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FEM MODEL OF PLASTERBOARD PARTITIONS FOR SEISMIC CAPACITY EVALUATION

C. Petrone[(1)], O. Coppola[(2)], G. Magliulo[(3)], P. Lopez[(4)], G. Manfredi[(5)]

[(1)] *Research Associate, Department of Civil, Environmental & Geomatic Engineering, University College London, c.petrone@ucl.ac.uk*
[(2)] *Research Fellow, Construction Technologies Institute, National Research Council of Italy, orsola.coppola@itc.cnr.it*
[(3)] *Assistant Professor, Department of Structures for Engineering and Architecture, University of Naples Federico II, gmagliul@unina.it*
[(4)] *Mechanics expert, Product Development Department, Etex Dry Co, Avignon, France, pauline.lopez@etexgroup.com*
[(5)] *Full Professor, Department of Structures for Engineering and Architecture, University of Naples Federico II, gamanfre@unina.it*

Abstract

The paper deals with the in plane seismic performance assessment of internal partitions. A finite element model capable to capture the inter-story drift ratio which causes the failure of plasterboard partition is presented. Both the steel stud and the plasterboards, constituting the panel, are modelled through linear elastic elements. The nonlinearity is lumped in the panel-to-stud screwed connections. A tri-linear force-displacement backbone curve is assigned to the screwed connections matching the experimental results of monotonic tests on such a connection. The Direct Strength Method is applied to assess the occurrence of different buckling failure modes, i.e. local, distortional and global failure modes. This method allows considering the restraining effect given by both the presence of the plasterboards and the screwed connections through the presence of linear springs on the steel stud cross section.

The validation of the proposed model is performed by comparing the analytical behavior with the experimental evidence of a quasi-static test campaign on a plasterboard partition conducted at the Laboratory of the Department of Structures for Engineering and Architecture at the University of Naples Federico II. The comparison between the numerical results and the experimental evidence on a plasterboard partition shows that the model well catches the buckling failure modes of the specimen. In particular, the inter-story drifts which cause either the local or the global buckling in the partition are well predicted. Moreover the analytical model highlights that the demand in the steel stud is concentrated across the two horizontal joints between the plasterboards, justifying the damage evidenced during the experimental test exactly under and over the joints.

Keywords: plasterboard partition; screw connections; numerical model; seismic performance; interstory drift

1 Introduction

In the last years several studies dealt with the seismic performance of nonstructural components. Nonstructural components are usually defined as secondary structures, since they are not designed to bear horizontal forces or vertical loads. Nevertheless, they must still have suitable features to ensure their integrity in the aftermath of an earthquake. Indeed, the damage of nonstructural components can have significant consequences on the operability of strategic buildings, on the human life safety, but can also have a relevant economic impact related to the post-earthquakes retrofitting actions. Taghavi and Miranda [1] evidenced that structural cost typically represents a small portion of the total cost of a building construction, corresponding to 18% for offices 13% for hotels and 8% for hospitals. Plasterboard internal partitions, object of the present study, are very common nonstructural components, typically employed in several building typologies all over the world. Different experimental studies aiming at the evaluation of the seismic capacity of such components are available in literature [2; 3]. Analytical models of plasterboard partitions are also investigated to simulate the seismic behavior of such components. In Kanvinde and Deierlein [4], the authors propose an analytical model to determine the lateral shear strength and initial elastic stiffness of wood and gypsum wall panels. A uniaxial spring model is defined, representing the nonlinear monotonic response that envelopes the cyclic response, and the cyclic nonlinear response including strength and stiffness degradation and pinching phenomenon. The parameters validation is performed by using experimental tests on full-scale wall panels. In Davies et al. [5] a description of the experimental results of full-scale tests performed on several cold-formed steel-framed gypsum partitions is reported. A tri-linear hysteretic macro-model is proposed, on the base of experimental results, to reproduce the in-plane mechanical behavior of the partitions: it allowed to include partitions in the model of an existing steel building in order to demonstrate the effect on the seismic behavior of the whole structure. A numerical macro-model for plasterboard partition is also proposed by Wood and Hutchinson [6]; a pinching material model, available in OpenSees [7] , is calibrated by a large number of experimental data obtained from about fifty tests performed on plasterboard partition walls [5]. The model is capable to reproduce the in-plane behavior of the partitions. Telue and Mahendran developed a finite element model of cold formed steel walls lined with plasterboard was and validated the proposed model by using experimental results. However, the behavior of the walls is investigated under compressive vertical loads. Buonopane et al. [8] also developed a numerical model for the in in-plane lateral behavior of cold-formed steel shear wall with wood panels, which focused on capturing the nonlinear behavior that occurs at the interface between cover panels and fasteners. Indeed, full scale lateral load tests on thirteen shear walls evidenced their failure due to the failure of the fasteners. The modelled walls were able to predict the cyclic response of the tested specimens, up to the peak point of lateral strength. All the mentioned research studies focused on steel stud shear walls, whose characteristics, e.g. stud typology, restraint at the base, failure mode, are different if compared to internal partition walls. A detailed numerical model of cold-formed steel-framed gypsum partition walls is proposed by Rahmanishamsi et al. [9]. The model provides nonlinear behavior of studs and tracks, and nonlinear behavior of connections, while the gypsum boards are simulated by linear shell elements. The contact between boards and concrete slabs as well as the contact between adjacent boards are also considered in the model. The model, validated by the comparison with experimental results on gypsum partitions walls, is able to predict the trend of the response and the observed damage mechanism. The developed numerical model does not incorporate failure due to buckling of the steel stud, which is the typical failure mode in tall plasterboard partition walls, as shown by Petrone et al. [10].

The aim of this research study is the definition of a proper finite element model able to evaluate the interstory drift ratio that induces the failure of a generic tall plasterboard partitions representative of European partition systems; the study is focused on the in in-plane seismic performance assessment. A simple modeling technique is proposed and validated. The validation is performed comparing the analytical numerical behavior of a specific specimen with the experimental results achieved in a quasi-static test campaign conducted at the Laboratory of the Department of Structures for Engineering and Architecture at the University of Naples Federico II. The quasi-static test is performed on a 5 m tall plasterboard internal partition, which is representative of the typical partitions used in industrial and commercial buildings in the European countries.

2 Methodology

2.1 Experimental test on the plasterboard partition specimen

The test system consists of a steel frame setup, the specimen, i.e. a plasterboard partition, a hydraulic actuator and a reaction wall (Fig. 1).

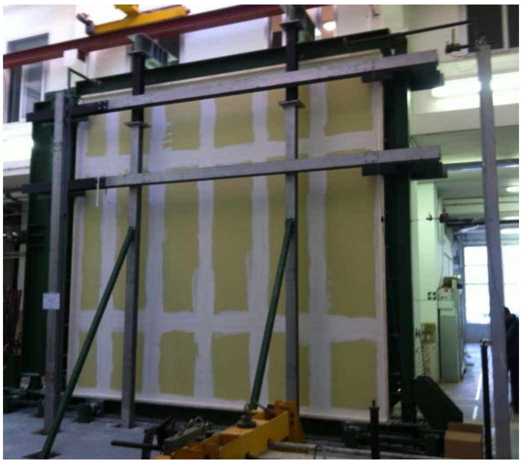


Fig. 1 - Global view of test setup

The specimen is 5.0 m high and 5.13 m wide and it is constituted, according to the mounting sequence, by:

- two horizontal U steel guides screwed, both at bottom and at top, in the wooden beams by 200 mm spaced screws;
- two vertical U steel guides screwed in the wooden beams by 500 mm spaced screws;
- five C-shaped steel studs , 900 mm spaced. They are placed vertically in the horizontal guides without any mechanical connection.
- two steel plates, with a rectangular cross-section 100 mm x 0.6 mm, connected to the studs at two different heights of the partition, i.e. 1200 mm and 3800 mm from the base.
- one 18 mm gypsum plasterboard layer for each side of the partition. The plasterboards are connected to the studs and to the steel plates by 250 mm spaced screws; they are assembled in three rows so as to define two horizontal joints at 1200 mm and 3800 mm from the base (see Fig. 2). The joints are sealed with paper and joint compound.

A cyclic test is performed in displacement control, according to the testing protocol provided by FEMA 461. Different measuring instruments are used in order to monitor the specimen behavior during the cyclic tests:

- two displacement laser sensors, placed at half the height of the column and at the top of the same column, respectively, in order to monitor top in-plane displacement and verify the rigid movement of the vertical column:
- two wire potentiometers, placed in parallel with respect to the laser sensors;
- two displacement transducers placed at the two edges of the top horizontal beam, which measure out-of-plane displacements, in order to validate the planarity of the motion;
- eleven strain gauges, divided between the steel studs and the plasterboards.

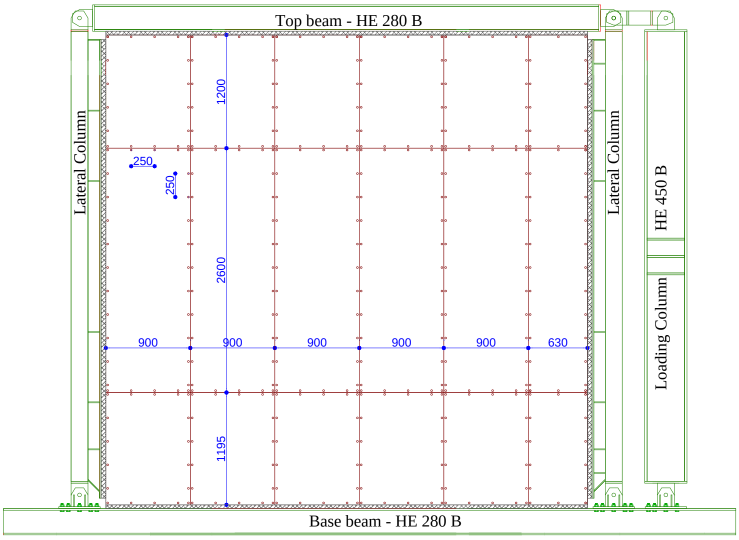


Fig. 2 - View of panel arrangement

The relationship among the top force and the top displacement, resulting from the quasi static test conducted on the partition, is shown in Fig. 3. It can be seen that the specimen exhibits a slightly non-symmetric behavior: in the positive quarter, i.e. the pushing direction, the force reaches its maximum value corresponding to a 20.2 mm displacement (0.40 % drift), while in the negative quarter the force reaches the maximum values to a 25.2 mm displacement (0.50 % drift). The specimen starts undergoing inelastic deformation and losing linearity at a 11.0 mm displacement (0.22 % drift): some sounds denote the screws bearing the connected plasterboards, the paper installed between the adjacent panels starts cracking and a minor drop of gypsum is observed. Corresponding to a 20.2 mm top displacement, the paper between the different panels completely cracks; at this displacement, the maximum force is recorded. Corresponding to a 68.4 mm displacement (1.37 % drift), a global out-of-plane curvature of the specimen is exhibited, i.e. the partition collapses due to the buckling of the studs. At this displacement value a significant strength degradation is visible on the hysteretic curve.

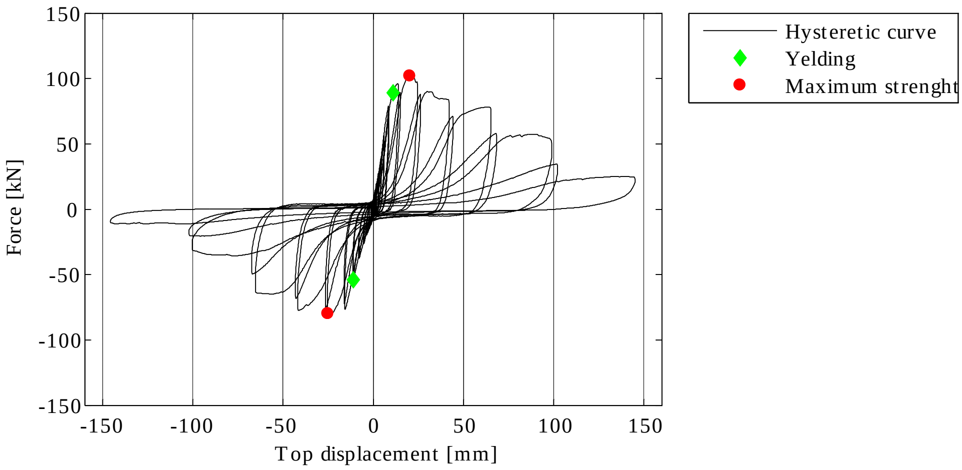


Fig. 3 - Hysteretic curve exhibited by the partition under the selected test protocol

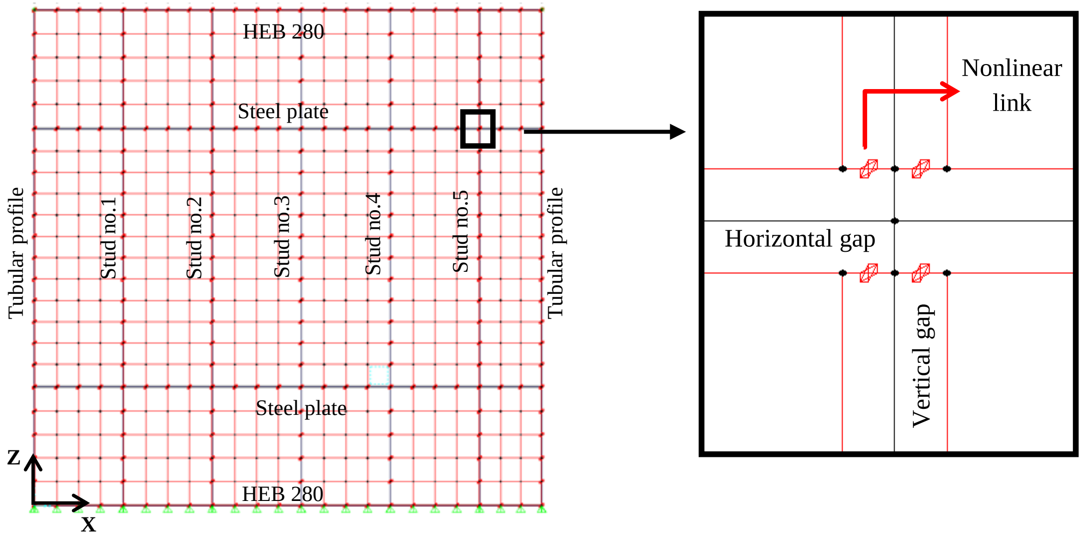
2.2 Definition of a finite element model for the partition

A model of the tested partition is defined in order to investigate the in-plane behavior through the numerical method. SAP2000 [11] program is adopted to perform finite element analyses.

The numerical model of the specimen is defined as a 2-D plane model, in order to reduce the computational effort. However, this assumption does not jeopardize the results thanks to the symmetry of the system with respect to the plane in which the partition is modeled. This model represents a starting stage for partitions modeling, since a tri-dimensional model could better catch the seismic in plane and out of plane behavior of such a partitions.

The whole system, shown in Fig. 4, is 5.00 m high and 5.13 m wide. It is characterized by 90 cm spaced steel studs and a single layer of plasterboards with a double thickness, i.e. 36 mm. The studs are modeled with frame elements with C-shaped cross-section; the boards, modeled as thin linear shell elements, are arranged in three distinguished horizontal rows, defining two horizontal joints. In order to reproduce the actual installation conditions of the boards, horizontal and vertical gaps are included between the plasterboards and the adjacent elements both in the horizontal and in the vertical directions (Fig. 4b). The plasterboards are properly meshed with 25 cm x 25 cm shell elements in order to introduce the panel-to-stud screw connections, according to their actual spacing. They are modeled as nonlinear springs, i.e. NLLINK objects in SAP2000, whose backbone curve is defined in Section 2.3. These links act along the two translational directions in the plane of the partition, namely x and z directions. A single link is representative of the behavior of two screws, which connect the two plasterboard layers either to the stud or the surrounding frame.

Two steel plates are placed at the two horizontal joints between the plasterboard panels; they are modeled by 100 mm x 1.2 mm rectangular cross section horizontal frames between two consecutive studs; internal hinges are placed at the end of each frame in order to reproduce the actual constraint given by a single screw. The whole system is surrounded by a 4-hinged steel frame, representative of the steel test setup. The base horizontal steel beam is externally restrained with several hinges, which fix the base of the specimen. The steel studs are only connected to the plasterboards through nonlinear links. They are not connected to the steel setup, both at the base and at the top.



(a) (b)

Fig. 4 - (a) Finite element model of the testes partition in SAP2000 and (b) particular of stud-to-panel connection evidencing the horizontal and the vertical gaps

The steel material adopted for the studs, the horizontal plates and test frame is characterized by the mechanical properties shown in Table 1a, whereas the gypsum wallboards mechanical properties are listed in Table 1b. Both the gypsum and the steel materials are modelled with a linear elastic material. The presence of the paper and the compound between adjacent plasterboards is neglected in the model.

Table 1 - Steel and gypsum wallboards mechanical properties, based on experimental tests

Steel properties	
Tensile strength, f_s $[MPa]$	301.0
Young modulus, E $[MPa]$	210000

Gypsum properties	
Compression strength, f_g $[MPa]$	8.18
Tensile strength, $f_{g,t}$ $[MPa]$	1.42
Young's modulus, E $[MPa]$	3601

It should be noted that, despite the large number of elements, the model of the partition is quite simple since the nonlinearity is lumped in the panel-to-stud screwed connections; this is widely supported by the experimental evidence that showed severely damaged screwed connections before the partition failure.

The occurrence of the failure mechanism, due to the buckling of the partition, is a-posteriori checked; it is based on the internal forces acting on the stud for a given level of displacement demand.

2.3 Calibration of the screwed connection backbone curve

In order to define the mechanical behavior of the connections, some experimental tests are performed on the screwed connection adopted in the considered partition. The tested connection refers to the specific configuration of the specimen; the test setup is shown in Fig. 5: two back-to-back studs are connected to two 18 mm thick single layer plasterboards through two screws for each side. The self-drilling screws are characterized by a 3.5 mm diameter and a 35 mm length. The screws strength is evaluated in terms of Rockwell hardness, whose value is around 44.

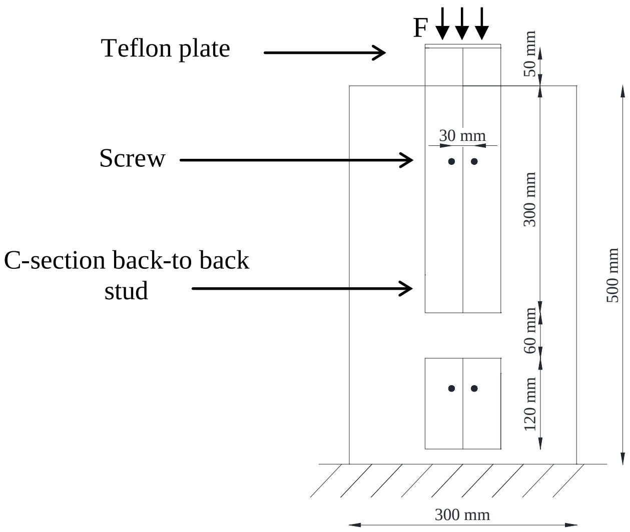


Fig. 5 - Schematic view of the tested specimen dimensions (in mm)

On the base of the experimental results a tri-linear backbone curve is assigned to the nonlinear link in the numerical model of the partition. The tri-linear envelope is shown in Fig. 6, where:

- F_{max} is the maximum force reached during the experimental test and d_{max} is the corresponding displacement;
- F_u and d_u are the ultimate force and displacement reached at the specimen failure, respectively;
- F_y value is obtained by imposing two conditions: (a) the initial stiffness k, i.e. the slope of the first branch of the tri-linear curve, is evaluated according to Schafer [12] as:

$$k = \frac{0.4F_{max}}{d_{0.4}} \quad (1)$$

in which $d_{0.4}$ is the displacement value that corresponds to 0.4 F_{max} force; (b) the dissipated energy up to F_{max} is the same both in the experimental and in the numerical force-displacement curve.

The yielding displacement d_y can be clearly evaluated as follows:

$$d_y = \frac{F_y}{k} \tag{2}$$

The third branch of the envelope is simply obtained assuming a linear envelope from the capping point, i.e. the point characterized by the maximum force F_{max}, to the ultimate point.

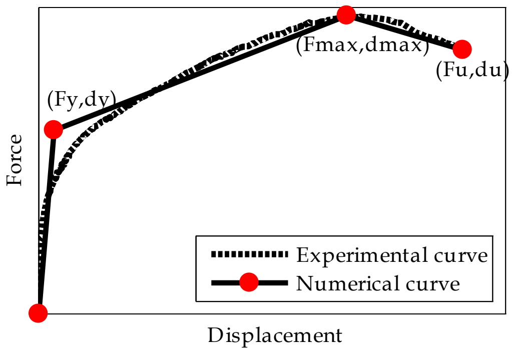


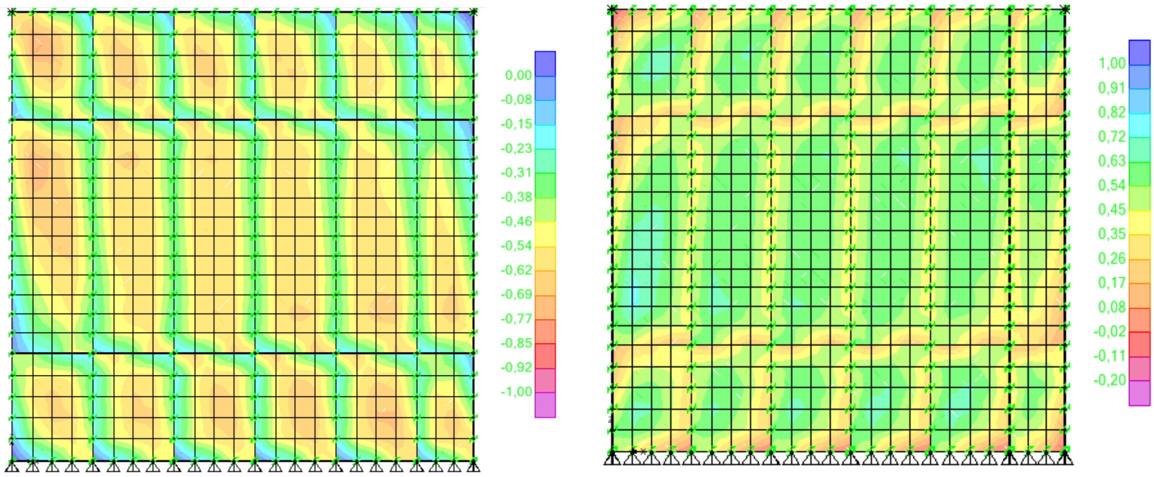
Fig. 6 - Experimental and tri-linear backbone curves of the screws connection

The forces of the modeled connection are obtained scaling down the forces by a factor of two, since the tri-linear curve in Fig. 6 is representative of the behavior of four screws whereas the nonlinear spring included in the model corresponds to two screws

3 Analysis results and numerical – experimental comparison

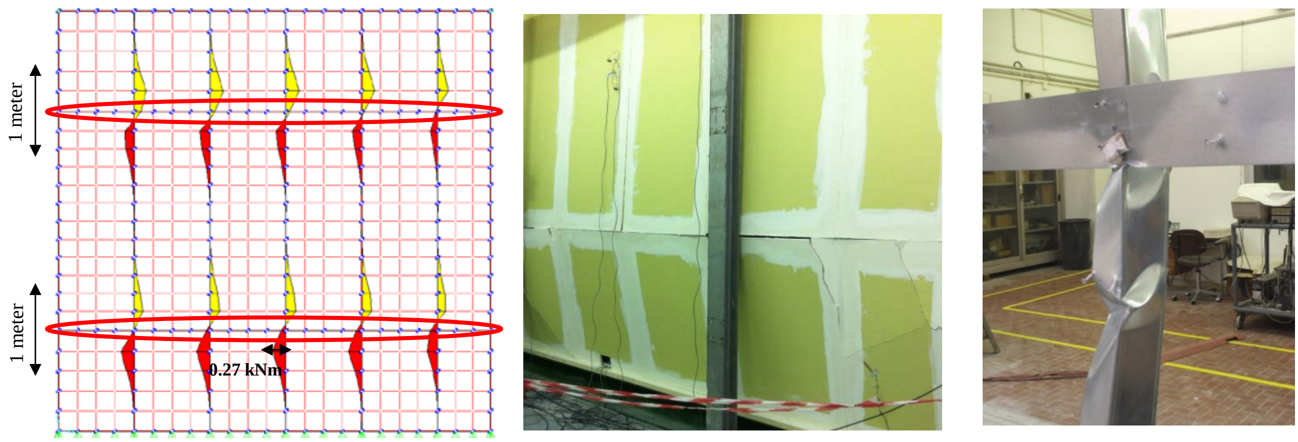
The numerical model of the specimen described in the previous Section is subjected to a large-displacement nonlinear static analysis in displacement control through the SAP2000 program [11]. A monotonic top displacement is applied in 20 consecutive steps reaching a 111 mm maximum displacement, i.e. 2.2% interstory drift. When the partition is pushed the force is transferred to the base through the plasterboard panels. The stress trends highlight that the compression stresses in the plasterboards (Fig. 7a) are concentrated in a diagonal strut, i.e. from the top left to the bottom right of each panel. The maximum stress values are close to 1.0 MPa at 111 mm top displacement. In Fig. 7b a tensile diagonal strut is visible in each plasterboard panel from the bottom left to the top right, the maximum tension stress value is about 0.9 MPa. The low level of stresses justifies the modeling of the gypsum material with a linear elastic behavior (Section 2.2). In turn the panels transfer the load to the studs through the screws; the studs are therefore subjected to both bending moment and compression axial force.

The bending moment diagram on studs reveals a concentration of stress values crossing the two horizontal joints (red circle in Fig. 8a). In these zones the high stress values can justify the concentration of damage, which is experimentally pointed out exactly over and under the two horizontal joints (Fig. 8b and c). The results of the performed analysis are remarkable since the behavior of the numerical model seems to reproduce quite accurately the experimental evidence.



(a) (b)

Fig. 7 - (a) Compression and (b) tension stresses (in MPa) diagram on plasterboards at last step of the analysis



(a) (b) (c)

Fig. 8 - (a) Bending moment diagram at a 68 mm top displacement and crossing horizontal joints (in red circles) (b) out of plane failure on the partition and (c) observed damage on stud across the horizontal joint

On the base of the studs internal stresses the occurrence of the partition buckling is assessed according to the Direct Strength Method (DSM) proposed by Schafer [12]. This method is usually used to design cold-formed steel stud walls braced by sheathing connected to the stud; the partition system investigated in this paper can be included in such a structural system typology. According to this method, the occurrence of the stud buckling is influenced: (a) by the mechanical and geometrical characteristics of the studs, (b) by both the sheathing system and (c) the board-to-stud connections, that provide a bracing restraint to the stud. The influence of the panels and the panel-to-stud connections are modelled through elastic springs that restraint the steel stud. Three different springs, i.e. two translational ones and a rotational one, are introduced at the fastener location. The stiffness values (k_x, k_y and k_ϕ) can be evaluated through closed-form formulas provided by Vieira and Schafer [12; 13], even if an experimental evaluation is preferred.

In the presented work, experimental values for k_x and k_y are considered. The evaluation of the k_x in-plane stiffness is explained in Section 2.3. k_y value is defined according to out-of-plane tests performed on the partition, here omitted for the sake of brevity. The application of the DSM method consists in determining the axial forces and bending moments that produce the instability of the stud. The nominal axial (P_n) and flexural (M_n) strength of the stud can be assessed by using the expressions provided by AISI-S100-

07 [14], based on the modelled stud cross section through the CUFSM software [15]. In particular, three resisting axial force and bending moment values are defined, i.e. one for each instability mode of failure.

The DSM method provides that the capacity of the stud is conceived as the minimum between local, distortional and global buckling load capacity. It is therefore implicit that the steel studs are designed in order not to exhibit any of the three considered buckling failure modes. However, since the aim of this work is to define a suitable numerical model of steel stud sheathed partitions in order to verify the seismic behavior of such components, the DSM is used as a-posteriori checking method. A domain for each buckling mode is defined to check its occurrence for a given level of displacement demand. The internal forces acting on the studs, in terms of axial force and bending moment, are compared to the limit curves.

In Fig. 9 the buckling occurrence is verified for the different studs of the tested partition at different interstory drift ratio levels. The local and the distortional domain are overlapped for the tested partition; a single limit curve is then plotted representing both local and distortional buckling (indicated for brevity as “Local instability”), as shown in Fig. 9. In Fig. 9 the occurrence of the different buckling modes is checked in stud no. 3, i.e. the stud that first exhibits buckling. As shown in Fig. 9a, at a 0.52% interstory drift the partition local instability occurs. When the partition reaches an interstory drift equal to 1.37% (Fig. 9b), the stud no. 3 globally buckles. For the same drift level also the stud no. 2 and no. 4 globally buckle, whereas in stud no.1 and stud no.5 only local buckling occurs. Nevertheless, the whole partition can be considered to be failed at this step.

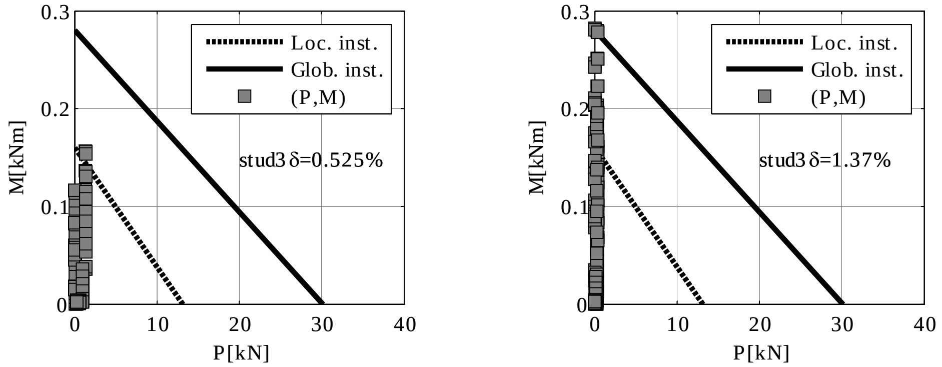


Fig. 9 - (a) Local and (b) global instability occurrence stud no. 3 according to DSM method

3.1 Experimental-numerical damage comparison

A comparison between the experimental damage of the tested partition and the prediction of the numerical model is performed in this Section. The comparison between the numerical pushover curve and the experimental force-displacement backbone curve is shown in Fig. 10. In the numerical model, nonlinear behavior of the screws occurs for a top displacement equal to 10 mm (green marker in Fig. 10); the failure of the partition (red marker in Fig. 10) occurs at 1.37% interstory drift, considering the occurrence of global instability in a single stud as the partition failure. Beyond this point the curve is plotted as a dotted line since it is not representative of the partition behavior.

The experimental curve exhibits an initial stiffness similar to the stiffness recorded in the numerical model. The screw bearing mechanism occurs at a 11 mm displacement, which is similar to the displacement required to yield some screws in the numerical model. Beyond a 10 mm top displacement, the numerical curve does not match the experimental one: both the strength and the stiffness are underestimated. This phenomenon is due to the non-inclusion of the paper and the compound in the model (see Section 2.2). However, this approximation is limited up to the failure of the paper and the compound that occurs at 20 mm top displacement, i.e. well before the failure of the specimen. Finally, the experimental evidence demonstrates that at 68 mm displacement (1.37% drift) the specimen starts showing a global out-of-plane curvature. It can be deduced that the model well catches the interstory displacement required to induce global buckling failure mode of the specimen. At that point the force acting on the partition is also well predicted by the model.

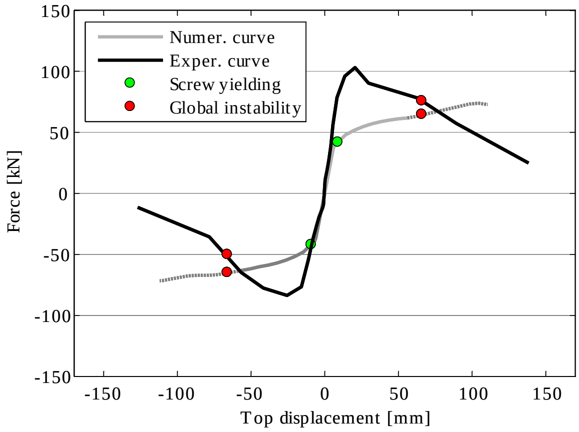


Fig. 10 - Numerical pushover curve and experimental force-displacement curve

4 Conclusions

The research study deals with the definition of a finite element model capable to capture the interstory drift ratio which causes the failure of a 5 m high plasterboard partition representative of European partition systems. The assessment is performed comparing the numerical behavior of a specific specimen with the experimental evidence of a quasi-static test campaign conducted at the Laboratory of the Department of Structures for Engineering and Architecture at the University of Naples Federico II. A simple model of the tested partition is defined. Both the steel stud and the plasterboards are modelled with linear elastic elements. The nonlinearity is lumped in the panel-to-stud screwed connections. A tri-linear force-displacement backbone curve is assigned to the screwed connections matching the experimental results of monotonic tests on such a connection.

The numerical results of a monotonic test on the defined model evidence tension and compression struts in the plasterboards due to the applied top displacement. The low stress values both in tension and in compression justify the adoption of a linear elastic material for the boards. The bending moment diagram on studs reveals large demand crossing the two horizontal joints between the plasterboards. Such an evidence can justify the damage, experimentally pointed out in the steel stud above and below horizontal joints among plasterboards. The Direct Strength Method (DSM) is applied to assess the failure of the partition due to the occurrence of different buckling failure modes, i.e. local, distortional and global failure modes, in the studs. The method evidences that the both local and distortional instability failure modes occur at a 0.52% interstory drift, whereas the global buckling is exhibited at an interstory drift equal to 1.37%. The experimental test evidences that for a 1.37% drift the specimen starts showing a global out-of-plane curvature: it can be therefore deduced that the model well catches the global buckling failure mode of the specimen. The experimental curve is well reproduced by the numerical one in terms of initial stiffness, whereas the model underestimates the strength of the partition.

Acknowledgements

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