CYCLIC BEHAVIOUR OF BRACING TYPE PURE ALUMINIUM SHEAR PANELS (BTPASPs): EXPERIMENTAL AND NUMERICAL ANALYSIS

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ABSTRACT :

Based on a wide experimental campaign carried out on four specimens subjected to cyclic loads, a non-linear numerical model for interpreting the behavior of bracing type pure aluminum shear panels has been set up. After a brief overview on the experimental tests, the calibration procedure of this models is presented in the current paper. It has been implemented on the basis of the hysteretic response revealed by tests, as well as on some characteristic measures which put in evidence the behavior of the proposed devices in terms of both stiffness and energy dissipation capacity. The obtained numerical results are in good agreement with the experimental ones, also because proposed model is able to capture the most important experimental evidences at any shear strain demands, such as the triggering of buckling phenomena and the observed deformed shapes. This allows the use of the numerical model as a convenient tool to check the stress state, the deformed shape and the main resistant mechanisms of the analysed shear panel typology.

KEYWORDS: Shear panels, FEM models, Experimental tests, Non linear numerical analyses, Dissipative devices

1. OVERVIEW

It has been already proved that both stiffened and unstiffened metal shear plates are very appealing to be used as specific devices for enhancing the seismic performance of new and existing buildings. Generally speaking, such devices should be rationally arranged into some of the meshes of the primary structure, they being either directly (full bay or partial bay type) or indirectly (bracing type or pillar type) connected to its columns and girders (see Figure 1).

Figure 1: Shear panel typologies installed into a framed steel structure: (a) full bay type and (b) bracing type.

In this way, the whole structural system formed by frames and metal panels (Metal Plate Shear Wall, MPSW) behaves as a vertical cantilever plate girder able to resist to horizontal story shear and overturning moment due to lateral actions. The plates act as the web of the resulting girder, the columns form its flanges and the horizontal floor beams act as transverse stiffeners (Astaneh-Asl 2001).
In the last thirty years, many studies on the performance of MPSWs have been undertaken based on the assumption that shear plates may offer a suitable post-buckling strength. Such studies highlighted the advantages that shear panels can provide with respect to the traditional lateral load resisting systems. In particular, it has been clearly pointed out that the assessment of security of buildings equipped with shear plates must be carried out considering a design ultimate limit state not corresponding to the out-of-plane buckling of the shear panels, as a tension field mechanism can advantageously arise (see Figure 2).

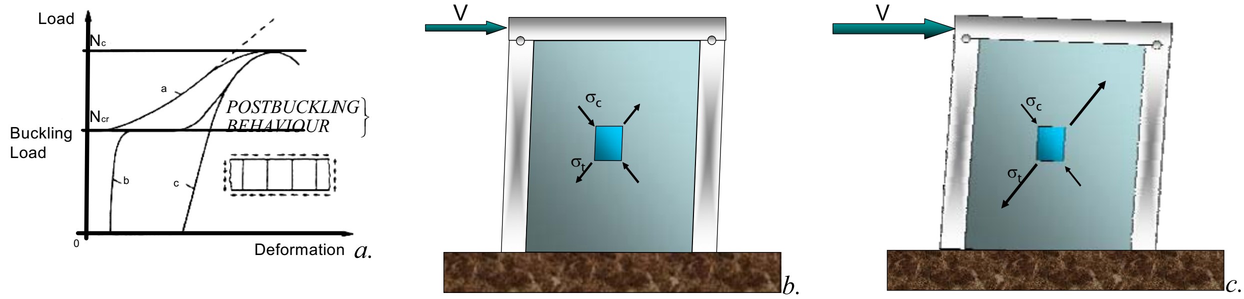


Figure 2: (a) Pre- and Post-buckling behaviour of shear platesb) Pre-buckling and c) Post-buckling stress state in shear panels

Nevertheless, only since the 80's many experimental and numerical studies on the post-buckling behavior of MPSWs have been carried out in order to provide useful rules and design criteria to be adopted in national and international Standards, i.e. the AISC Specifications(2005). They were based on the assumption that a shear plate can provide an effective initial stiffness, thus limiting inter-story drifts value under low seismic actions, it being also able to develop a tension field resisting mechanism, once that buckling phenomena are triggered off, behaving in a very ductile way with a significant amount of energy dissipated under cyclic loads. In particular, the stiffening and the dissipative function of structures equipped with MPSWs, under both static and dynamic actions, have been deeply investigated, also comparing their performances with the ones offered by different seismic devices. Moreover, accurate finite element models have been implemented in order to identify the principal behavioral phenomena acting in a shear panel, and various metal materials and geometrical configuration, i.e. characterized by different disposition of applied stiffeners or by holes wisely made on the base plate, have been considered as well.
Two main different panels typologies can be distinguished. The first, based on using thin unstiffened plates, is adopted to increase the stiffness and the strength of the main structural facility, with a dissipative function that is affected by pinching phenomena on the hysteretic cycle. The latter, based on using dissipative panels, is characterized by the adoption of proper stiffeners, able to avoid premature plate buckling, therefore conferring an attractive hysteretic behavior under cyclic loading. In this case, the dissipative function of the panels can be activated for very low interstory drift demands when either low yield materials or properly weakened geometrical configurations are adopted. In particular, the use of low yield metals has been particularly appreciated in the last decade, also because these materials are commonly characterized by a large hardening ratio and a considerable isotropic hardening, allowing a beneficial effect on the ultimate strength of the system. These features favor a very suitable stable cyclic behavior for large shear strain demands.
In this framing, a new panel typology has been recently proposed by the authors (De Matteis et al, 2003). It is made by using the AW 1050A alloy, which is almost pure aluminum, representing a novelty in the structural engineering field. The shear panel is preliminarily subjected to a heat treatment in order to eliminate both the initial hardening and the residual stresses produced by impurities and manufacture processes. As a consequence, a yielding stress of around 20 MPa and a ductility of about 50% are obtained. Such material features are adequate for allowing a very good dissipative behavior of the shear panels, with fat hysteretic cycles also for large shear deformation demands.
Initially, full-bay shear panels have been proposed and studied from both the experimental (De Matteis et al,

2008[a]) and numerical (De Matteis et al, 2007[a]) points of view. Based on the experimental results, it was shown that the structural response of pure aluminum full-bay shear panels, in terms of energy dissipation and damping capability, is remarkable for medium-large lateral displacements. On the other hand, some slipping phenomena were observed for small displacement levels, thus limiting the capability of the system. In order to increase the local dissipative demand of shear panels also for reduced inter-storey drift levels of the primary framed structure, a bracing type configuration for shear panel has been proposed. It is characterized by reduced dimensions respect to surrounding frame field, so to obtain a favorable ratio between the inter-storey drift of the primary framed structure and the shear deformation of the panels.
In the current paper, the experimental activity carried out on these type of devices as well as the setting up of suitable numerical models on the basis of the obtained experimental outcomes are described.

2. EXPERIMENTAL ACTIVITY

2.1. Testing procedure and selection of specimens

Four 5.00 mm thick bracing type pure aluminum shear panels have been tested under a quasi-static diagonal cyclic forces (see Figure 3-a), according to the displacement history represented in figure 3-b. They have been inserted into a pin jointed steel framework and linked to its channel elements along their edge by means of tightened steel bolts.

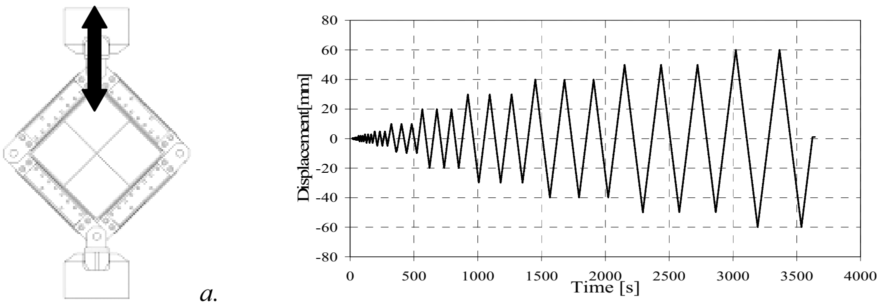


Figure 3. Loading scheme (a) and applied displacement history (b).

The tested specimens (see Figure 4) are characterized by welded rectangular-shaped ribs placed on both the two faces of the base plate, with slenderness ratio values a_w/t_w equal to 100 (BTPASP "type 1"), 25 (BTPASP "type 2"), 33 (BTPASP "type 3") and 25 (BTPASP "type 4"), respectively. The applied stiffeners have a depth of 60 mm and are made of the same material and thickness of the base aluminum plate.
The spacing and depth of ribs have been selected to fulfill the requirements provided by the European Standard, in order to ensure shear buckling of the plate after shear yielding. In fact, Eurocode 9 (2007) provides the following limit value of a slenderness parameter λ_w upon which shear buckling resistance results higher than the yielding resistance:

$$\overline{\lambda}_w = \frac{0.83}{\eta} = 0.69 \tag{1}$$

where the η factor, for which Eurocode 9 fixes a superior limit value of 1.2, is given by the following expression:

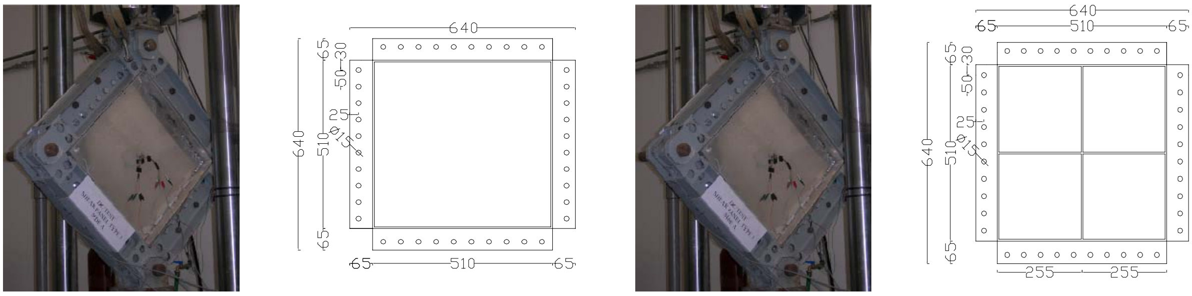
$$\eta = 0.7 + 0.35 \cdot \frac{f_{aw}}{f_{ow}} \tag{2}$$

In the case being, the η factor has been conventionally assumed equal to 1.2, even though the hardening ratio α (f_{aw}/f_{ow})= 3 would lead to a corresponding η value equal to 1.75. Hence, it is possible to observe that the studied shear panels guarantee a slenderness value lower than the limit one. In fact, provided that the expression of slenderness is given by eq.(3) for panel "type 1", which can be related to a "plate girders with web stiffeners at support", while by eq. (4) for the others panels, which behave as a "plate girders with intermediate web stiffeners", the value of slenderness of the selected panels result: λ_{w1}=0.66 ("type 1"), λ_{w2}=0.51 ("type 2"),

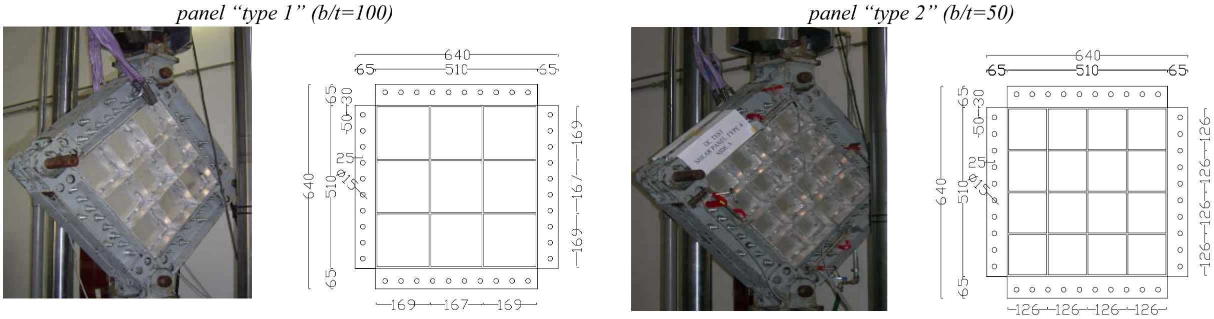
λ_{w3}=0.22 (“type 3”) e λ_{w4}=0.17 (“type 4”).

$$\lambda_w = 0.35 * \frac{b_w}{t_w} * \sqrt{\frac{f_0}{E}} \tag{3}$$

$$\lambda_w = \frac{0.81}{\sqrt{k_\tau}} * \frac{b_w}{t_w} * \sqrt{\frac{f_0}{E}} \tag{4}$$



panel “type 1” (b/t=100) panel “type 2” (b/t=50)



panel “type 3” (b/t=33) panel “ type 4” (b/t=25)

Figure 4. The tested specimens

2.2. The obtained results

The main experimental results are provided in Fig. 5, where the obtained hysteretic cycles for each tested specimens are depicted. In the whole a very a good hysteretic performance of tested shear panels can be noticed, which indeed resulted influenced by different collapse modes depending on the applied stiffener configuration. In fact, it has to be observed that the internal rib system, in addition to influence the local slenderness of the plate, acted as an internal framework, providing an additional strength contribution to the shear panel. Therefore, shear panel configurations characterised by a lower slenderness ratio were subjected to larger forces. In this case, the connecting system represented the weakest component of the device, provoking an anticipated failure mode.
Nevertheless, it is evident that panels “type 3” and “type 4” presented the higher energy dissipation capacity, as pinching effects did not influence their hysteretic behaviour. But the higher strength offered by panel “type 4” (around 10-20 kN), when a diagonal displacement of ±20.0 mm was applied, entails an anticipate failures of the perimeter connecting system and, thus, a sharp lost of strength capacity for a diagonal displacement of ±40.0 mm. As a consequence, it is possible to state that the panel “type 3” configuration represents the optimum solution among tested panels.

3. CALIBRATION OF NUMERICAL MODELS

3.1. General

For each tested specimens, a detailed numerical FEM numerical model has been set up by means of the ABAQUS 6.5 non linear analysis software (ABAQUS Inc., 2004). The reliability of such a models has been proved by the above experimental results. The obtained numerical models may be used as a convenient tool to check the stress state, the deformed shape and the main resistant mechanisms of the analysed shear panels prototypes. Moreover, a parametrical investigation may be carried out.

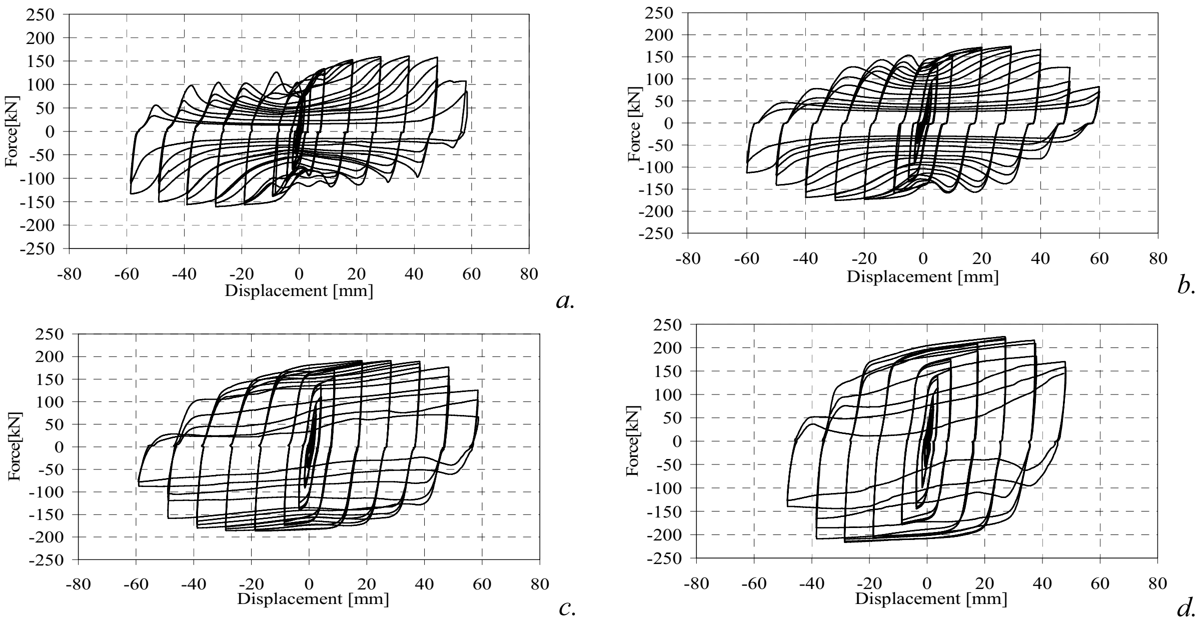


Figure 5. Hysteretic cycles: “type 1” (a), “type 2” (b), “type 3” (c) and “type 4” (d).

3.2. The adopted FEM model

The proposed FEM model reproduces the actual geometry of the studied system. The four arms of the perimeter steel frame have been modeled by using a first order two-node three-dimensional B31 BEAM element, while the four-nodes bilinear (with reduced integration and a large strain formulation) S4R SHELL finite element has been used to model the aluminum sheeting and the applied stiffeners. Five integration points have been taken into account through the thickness of the shell elements, as this number is generally sufficient to interpret also highly nonlinear problems. The beam elements have been constrained to each other by means of three-dimensional two-nodes hinge connector elements CONN3D2, while the whole external frame and the panel zone included into the steel arms have been restrained towards the out-of plane deformations by means of effective boundary conditions. The bottom point of the surrounding frame has been fixed in the horizontal direction while it has been left free to move in the vertical direction for a range of ± 0.60 mm in order to simulate the initial slip phenomena registered during the experimental tests. This type of behavior has been realized restraining the aforementioned point to the ground by means of axial connector elements CONN3D2, characterized by a rigid-plastic behavior with a yielding force of 3.00 kN, and imposing a STOP OPTION when a translational displacement of ± 0.60 mm in the diagonal direction is achieved. The frame-to-panel connection, which is realized by means of tightened steel bolts located with a pitch of 50 mm, has been introduced in the model by considering that no slip between the different parts occurs. This has been modeled by using the TIE constraint of the ABAQUS program library, which has been applied between the panel edges and the corresponding frame members. Although this approximation might influence the system response for high displacement values, since the experimental details on the actual behavior of the applied connections are not still available as any particular connection modeling has been introduced so far. Similarly, the same TIE command has been employed to model the interaction between the stiffeners and the aluminum plate. The initial geometrical imperfections of the system, mainly due to the stiffeners welding, have been taken in account considering as initial deformed configuration a combination between the buckling mode shapes read from the first and third eigenvectors of a previously implemented buckling analysis and assigning to both of them a maximum out-of-plane displacement equal to 1/100 of the free length involved in buckling phenomena. Such two modes have been combined to each other since they are characterized by the same sinusoidal shape along the two diagonal directions.

As far as the material modeling is concerned, a detailed description is provided in De Matteis et al. (2007[b]), where both the monotonic and cyclic behavior of the base material has been calibrated on the basis of experimental tests.

In figure 6, the four numerical models are shown. It is possible to observe that a mesh characterized by 25x25 mm base elements has been adopted. In fact, by choosing 12.5x12.5 mm mesh elements, only a slightly higher accuracy could be achieved, whereas an increment of the analysis running time of 5 times was noticed.

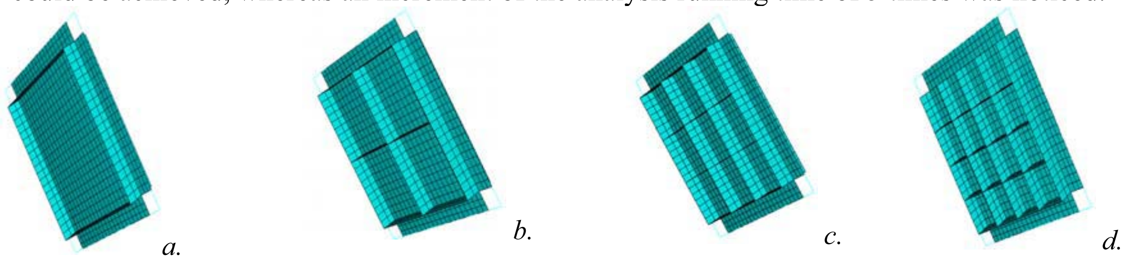


Figure 6. The proposed FEM numerical models: “type 1” (a), “type 2” (b), “type 3” (c) and “type 4” (d).

3.3. Comparison between experimental and numerical results

The simulation of tests described in section 2 allowed to compare numerical and experimental results and to demonstrate the reliability of the proposed modeling. According to the experimental lay-out, an external diagonal displacement has been statically applied to the top beam of the external pin jointed frame of the FEM models. In Figure 7, the comparison is provided in terms of hysteretic cycles.

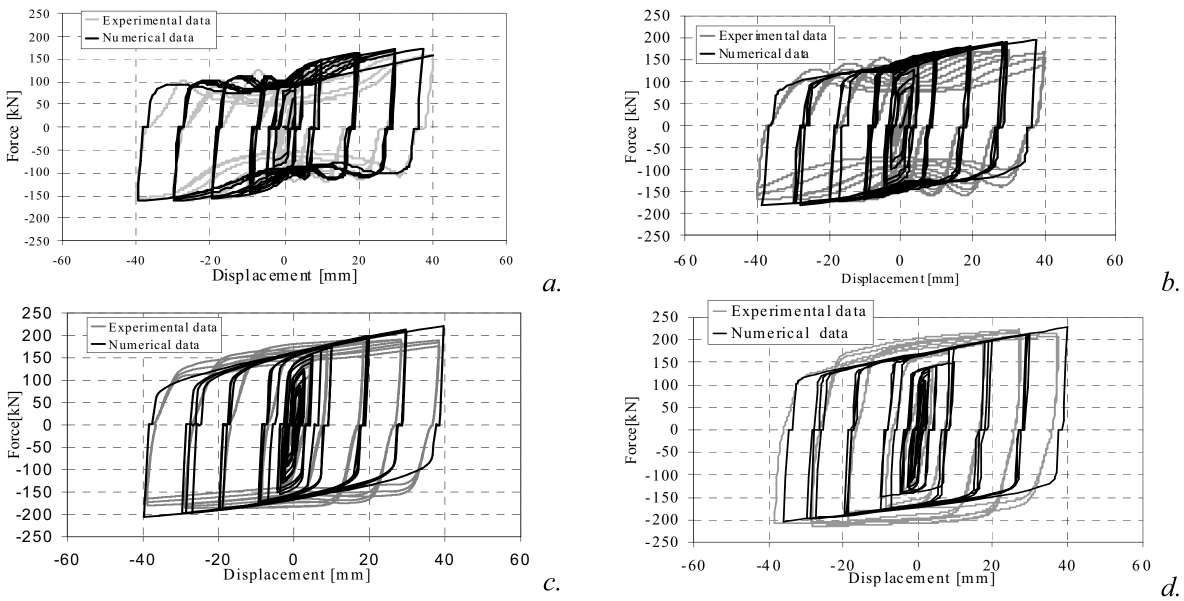


Figure 7. Comparison between numerical and experimental results in terms of hysteretic cycles: “type 1” (a), “type 2” (b), “type 3” (c) and “type 4” (d).

It is to be noticed that only diagonal displacement demands ranging from -40 mm to +40 mm have been taken into account. In fact, when larger displacements are attained, the system response is influenced by both the failure of the perimeter connecting system and the fracture of the base plate, which are not contemplated in the numerical model. These aspects are evident from a careful inspection of the experimental hysteretic loops from which it is possible to observe a contraction of the ±40.0 mm second and third cycles.

In Figure 8, the comparison in terms of secant stiffness is provided, while in Figure 9, the equivalent viscous damping measured at each shear strain demands is considered. The obtained results prove the reliability of the numerical model, which is able to capture all the main behavioral aspects of the system, namely, the strength, the stiffness and dissipative features, including the pinching effects due to the buckling phenomena. It is also evident that the initial slipping phenomena, which are unavoidable for the practical tolerance in every steel structure and whose entity has been interpreted during the calibration procedure of the numerical model, lead to a degradation of the dissipative response of the system for small displacement values. On the other hand, for medium-high shear deformation levels the dissipative capability reaches its maximum level, with peak values of the equivalent viscous damping factor of 40-45%.

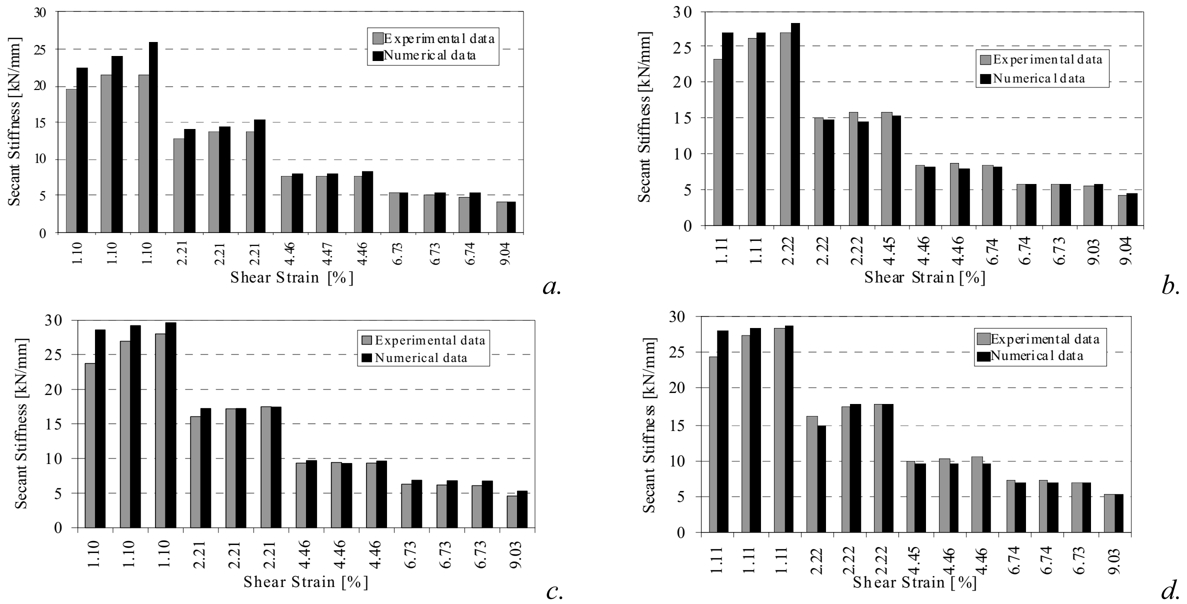


Figure 8. Comparison between numerical and experimental results in terms of secant stiffness: “type 1” (a), “type 2” (b), “type 3” (c) and “type 4” (d).

Finally, in Figure 10 a comparison is also provided in terms of ultimate deformed shapes. For the sake of brevity only two of the four studied panels are shown.
Also in this case, a good agreement between numerical and experimental results is recognizable. From the same picture, it is also possible to evidence the ultimate stress values.

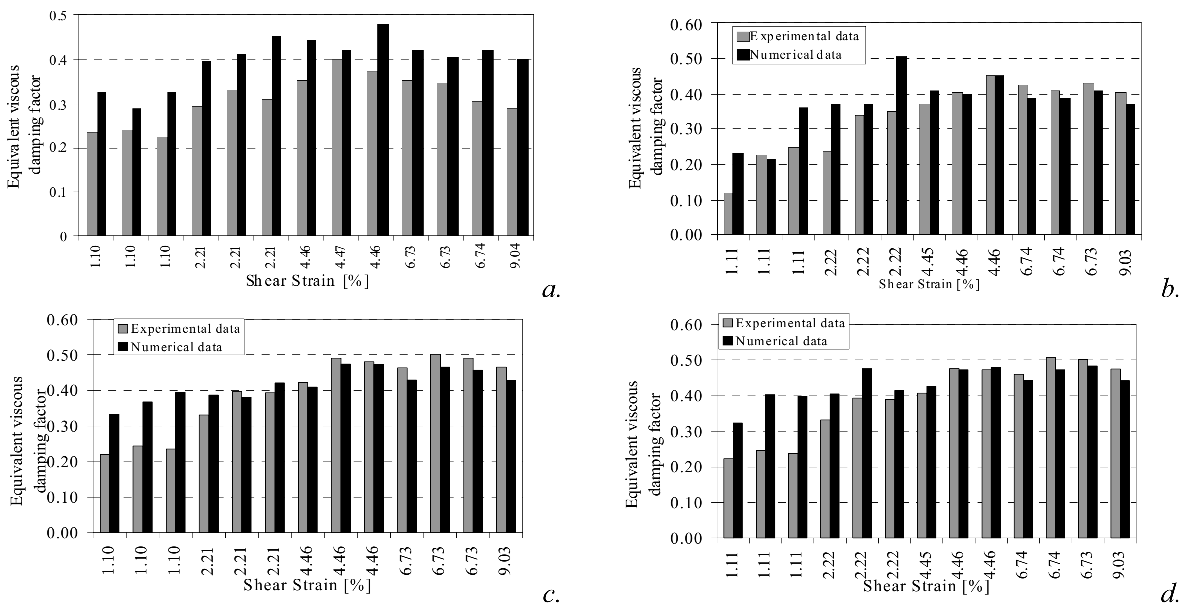


Figure 9. Comparison between numerical and experimental results in terms of equivalent viscous damping factor: “type 1” (a), “type 2” (b), “type 3” (c) and “type 4” (d).

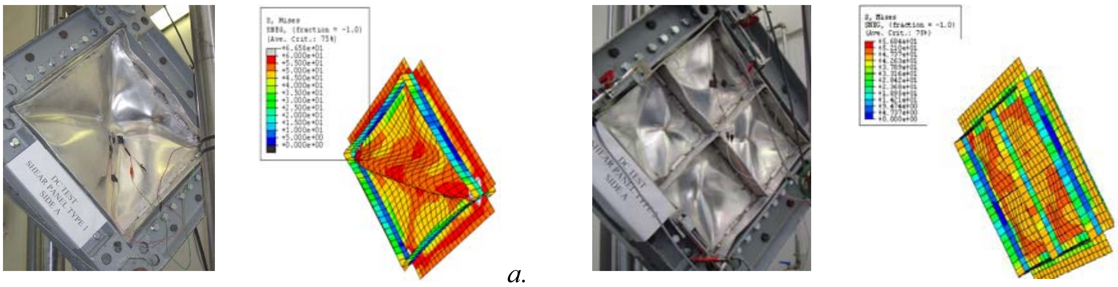


Figure 10. Comparison between numerical and experimental results in terms of ultimate deformed shapes: "type 1" (a), "type 2" (b).

4. CONCLUSIONS

In the current paper a suitable FEM numerical models of four 5.00 mm thick bracing type pure aluminum shear panels characterized by different slenderness values has been set up. A calibration procedure has been carried out on the basis of four experimental tests, whose results have been considered as a term of comparison for the numerical analyses. It has been shown that the proposed numerical model is able to well reproduce the behavior evidenced by shear panels during the tests, in terms of secant shear stiffness, attained strength and dissipative features, the latter represented by means of equivalent viscous damping ratio.
Therefore, the obtained numerical models may be used as a convenient tool for parametrical investigations aimed at selecting optimal shear panels configurations.

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