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AN EXPERIMENTALLY VALIDATED MODELLING TECHNIQUE FOR THIN RC SANDWICH WALLS UNDER SEISMIC LOADINGS

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Abstract

The seismic performance assessment/simulation of a novel structural system composed of lightly reinforced concrete sandwich panels characterized by thin concrete layers reinforced by low diameter steel wire mesh of particularly reduced diameter is the subject of the paper. The construction system (which is obtained by spraying thin layers of shotcrete upon a prefabricated polystyrene support panel) has high thermal and structural performances as well as cheap construction cost, so that it caught the attention of several developing countries. The assessment/modelling of the cyclic non-linear behavior of such system is particularly challenging but fundamental in order to fully exploit its earthquake-resistant capacities. In the present work a modelling technique (which make use of a recently proposed concrete damage model) for such structural system is proposed and validated through experimental results in terms of both force-displacement response and damage progression.

Keywords: lightly reinforced concrete sandwich panels, seismic behavior, damage model

1. Introduction

Reinforced concrete (RC) sandwich panels are typically composed of two external concrete layers separated by an internal insulating layer. Once in place, the sandwich wall panels provide the dual function of load carrying capacity and insulation. They may be used solely for cladding (i.e. non-structural components), or they may act as bearing walls, or shear walls. The panels are typically precast at a manufacturing plant, trucked to the construction site and erected by cranes. Sometimes only the formworks are precast at manufacturing plants, and the structural concrete is cast in situ after the formworks and additional reinforcements are positioned (see http://www.nidyon.net).

The interest in sandwich panels has increased in the past few years because they exhibit a mix of desirable characteristics such as durability, economical convenience, fire resistance, vertical and horizontal load bearing capacity as well as excellent insulation properties providing superior energy performance as compared to many other walls. However, a limited number of investigations have been published on the structural behavior of sandwich panels and on their seismic performances (Seeber [1]).
Among these structural system, of particular interest are the ones developed by two Italian companies (Nidyon Costruzioni and Emmedue S.p.A.) due to their particular low cost of construction and the monolithic aspect of the final construction. Nonetheless, at the moment no modelling techniques appear to be readily available and fully validated for the assessment of the non-linear seismic behavior of such panels which is of fundamental importance to fully exploit their potential properties (in terms of ductility and dissipation of seismic energy) in seismic areas.

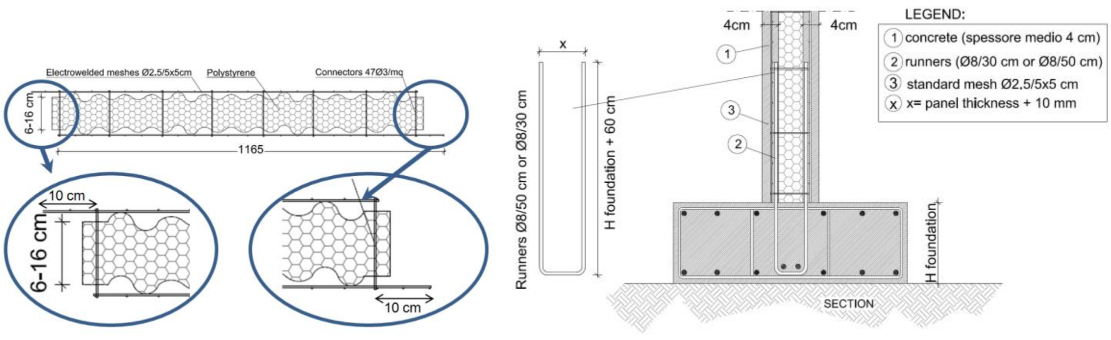
The aim of this paper is to present a model for such a system which is validated through the results of an experimental campaign carried out in Bologna and Pavia (Palermo et al. [2] and Ricci et al. [3]). The validation is aimed at: (i) reproducing the global force-displacement behavior and (ii) providing reasonable indications of the damage in terms of localization, extent and level.

2. The structural system and the experimental campaign

2.1 The main characteristics of the structural system

The novel structural system here analysed is based on the production and use of prefabricated reinforced polystyrene panels (referred to as modular panels) with length of 1120 mm and height equal to the interstorey height. The panels are made of a single expanded polystyrene layer (with thickness between 60÷160 mm), reinforced by two nets of Ø2.5 mm steel wire mesh (spaced at 5 cm) anchored to the external faces by means of Ø19 mm through steel ties, that connect also the opposite nets.

Once erected at the construction site, the modular panels act as support for the cast of the external 4 cm tick concrete layers (e.g. shotcrete). Additional reinforcements (typically 1+1 Ø 12 bars and Ø 8/50 cm U-shaped bars, made up of B450C steel) are added: (i) around the openings (doors and windows), (ii) at the lateral edges of the wall (to provide extra strength along the side). The connections between the walls and the foundations are made through U shaped Ø 8 mm anchor rods typically spaced at 50 cm. The amount of reinforcement provided by the steel meshes (Ø 2.5/50x50 mm) results in a low reinforcement ratio ρ=0.245%. Figure 1 provides a schematic representation of the modular panel and wall-foundation connection. It has to be noted that no specific prescriptions and recommendations are actually available in the Italian building code (NTC08) for constructions made through such technology.



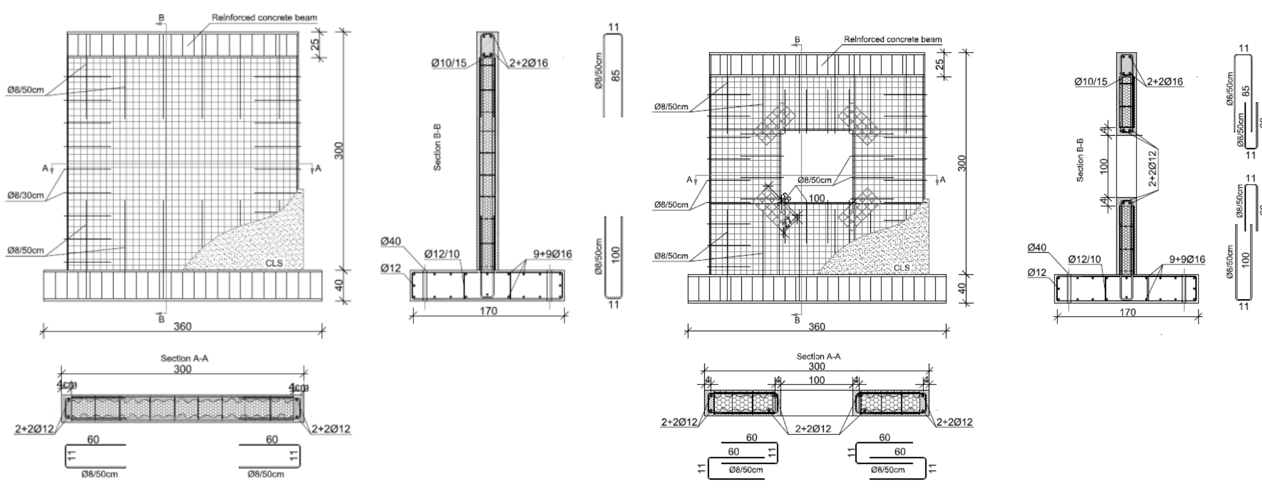
(a) (b)

Fig. 1 –(a) The modular panel; (b) typical wall-foundation connection.

2.2 The fundamental results of the experimental campaign

To assess the seismic performances of RC sandwich panels, the University of Bologna and the Eucentre labs of Pavia (Italy) jointly carried out an experimental campaign. A total of five Planar Wall (PW) specimens were tested under cyclic horizontal loads with load reversals and a constant vertical load (the vertical load was kept constant throughout each test, but its value differed from test to test as indicated in Table 1). A complete description of the experimental results is available in the work by Ricci et al. [3]. In the present section the relevant information for the complete understanding of the numerical modelling will be provided. The five planar walls are composed by a single square panel with 3m side length. Three out of five panels have no openings (namely PW1, PW2 and PW3), while the reaming two have a 1m x 1m central opening (namely PW4 and PW5). The total panel thickness is 18 cm with a central expanded polystyrene core 10 cm-thick, and two external 4 cm-thick shotcrete layers. The connections between panel and foundation are those used in practice (U shaped Ø 8 mm anchor rods spaced at 50 cm, as already mentioned before).

The geometry and the reinforcement drawings of each sandwich panel are given in Figure 2. A summary of the main characteristics of each specimen is presented in Table 1. The concrete cubic compressive strength at 28 days is 30 MPa. The steel yield strength was 450 MPa. The vertical load ratio is defined as N/(A_c fc), where N is the applied vertical load, A_c is the concrete gross section, fc is the concrete compressive strength.



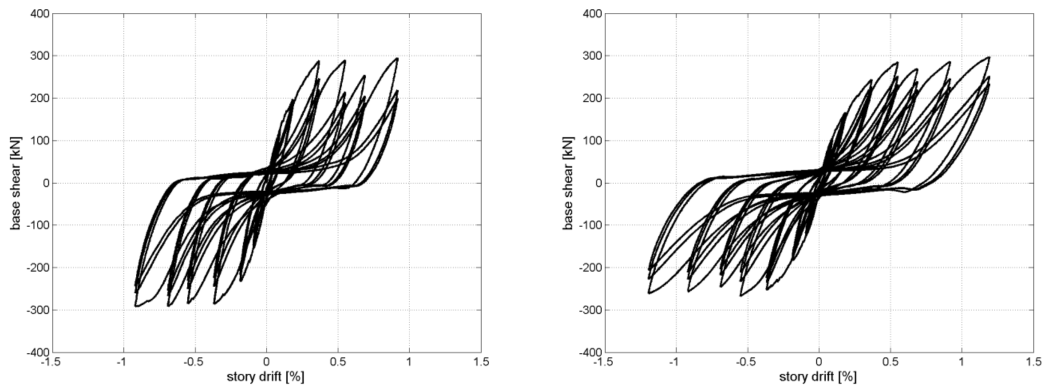
(a) (b)

Fig. 2 –Reinforcement layout: (a) solid sandwich panel; (b) sandwich panel with a central opening. (Adapted from Ricci et al. (2013)).

Table 1 – Specimen properties

ID	Wall type	Dimensions [m] x [m]	Opening dimensions [m] x [m]	Self-weight [kN]	Vertical load [kN]	Vertical Load Ratio (VLR)
PW1	plane	3.0 x 3.0	-	19.9	50	0.87%
PW2	plane	3.0 x 3.0	-	19.9	100	1.73%
PW3	plane	3.0 x 3.0	-	19.9	250	4.35%
PW4	plane	3.0 x 3.0	1.0 x 1.0	17.9	50	0.87%
PW5	plane	3.0 x 3.0	1.0 x 1.0	17.9	100	1.73%

The global experimental response in terms of base shear vs. normalized storey drift (i.e. the drift as recorded at the top divided by the height of the specimen) for one solid panel and one panel with a central opening is represented in Figure 3. Hereafter the normalized storey drift will be simply referred to as drift. In the work by Ricci et al. [13] it was concluded that the overall seismic performances of the tested panels in terms of stiffness, strength and ductility was similar to that of conventional RC shear walls characterized by similar geometrical and mechanical properties (concrete strength, reinforcement ratio).



(a) (b)

Fig. 3 – Base shear versus normalized interstorey drift (adapted from Ricci et al. 2013) in specimens: (a) PW1 (solid, VLR =0.87%); (b) PW4 (opening, VLR =0.87%).

Nonetheless, the shape of the hysteretic loops evidences a significant amount of pinching as well as a clear strength degradation (especially between the first and second cycle within the same imposed drift level). In general terms, both phenomena are also experienced by standard squat RC walls (see, for instance, the experimental response of the RC squat walls tested by Hidalgo et al. [4], realized according to ACI 318 prescriptions [5]), even though with smaller evidence. As well known, pinching and strength degradation (due to repeated cycles or increasing drifts) in standard RC shear walls are typically related to damage due to opening-closing of cracks, bond slip between concrete and bars, and in minor part to little sliding between wall and foundation.

Given the peculiar characteristics of the tested panels, two main reasons could probably have caused a more pronounced pinching and strength degradation: (i) the presence of a fine spaced grid made of steel wires (Ø 2.5 mm smooth bars); (ii) the steel connectors at the base which yielded prior the wall experienced its full non-linear behavior.

3. A nonlinear model for the sandwich panels

3.1 Modelling issues

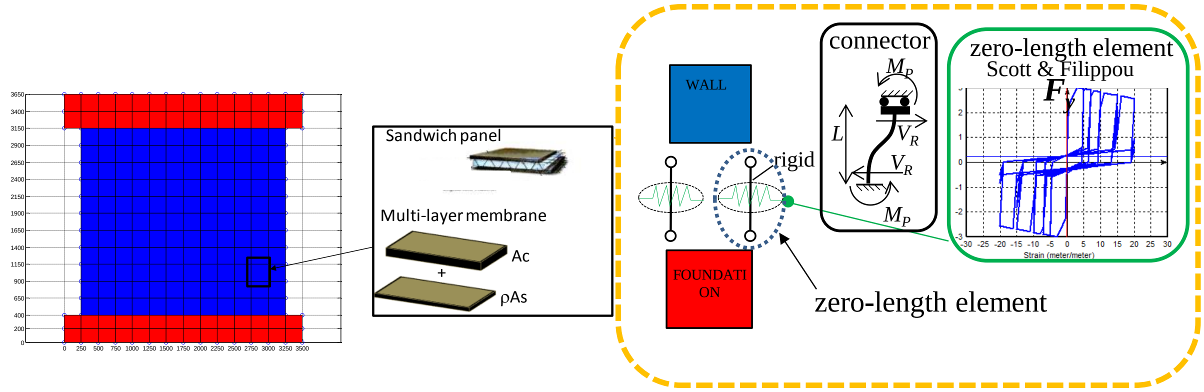
In order to account for the specific features of the tested sandwich panels and panels-foundation connections, the authors decided to model (i) the sandwich panel with a multi-layer membrane model integrating the concrete damage material model, developed by researchers in Padua (Tesser et al. [6]), with the Menegotto-Pinto steel model incorporating isotropic hardening (Filippou et al. [8]) and (ii) the connections with non-linear special elements (Mazzoni et al. [9]) as detailed in section 4.3. The use of multi-layer membrane element have already proved to balance accuracy and computational efficiency for the cyclic nonlinear analysis of standard reinforced concrete panels and walls (Tesser et al. [6, 7]) .It has to be clarified that the authors of this work did only contribute in the testing (validation) phase of the concrete damage material model and not in its development. At the moment the concrete damage model is not yet available in any commercial structural analysis software. The numerical simulations, whose results will be presented in the next section, have been developed by using the research open source software Open System for Earthquake Engineering Simulation (OpenSees) developed at University of California, Berkeley (Mazzoni et al. [9]).

The single panel is modelled with equivalent multilayer membrane elements, which are composed by: (i) one layer of concrete with the constitutive behavior described by the damage model by Tesser et al. [6] and (ii) one layer of steel with the Menegotto-Pinto constitutive behaviour (Filippou et al. [8]). The concrete layer has a thickness equal to the sum of the two concrete layers (t_c). The steel layer has a thickness equal to $\rho \cdot tc$ (according to the smeared approach). The two membranes are connected so that no relative slip can occur (i.e. perfect bound between concrete and reinforcement is assumed). Figure 4a provides a schematization of the numerical model of the single panel. The polystyrene insulation layer is not included in the model, given that its elastic modulus is three order of magnitude smaller than that of concrete.

The connections at the base of the walls are modelled with zero-length elements, which are interposed between the wall and the foundation. A zero-length element (available in the Opensees library) connects two nodes having the same initial position. The element can be described by any force-displacement relationship. In this specific case, the zero-lengths elements are active along the horizontal direction only (in other words only relative horizontal displacements are admissible) and their hysteretic response is modelled using the Scott and Filippou hysteretic model available in the OpenSees library. The remaining degree-of-freedoms are condensed (i.e. are rigidly connected). A sketch of the model of the panel-foundation connection is given in Figure 4b. The initial yield strength of each individual zero-length element is set to be equal to:

$$F_y = \frac{sV_R}{n} \tag{1}$$

where s is the spacing of the connectors, n is the number of zero-length elements. $V_R = 4M_P / L$ is the shear strength of the single connector having a plastic bending moment M_P (evaluated assuming a double-clamped schematization with unbonded length L of roughly 1 cm).



(a) (b)

Fig. 4 – (a) The panel; (b) panel-to-foundation connection.

3.2 The models of the tested panels

The mesh geometry of each PW is made of four-node membrane elements (Dvorkin and Bathe [10]) with side length of 25 cm. The foundation and the loading beam are modelled with linear elastic elements. The fundamental parameters of the concrete material model have been obtained from the results of the standard compression tests conducted on concrete cylindrical samples (average compressive strength equal to of 26 MPa, average elastic modulus equal to 25000 MPa).

The concrete tensile strength has been set equal to zero due to experimental evidences of pre-cracks in terms of both stiffness and strength, as mentioned in section 3.3. The influence of the tensile strength on the panel has been also evaluated by comparing the numerical response of the panel under monotonically increasing lateral loads (pushover curves) and the envelope of the experimental cyclic response. The remaining model parameters (describing the non-linear hysteretic material behavior) have been directly suggested by the authors of the model and refers to the average non-linear behavior of standard RC squat walls.

The steel yield strength is set equal to 450 MPa. Zero-length elements are interposed between the wall and the foundation, thus allowing relative horizontal displacements between the wall and foundation (type A constrain). In addition, models with perfect continuity between wall and foundation has been developed in order to under investigate of the influence of the connectors on the cyclic behavior (type B constrain).
Table 2 summarized the models developed.

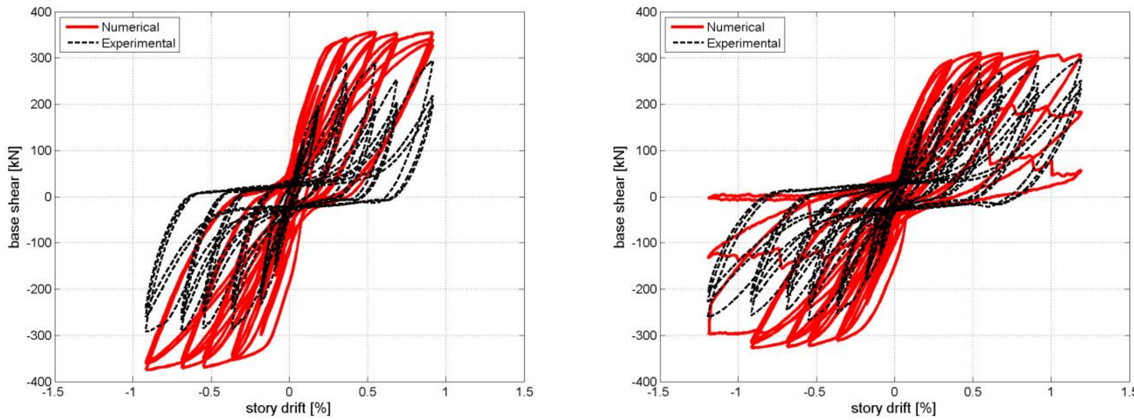
Table 2 – The numerical models

	Specimen PW1	Specimen PW2	Specimen PW3	Specimen PW4	Specimen PW5
Type A constrain	PW1-A	PW2-A	PW3-A	PW4-A	PW5-A
Type B constrain	PW1-B	PW2-B	PW3-B	PW4-B	PW5-B

4. The simulation of the experimental tests

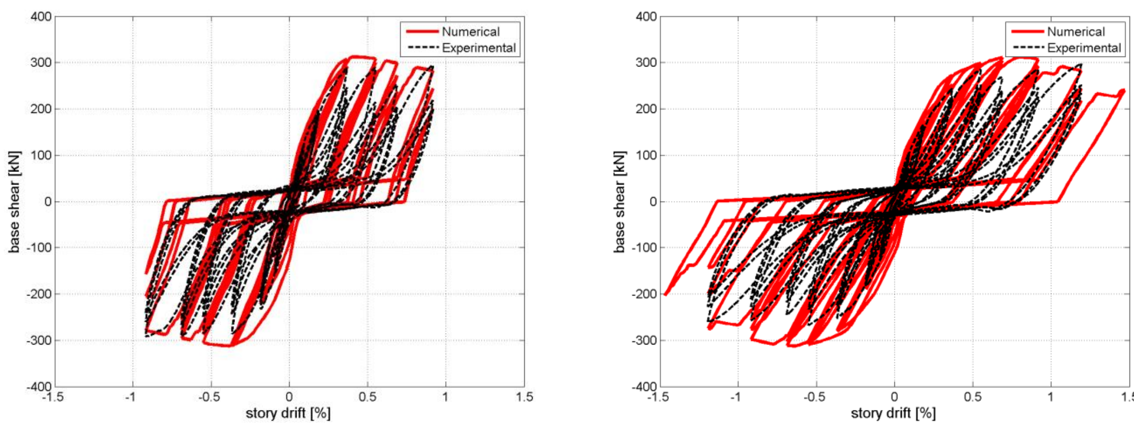
4.1 Force-displacement response

The force-displacement cyclic responses as obtained from the numerical simulations on models with type A and type B constrains are represented in Figures 5 and 6, respectively. It can be noted that the concrete damage model used to model the panel and the hysteretic model of the connections leads to a accurate evaluation of the shear capacity and to a good description of both hysteretic loops and pinching (Figure 6). On the contrary the models with perfect continuity between walls and foundations do not allow to describe adequately the hysteretic loops and pinching, thus indicating a significant hysteretic response of the connectors.



(a) (b)

Fig. 5 – Experimental vs. numerical (type A constrain) cyclic response: (a) PW1 and (b) PW4.



(a) (b)

Fig. 6 – Experimental vs. numerical (type B constrain) cyclic response: (a) PW1 and (b) PW4.

4.2. Damage maps: Type A vs Type B base constrains

The damage evolution in terms of damage maps as obtained from monotonic pushover analyses performed on one solid panel and one panel with a central opening, considering type A constrain at the base, are shown in Figure 7 and 8.

It can be noted that the initial resisting mechanisms of the full panels can be clearly identified with the diagonal compressed strut from the top left corner to the bottom right corner. The strut width can be also appreciated. As the lateral drift further increases the damage concentrates to the bottom corner (red means loss of strength, e.g. concrete crushing). From a qualitative point of view the damage mechanism (diagonal cracks followed by concrete crushing at the corner base) reflects the experimental observations. Nonetheless, at a given drift, the damage level as obtained from the numerical simulation is higher than the damage observed during the experiments. For instance, in the numerical simulations the corner crushing initiated around 0.90% drift, while during the tests the corner crashing was observed for drifts larger than 1%. The panels with the central opening show a similar resisting mechanism (diagonal strut) and a similar mechanism of failure (concrete crushing at the corner base), even though the corner crushing is anticipated at drift values around 0.75%, thus even smaller when compared with the numerical responses of the solid panels.

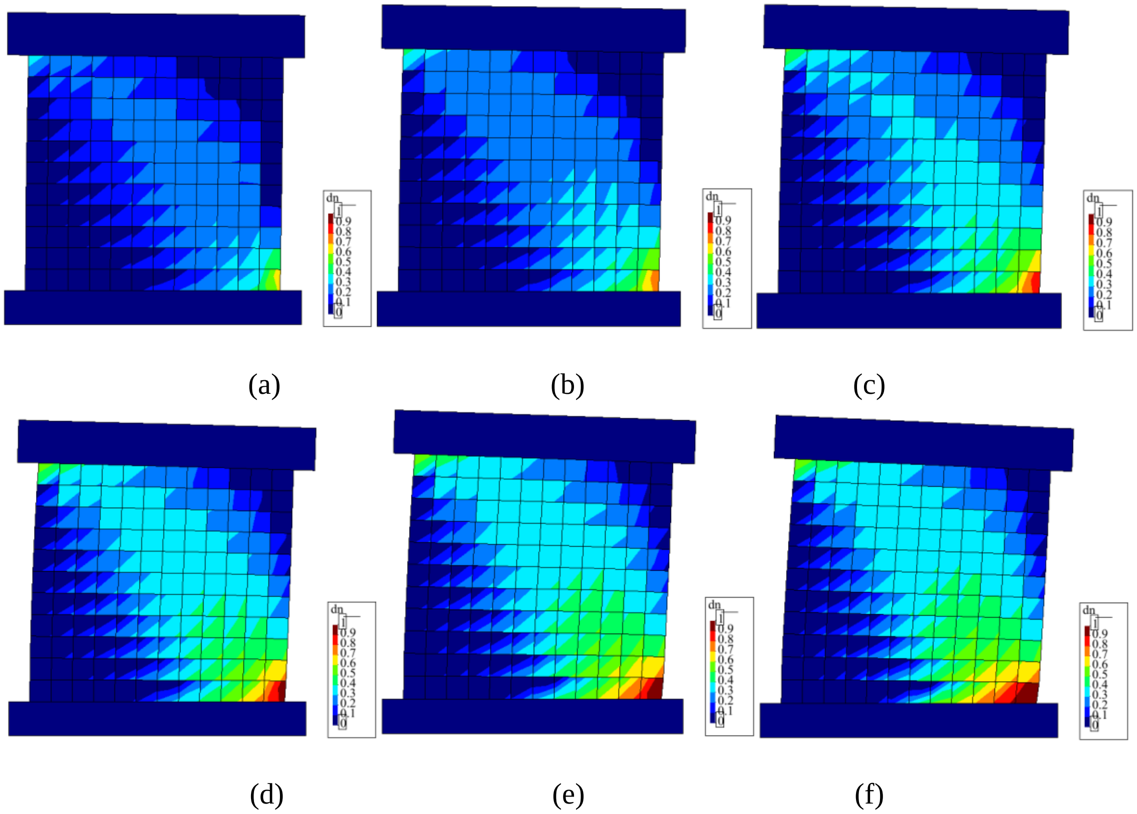


Fig. 7 – Damage maps for model PW1-A at selected drifts: (a) 0.25%; (b) 0.35%; (c) 0.50%; (d) 0.70%; (e) 0.84%; (f) 0.95%. (Deformations are magnified).

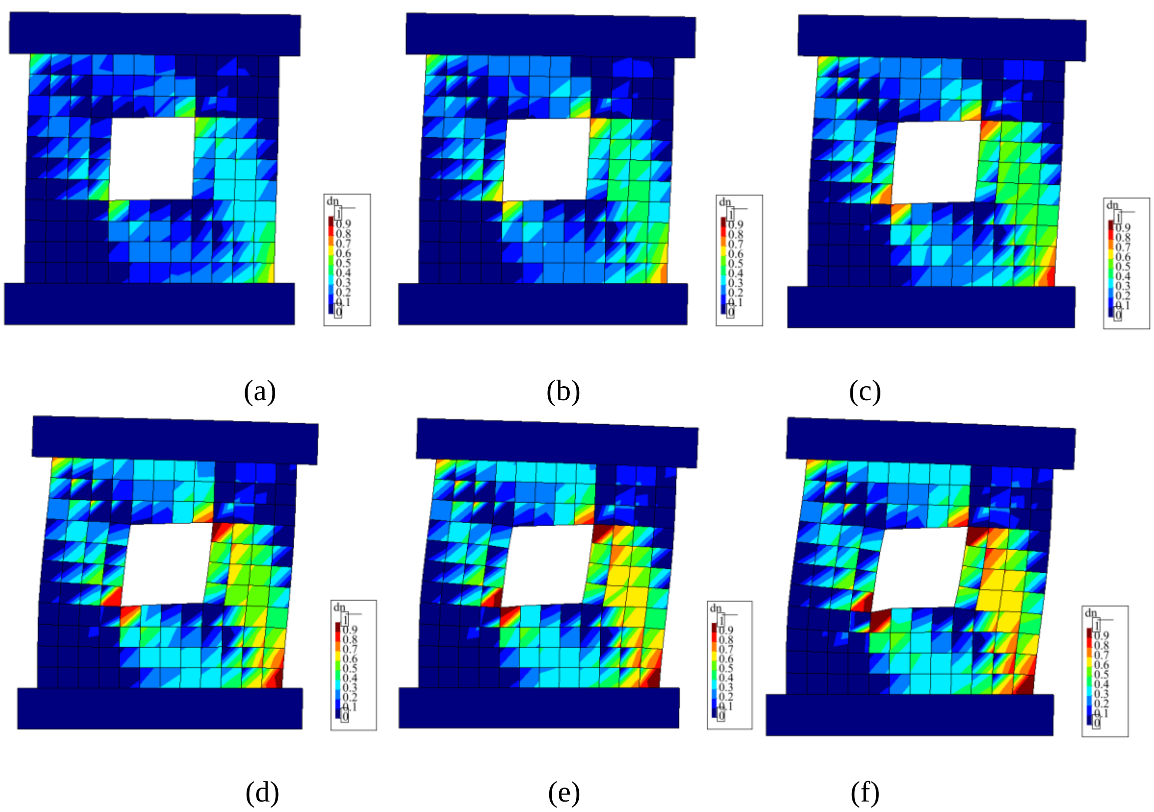
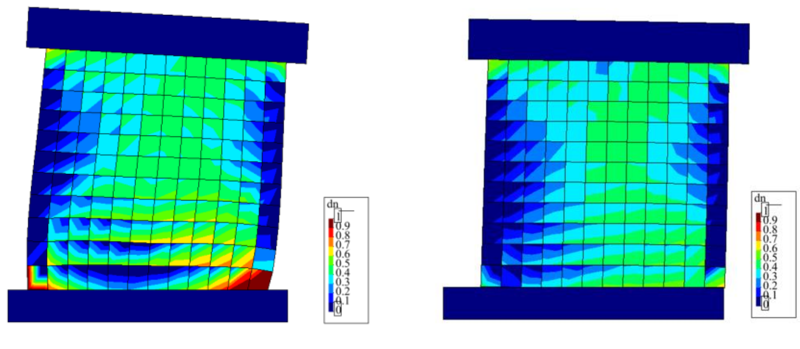
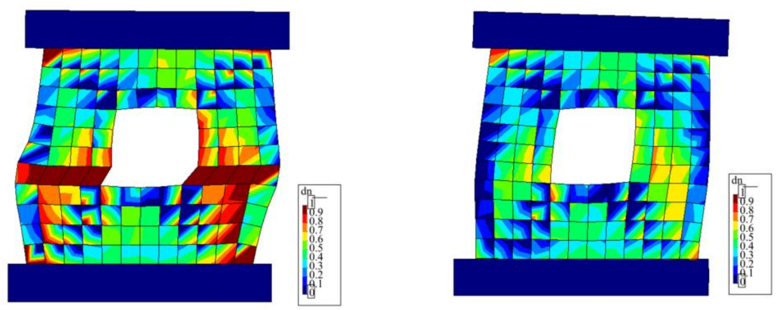


Fig. 8 – Damage maps for model PW4-A at selected drifts: (a) 0.12%; (b) 0.20%; (c) 0.30%; (d) 0.40%; (e) 0.55%; (f) 0.70%. (Deformations are magnified).



(a) (b)

Fig. 9 – Damage maps at 1.0%. drift from reversed cyclic simulations : (a) PW1-A model (b) PW1-B model.



(a) (b)

Fig. 10 – Damage maps at 1.0%. drift from reversed cyclic simulations : (a) PW4-A model (b) PW4-B model.

Figures 9 a and b show the damage maps at 1% drift for a full panel as obtained from the cyclic simulations on PW1-A and PW1-B models, respectively. From Figure 9b it is clear that the insertion of the zero-length elements allowing relative horizontal displacements between the wall and the foundation in PW1-B model, leads to a significant reduction of the damage in the panel which is consistent with the experimental observations. A similar comparison is showed in Figure 10 and b for a panel with the central opening (PW4-A and PW4-B models, respectively). In this case, inserting zero-length elements at the base (PW4-B model) allows the occurrence of a relative slip between the wall and the foundation, and clearly appears to change the mechanism, similarly to what is shown by the experiments.

5. Critical considerations

The experimental evidences and the main results of numerical analysis here presented allow to make some interesting comments on the seismic behavior and modelling of the studied system. First, based on the comparisons showed in the previous section, the structural system can be adequately modelled through: (a) equivalent multi-layer membrane elements composed by one layer of thickness equal to the sum of the two external concrete layers plus a steel layer of equivalent thickness as resulted by the smeared approach accounting for the contribution of the steel reinforcement with constitutive law according to the Menegotto-Pinto model, (b) zero-length elements (active along the horizontal direction only) to model the connections at the base. The use of such modelling techniques would allow to well reproduce the overall non-linear force-displacement response and to adequately describe the damage evolution, thus suggesting its potential use for the seismic analyses of real structures realized with a similar technology. In this regard, it has to be noted that very few experimental tests has been conducted on large scale structures. For instance, a shacking table test on a 3-storey full scale building entirely made of RC sandwich panels (including the slabs) has been recently carried out by the authors (Palermo et al. [11]).

Nonetheless, due to a large unexpected concrete tensile strength, the building remained almost within the elastic field also for a seismic input of 1.2 g (close to the table capacity), and the nonlinear behavior of both the panels and the connections could hardly be exploited. The main advantage (from designers point of view) in using a concrete damage model is, in addition to its computational efficiency, the possibility to visualize the damage progression with the damage maps. Damage maps may assist the practitioner in the identification of the resisting mechanisms, attainment of limit states and accumulation of physical damage, as showed by the authors also in another recent work (Tesser et al. [8]). Nonetheless, as showed in the present paper, the identification of reference values of the local damage variables corresponding to the achievement of a certain limit state (e.g. first concrete spalling, "wall yielding", concrete crushing, ...) requires a critical and expert judgement and the availability of experimental data, since the values of the local damage variables are, in general, affected by the size of the mesh (i.e. localization effects). From a practical perspective, as an alternative to using local damage variables, a global damage index could be introduced (based on an appropriate "weighted" integration of the local values, thus requiring additional post processing of the simulation results). A similar approach has been already proposed in the scientific literature for the analysis of frame structures made of beam elements leading to the definition of a global damage index (Scotta et al. [12]). The implementation of such global damage-index and its rendering by means of damage maps may help in identifying the achievement of the various limit states, to the advantage of designers who may perform their safety checks with no need of mastering damage mechanics.

6. Conclusions

In the present paper, a non-linear model suitable for the seismic analysis of building structures made by RC sandwich panels, characterized by thin concrete layers and low amount of reinforcement, has been proposed. The model has been validated against experimental results from cyclic tests on single solid walls and walls with a central opening.

The comparisons between experimental and numerical results showed that the model is able to well reproduce the global in-plane force-displacement response and to adequately describe the evolution of the mechanisms of damage. Data and information gathered from experimental and numerical results may be used to enhance actual codes prescriptions for the design of constructions made by RC sandwich panels.

References

[1] Seeber KE (1997). PCI Committee on Precast Concrete Sandwich Wall Panels. State of the art of precast/prestresses sandwich wall panels. PCI J 1997;42(2):92–133.

[2] Palermo M, Gil-Martín LM, Trombetti T, Hernández-Montes E. (2013). In-plane shear behavior of thin low reinforced concrete panels for earthquake re-construction. Materials and structures, 46(5), 841-856.

[3] Ricci I,Palermo M, Silvestri S, Gasparini G, Trombetti T (2013). Results of pseudo-static tests with cyclic horizontal load on cast in situ sandwich squat concrete walls. Engineering Structures, 2013. DOI 10.1016/j.engstruct.2013.03.046.

[4] Hidalgo PA, Ledezma CA, Jordan RM. (2002). Seismic behavior of squat reinforced concrete shear walls. Earthquake Spectra; 18(2):287–308.

[5] ACI 318-11 (2012) Building Code Requirements for Structural Concrete and Commentary. American Concrete Institute. Retrieved 8 Aug.

[6] Tesser L, Filippou FC, Toledo DA, Scotta R, Vitaliani R (2011). Nonlinear analysis of R/C panels by a two parameter concrete damage model. In: III ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering COMPDYN, Corfu, Greece, 25-28 May 2011, ISBN: 978-960-99994-0-3 (CD ROM proceedings).

[7] Filippou FC, Popov EP, Bertero VV (1983). Effects of Bond Deterioration on Hysteretic Behavior of Reinforced Concrete Joints. Earthquake Engineering Research Center, University of California, Berkeley.

[8] Tesser L, Palermo M, Filippou F (2014). Nonlinear analysis of shear wall structures by an efficient reinforced concrete membrane element. In: Proceedings of 2nd European Conference on Earthquake Engineering and Seismology (2ECEES), 24–29 August, Istanbul, Turkey.

[9] Mazzoni S, McKenna F, Scott MH, Fenves GL, et al. (2006). OpenSees Command Language Manual. Available on-line at http://opensees.berkeley.edu/OpenSees/manuals/usermanual/.

[10] Dvorkin EN, Bathe K-J (1984) A continuum mechanics based four node shell element for general nonlinear analysis, Eng.Comput.,1,77-88.

[11] Scotta R, Tesser L, Vitaliani R, Saetta A. (2009). Global damage indexes for the seismic performance assessment of RC structures. Earthquake Engineering & Structural Dynamics, 38(8), 1027-1049.