements on the global and local seismic response of the building as compared to the bare-frame model. Since non-structural elements are included in the structural model, a performance-based approach for DLS assessment can be derived monitoring different damage conditions attained by the cladding panels under seismic events. Specifically, the local panel-related damage criteria proposed to characterize the attainment of the DLS for a structural system are listed and described in Table 1. Given the lack of sufficiently large studies on this topic, the adopted damage conditions are based on engineering judgment while the implemented approach is inspired by recent studies made for masonry infill walls used in reinforced concrete buildings [19].

3. Nonlinear modelling of cladding panels

3.1. Hysteretic model

The inclusion of the non-structural elements in the nonlinear model of the building requires that the nonlinear cyclic behaviour of each panel (or assembly of panels) is simulated through properly calibrated hysteretic models. The availability of a suitable hysteretic model is fundamental to realistically simulate the cyclic response of the cladding panels. To this aim, the Pinching4 [20] one-dimensional model implemented in OpenSees [21] is adopted to catch the main features of the shear-displacement experimental response observed for non-structural elements as used in steel constructions in Europe [4,5], i.e. the pinched shape of the cycles and the strength and stiffness deterioration under cyclic loading.

In the Pinching4 implementation, the hysteretic model is governed by 34 parameters, 16 control the multilinear envelope curve (response backbone curve), 6 control the trilinear unload-reload path, and 12 set the properties of three possible hysteretic damage rules, i.e. unloading stiffness degradation, reloading stiffness degradation and envelope strength degradation. For the purposes of the present investigation, the aforementioned parameters are calibrated in order to fit the cyclic response of the cladding panels considered in this study for which experimental tests are available in the scientific literature, e.g. [4,5].

Table 1
Panel-related damage conditions.

Damage level	Description
Low-damage	An assigned percentage of panels exceed their elastic response limit (in this study investigations are made using 30% and 50% values);
Medium-damage	The totality of panels exceed their elastic response limit;
Strong-damage	At least one panel exceeds its maximum shear resistance, thus, providing negligible contribution in terms of both stiffness and strength.

3.2. Example of calibration based on experimental tests

Two different cladding panels are considered in this work, hereafter referred to as type A and C according to the terminology adopted in De Matteis and Landolfo [5]. These panels belong to a typology of lightweight sandwich panels which are widely used as enclosure elements in both industrial and civil buildings [4,5]. Both typologies consist of sandwich panels made of external steel sheets (thickness of 0.6 mm) and insulating polyurethane core for a total thickness of 40 mm. Embossed sheets with slight stiffening ribs are provided to the panel type A, while completely flat sheets are employed for the type C panel. Furthermore, the type C panel edges are reinforced with internal cold formed channel profiles connected to the sheets through aluminium rivets of 6.3 mm diameter. Panels are usually connected to the main structural frame through proper cladding rails (vertical cladding panels) or purlins (roof panels) by means of bolts (generally of 8 mm in diameter and 110–120 mm spacing) [5].

Each panel is modelled in OpenSees by means of a couple of diagonal truss elements with nonlinear axial behaviour following the Pinching4 model presented above. For both panel typologies the parameters governing the shear-displacement relationship are calibrated based on the experimental results available in [4,5], consisting of cladding sandwich panels type A and C (dimension $1000 \times 2500 \times 40$ mm) subjected to cyclic shear loading.

The backbone curve for panel A is schematically represented in Fig. 1a, superimposed to the envelope of the experimental cyclic response. It consists of a trilinear model characterized by: a yielding point at 12 kN (d = 20 mm), a hardening behaviour until the maximum shear of 17 kN (d = 80 mm), and a gradual softening branch where the force drops to zero at d = 320 mm. The simulated quasi-static nonlinear response for panel A is shown in Fig. 1b, and it is worth noticing its satisfactory match with the pinched experimental shear-displacement response (Fig. 1a).

The backbone curve for the panel C, shown in Fig. 2a, consists of a quadrilinear model having the following features: a first yielding point at 11 kN (d = 5 mm), followed by a branch with slightly reduced stiffness until the maximum strength of 23 kN (d = 25 mm), and a perfectly plastic response extended until the 80 mm of displacement, followed by a gradual softening behaviour with the null shear force attained at d = 320 mm. The simulated quasi-static nonlinear response for panel C is reported in Fig. 2b.

It is important to clarify that the latter part of the backbone curves (softening trends) for both A and C panels were extrapolated from the experimental results, trying to realistically describe the complex deteriorating behaviour (panels buckling, connections failure, etc.), although the available test results do not characterize the response beyond the 80 mm displacement threshold (Fig. 1a and Fig. 2a).

4. Numerical application

4.1. Case studies

The structural system investigated in this paper consists of single-storey industrial steel buildings composed of five single span duo-pitch portal frames equally spaced (Fig. 3). The portal frames are connected in the longitudinal direction by hot-rolled beams at the apex, at the eaves and at the crane-supporting bracket level. Horizontal forces are withstood by two different seismic-resistant mechanisms: in the transverse direction (X-direction) the resistance is due to continuous and rigid frame action, which exploits the moment resisting connections between beams and columns; along the longitudinal direction (Y-direction) the resistance to horizontal forces is provided by vertical bracings placed in the outer spans of the building. Purlins, used to support