Experimental Investigation on the Seismic Performance of LWS Drywall Architectural Non-Structural Elements

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Experimental Investigation on the Seismic Performance of LWS Drywall Architectural Non-Structural Elements

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Abstract. The paper presents the research activity carried out for evaluating the seismic performance of Lightweight Steel (LWS) drywall architectural non-structural elements made of Cold-Formed Steel (CFS) profiles sheathed with gypsum-based or cement-based boards, i.e. indoor partition walls, outdoor façade walls and suspended continuous ceilings. The experimental investigation was organized in three phases: ancillary tests, element-level tests and assembly-level tests. Ancillary tests were carried out for assessing the local behaviour of partitions, façades and ceilings thought tests on steel material, screws, sheathing boards and board-to-frame fixings. Element-level tests on partitions, specifically 22 out-of-plane quasi-static monotonic tests, 11 out-of-plane dynamic identification tests and 12 in-plane quasi-static reversed cyclic tests, were performed. Finally, the dynamic behaviour was investigated through 5 shake table tests on different assemblages of partitions, façades and ceilings. The influence on the seismic response of stud spacing and basic and enhanced anti-seismic solutions, corresponding to the use of fixed or sliding connections at the wall and ceiling perimeter, was investigated. Test results were analysed in terms of strength, stiffness, damage phenomena, dynamic properties, dynamic amplification and seismic fragilities. The study categorized the element behaviour in three classes, i.e. elements with low, intermediate and high fragilities, by demonstrating that the tested architectural non-structural elements are able to exhibit a good seismic behaviour with respect to the damage limit states according to the inter-storey drift (IDR) limits given by Eurocode 8 Part 1.

Keywords: Indoor Partition Walls, In-Plane and Out-of-Plane Behaviour, Lightweight Steel, Outdoor Façade Walls, Suspended Ceilings.

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1. INTRODUCTION

The lack of understanding on the seismic behaviour of architectural non-structural components is becoming one of the most important issue of the structural design within the framework of performance based-design. The main aim of the current researches and codifications [ASCE, 2010; ASCE, 2013; CEN, 2005] is the introduction of specific design requirements in terms of strength and deformation for architectural non-structural components in order to ensure collapse prevention and to reduce the seismic vulnerabilities by imposing limits for the damage limitation control. The current work intends to deepen the seismic behaviour of architectural non-structural lightweight steel (LWS) drywall components, i.e. indoor partition walls, outdoor façades and suspended continuous ceilings, named in the following simply partitions, façades and ceilings, respectively. To this end, an extended research activity was performed at the University of Naples "Federico II", in the context of more general studies dealing with LWS constructions and non-structural components [Terracciano *et al.*, 2018; Fiorino *et al.* 2016; Fiorino *et al.*, 2017a; Fiorino *et al.*, 2017b; Macillo *et al.*, 2017; Fiorino *et al.*, 2017c; Fiorino *et al.,* 2018b] and other researches [Tartaglia *et al.*, 2018a; Tartaglia *et al.*, 2018b; Nastri *et al.*, 2017].

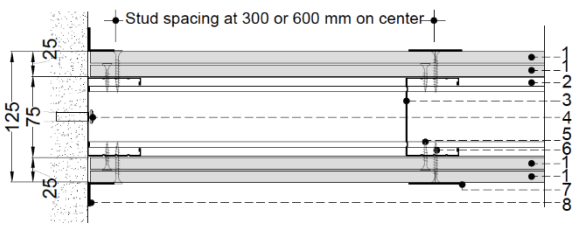
2. EXPERIMENTAL PROGRAM

2.1 TESTED NON-STRUCTURAL COMPONENTS

The focus of the experimental research was the assessment of the seismic behaviour of architectural non-structural LWS drywall components, i.e. partitions, façades and ceilings. The tested non-structural components were made of LWS frames made with the adoption of cold-formed steel (CFS) profiles and sheathed with gypsum-based or cement-based boards. All basic components were dry assembled. In particular, the interaction between partitions and surrounding elements and/or façades and ceilings was taken into account during the experimental activity. To this end, four cases of practical application of architectural non-structural LWS drywall components installed in a surrounding structure, i.e. reinforced concrete structure, were considered: (a) Case A, in which partition interacted with structural elements; (b) Case B, in which partition interacted with both structural and non-structural elements; (c) Case C, in which façade interacted with structural elements; (d) Case D, in which ceiling interacted with non-structural elements. Therefore, four architectural non-structural LWS drywall components representative of the corresponding cases of application were identified: (1) Component 1 representing Case A, in which partition was infilled in the surrounding structure and enclosed by structural elements on all sides (i.e. floors or beams and columns); (2) Component 2 representing Case B, in which partition was enclosed by structural elements at the top and bottom (i.e. floors or beams) and connected at its ends to transversal façades (return walls); (3) Component 3 representing Case C, in which façade was infilled in the surrounding structure and enclosed by structural elements on all sides (i.e. floors or beams and columns); (4) Component 4 representing Case D, in which ceiling was suspended from the above floors and connected at the perimeter to partitions and façades.

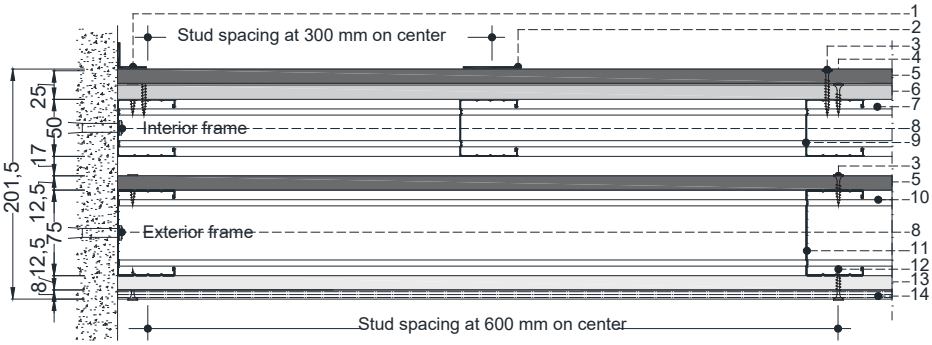
The tested partitions (Figure 1) were made of a single LWS frame, i.e. lipped channel section stud profiles spaced at 300 or 600 mm on centre and connected at the ends to unlipped channel section track profiles, sheathed with double layer of 12.5 mm thick standard gypsum (GWB) or gypsum-fibre (GFB) boards installed on both partition faces. The total partition thickness was equal to 125 mm. The façades (Figure 2) were realized with double LWS frames, i.e. interior and exterior frames. The interior frame was sheathed with two layers of 12.5 mm thick GWB and impact resistant gypsum boards (RGWB) installed on the outer frame face, whereas 12.5 mm thick RGWB and outdoor cement boards (CP) were placed at the inner and outer face of the exterior frame, respectively. The total façade thickness was equal to 201.5 mm. The ceilings (Figure 3) were made of a double level of LWS profiles, i.e. upper carrying profiles spaced at 1000 mm on centre and connected with metallic clips to transversal lower furring profiles spaced at 500 mm on centre.

Carrying profiles were placed at a distance equal to 500 mm from the floor by means of variable adjustable suspenders. The steel frame was completed at the bottom face with a single layer of 12.5 mm thick sound shield boards (SSB). The total ceiling thickness was equal to 68.6 mm. The tested non-structural components were connected to both structural and non-structural surrounding elements by means of two different typologies of connections: basic and enhanced anti-earthquake connections. The relative displacements between non-structural components and surrounding elements were restrained in the case of basic connections, whereas the in-plane displacements were allowed in the case of enhanced anti-earthquake connections. Specifically, basic connections were made by fixing sheathing boards to surrounding profiles, whereas surrounding profiles and sheathing boards were not connected in the case of enhanced anti-earthquake connections. Figure 4 shows the adopted connection typologies for partitions and ceilings.



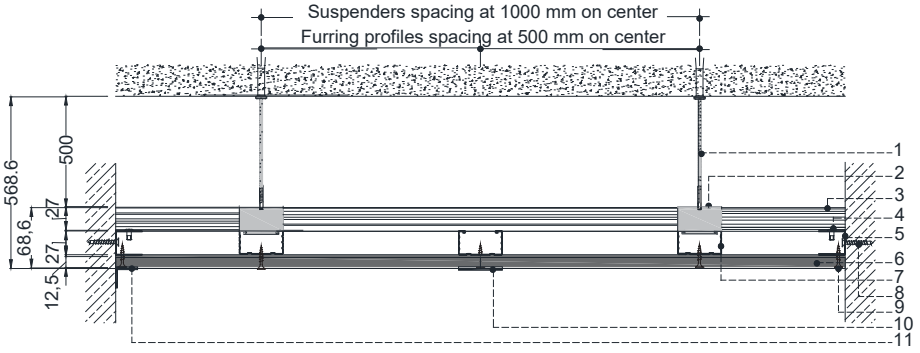
1. 12.5 mm thick standard gypsum or gypsum fibre boards; 2. Track profile (75x40x0.6 mm); 3. Stud profile (75x50x7.5x0.6 mm); 4. 6 mm (or 8 mm) drilling hole diameter steel or plastic dowel spaced from 500 to 900 mm; 5. 3.5 mm nominal diameter self-piercing screws spaced at 700 mm; 6. 3.5 mm nominal diameter self-piercing screws spaced at 250 mm; 7. Glass fibre tape with alkaline-resistant coating (or paper tape) fixed with gypsum-based plaster; 8. Glass fibre tape fixed with gypsum-based plaster (or self-adhesive paper tape)

Figure 1. Horizontal section of the tested partitions (lengths in mm)



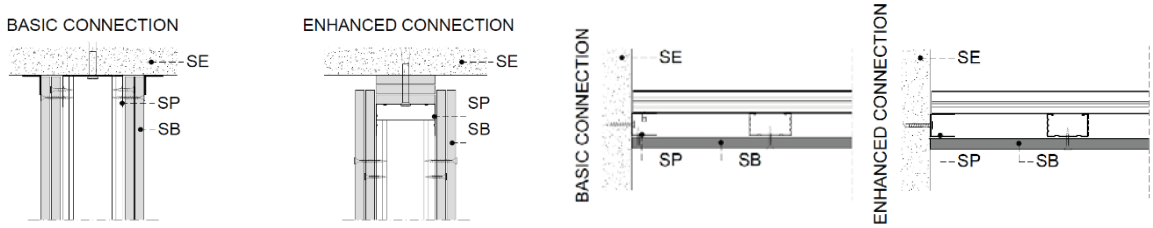
1. Self-adhesive paper tape; 2. Paper tape fixed with gypsum-based plaster; 3. 3.9 mm nominal diameter self-piercing screws spaced at 250 mm; 4. 3.5 mm nominal diameter self-piercing screws spaced at 700 mm; 5. 12.5 mm thick impact resistant gypsum board; 6. 12.5 mm thick standard gypsum board; 7. Track profile (50x40x0.6 mm); 8. 6 mm (or 8 mm) drilling hole diameter plastic dowel spaced from 500 to 600 mm; 9. Stud profile (50x50x7.5x0.6 mm) spaced at 300 mm; 10. Track profile (75x40x0.8 mm); 11. Stud profile (75x50x7.5x0.8 mm) spaced at 600 mm; 12. 4.2 mm nominal diameter self-drilling screws spaced at 200 mm; 13. 12.5 mm thick outdoor cement board; 14. Glass fibre tape with alkaline-resistant coating fixed with cement-based plaster

Figure 2. Horizontal section of the tested façades (lengths in mm)



1. Variable adjustable suspender; 2. Metallic clip; 3. Carrying profile (50x27x7.5x0.6 mm); 4. 4.2 mm nominal diameter self-tapping screw; 5. Track profile (27x30x0.6 mm); 6. 12.5 mm thick sound shield board; 7. Furring profile (50x27x7.5x0.6 mm); 8. 3.5 mm nominal diameter self-piercing screw spaced at 250 mm; 9. 3.5 mm nominal diameter self-piercing screw spaced at 200 mm; 10. Paper tape fixed with gypsum-based plaster; 11. Self-adhesive paper tape

Figure 3. Vertical section of the tested ceilings (lengths in mm)



a) b)

SE: surrounding element; SB: sheathing board; SP: surrounding profile

Figure 4. Connections for partitions (a) and ceilings (b)

2.2 TEST PLAN

With the aim of having a broad vision of the local and global response of the tested components, the research activity was organized in three levels: ancillary tests, component tests and assembly tests. The attempt of ancillary tests was to characterize the mechanical behaviour of steel material, screws, sheathing boards and board-to-frame fixings [Fiorino *et al.,* 2017d]. Component tests were performed on full-scale partitions for assessing the out-of-plane and in-plane behaviour. The goal was to provide answers to the prescriptions for out-of-plane and in-plane design of partitions according to Eurocode 8. As far as the out-plane design of partitions is concerned, the main unknown variables, which play a significant role in the seismic verification of acceleration-sensitive components, to be to estimated are the out-of-plane design resisting force and the fundamental vibration period. Therefore, three-point bending tests under quasi-static monotonic loads were performed in the out-of-plane direction of full-scale partitions for evaluating the wall design resisting force and out-of-plane dynamic identification tests, namely step-relaxation tests, were carried out for defining the fundamental vibration period [Fiorino *et al.*, 2018a]. Specifically, out-of-plane quasi-static monotonic and dynamic identification tests were performed on Component 1. As far as the in-plane behaviour is concerned, the seismic verification of deformation-sensitive components defined according to Eurocode 8 requires that the non-structural components should satisfy the damage limitation requirement obtained by limiting the design inter-storey drifts (IDRs) of the main structure to the code-specific values. Therefore, in-plane quasi-static reversed cyclic test were performed for investigating the damages and the seismic fragilities of partitions [Pali *et al.*, 2018]. In particular, in-plane quasi-static reversed cyclic tests were conducted on Components 1 and 2. Finally, the dynamic behaviour was estimated by means of shake table tests, which were carried out for evaluating the out-of-plane behaviour of partitions and the in-plane behaviour of partitions, façades and ceilings [Fiorino *et al.*, 2019]. In particular, shake table tests were performed on different assemblages of Components 1, 2, 3 and 4. Table 1 summarizes the matrix for component and assembly tests.

Table 1. Test matrix

Test type		Component	Direction of the seismic action[a]	No. of tests
Component tests	Out-of-plane quasi-static monotonic tests	1	Out-of-plane	22
	Out-of-plane dynamic identification tests	1	Out-of-plane	11
	In-plane quasi-static reversed cyclic tests	1	In-plane	8
		2	In-plane	4
Assembly tests	Shake table tests	1	Out-of-plane, in-plane	5
		2	Out-of-plane	
		3	In-plane	
		4	In-plane	
			Total no. of tests	*50*

[a] Direction of the seismic action respect to the component plane

3. OUT-OF-PLANE TESTS

3.1 QUASI-STATIC MONOTONIC TESTS

Out-of-plane quasi-static monotonic tests were performed with the main aim to identify the out-of-plane behaviour of partitions in terms of strength, stiffness and damage phenomena. In particular, three-point bending tests under quasi-static monotonic loads were carried out on the Component 1 (Figure 5). Two typologies of partitions (Component 1) were tested: (1) 1800 mm long and 2700 mm high walls, named "tall partitions"; and (2) 1800 mm long and 600 mm high walls, named "short partitions". For the sake of brevity, only tests on "tall partitions" are deepened in this paper. The main objectives of tests on "tall partitions" were to investigate the damage phenomena and evaluate the out-of-plane strength and fundamental vibration period that are required by the seismic verification of acceleration-sensitive components according to Eurocode 8 Part 1. Test program was organized in order to investigate the following parameters: (1) Partition height (600 or 2700 mm); (2) stud spacing (300 or 600 mm); (3) types of partition-to-surrounding connections, (basic or enhanced anti-earthquake connections) for realizing the horizontal connections between partitions and top and bottom beams; (4) dowel types for realizing the partition-to-surrounding fixings, i.e. plastic or steel; (5) gap between sheathing boards and surrounding elements, i.e. 20 or 30 mm for enhanced anti-earthquake connections. A total number of 14 test were carried out on "tall partitions". The tests were performed by adopting a specific test set-up designed for applying the monotonic load in the out-of-plane direction at the mid-span of partitions arranged in horizontal position. Specimens were subjected to progressive displacements up to failure under a displacement-controlled procedure.

The parameters used to describe the experimental behaviour were defined on the load (F) versus displacement (d) curves. The response curves obtained by monotonic tests on "tall partitions" are shown in Figure 6. The partitions showed a behaviour initially characterized by an increasing trend of the load as the displacement increased until the first-peak load was reached. After that, a softening behaviour followed by a load increasing up to the second-peak load was observed and the load reduction was detected at the end of tests. The defined parameters were the first-peak strength (F_{1st}), the second-peak strength (F_{2nd}) and the conventional elastic stiffness (k_e), which was assumed equal to the ratio between the conventional elastic limit load equal to $0.4F_{1st}$ and the relevant displacement. Physical phenomena related to the wall framing local buckling generally characterized the initial behaviour of the tested walls and, in particular, stud local buckling (Figure 7) was observed at the first-peak load, whereas the second-peak load was reached when flexural cracking of sheathing boards occurred. The examination of test results highlighted that the out-of-plane quasi-static response of partitions was affected by stud spacing (300 or 600 mm) and the strength and stiffness doubled their values when 300 mm stud spacing was used. Partition-to-surrounding connections

do not significantly affect the out-of-plane quasi-static response and they can be schematized as simple supports.

Figure 5. Out-of-plane quasi-static monotonic tests on Component 1

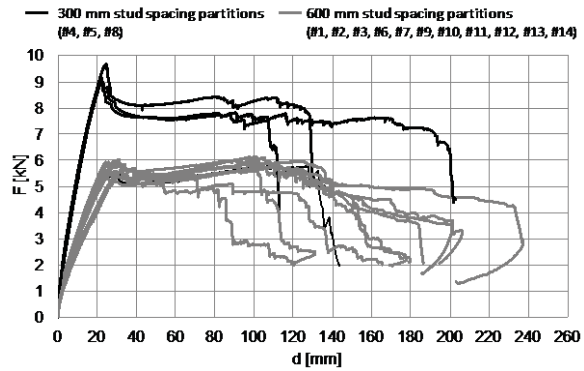


Figure 6. Response curves for out-of-plane quasi-static monotonic tests on "tall partitions"

Figure 7. Stud local buckling

3.2 DYNAMIC IDENTIFICATION TESTS

Out-of-plane dynamic identification tests were carried out only on "tall partitions" in order to experimentally quantify the fundamental vibration frequency and damping ratio. A total number of 11 tests were carried out on "tall partitions". The tests were performed by adopting the same test set-up designed for out-of-plane quasi-static monotonic tests and described in the previous section, with few changes in order to allow the free vibration of the wall.

Dynamic identification tests provided the displacement (*d*) versus time (*t*) curves and Figure 8 shows typical experimental curves. The damping ratio and experimental fundamental vibration frequency were evaluated. Stud spacing influences also the out-of-plane dynamic response of partitions, with a reduction of damping ratio and an increasing of the fundamental vibration frequency for 300 mm stud spacing partitions. According to the definition of rigid and flexible architectural non-structural components provided by ASCE/SEI 7-10, partitions with 600 mm stud spacing can be considered as flexible components because they were characterized by values of frequency lower than 16.67 Hz, whereas partitions with 300 mm stud spacing had a borderline behaviour in terms of dynamic stiffness classification.

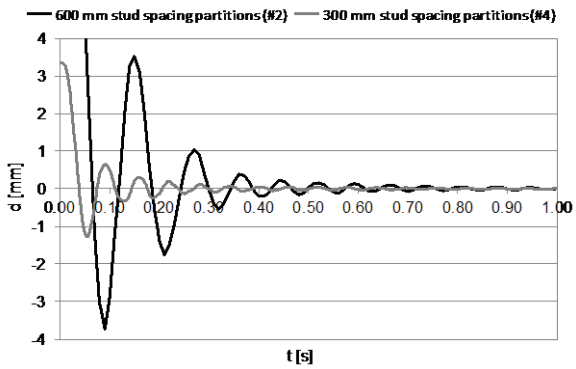


Figure 8. Response curves for out-of-plane dynamic identification tests

4. IN-PLANE TESTS

In order to experimentally assess the seismic fragility and the related damage levels in accordance with the IDR limits defined by Eurocode 8 Part 1 for deformation-sensitive components, an experimental campaign involving in-plane quasi-static reversed cyclic tests on partitions (Components 1 and 2) was performed (Figure 9). In particular, 2400 long and 2700 high partitions were used for Components 1 and 2 and 600 long and 2700 high façades were selected as return walls in Components 2. Different parameters were investigated for defining the experimental program: (1) stud spacing (300 or 600 mm); (2) types of partition-to-surrounding connections (basic or enhanced anti-earthquake connections) for realizing the horizontal and vertical connections between partitions and surrounding elements; (3) sheathing board types (GWB or GFB); (4) jointing finishing types. A total number of 8 and 4 tests were carried out on Components 1 and 2, respectively. A specific test set-up, which replicated the behaviour of a typical storey of a building structure, was designed to carry out the in-plane cyclic tests. The in-plane cyclic tests, performed in displacement-controlled test procedure, were carried out by adopting a loading protocol defined according to FEMA 461 [2007], which consisted of repeated cycles of step-wise increasing deformation amplitudes. In the specific case, the loading protocol included 18 steps with imposed IDRs, which are defined as the ratios between the recorded displacement at the wall top and the partition height (2700 mm), ranging from 0.08% to 8.40%.

Response curves were provided in terms of load (*F*) versus IDR. Figure 10 shows the response curves obtained for #1 (Component 1 with basic connections) and #6 specimens (Component 1 with enhanced anti-earthquake connections). The hysteretic behaviour of partitions was strongly characterized by pinching phenomenon, stiffness and strength degradation when IDR increased. The obtained first cycle envelope curves (Figure 11) show that partitions with basic-connection provided additional strength and stiffness to the surrounding elements starting from the initial phase of the response. On the contrary, partitions with enhanced-connection provided additional strength and stiffness for more high IDRs, i.e. when the contact between sheathing boards and surrounding elements was restored. Partition-to-surrounding connections significantly affect the in-plane response and a lower stiffness and strength were recorded for enhanced connections compared to basic connections. Finally, the stud spacing (300 or 600 mm) does not influence the in-plane response.

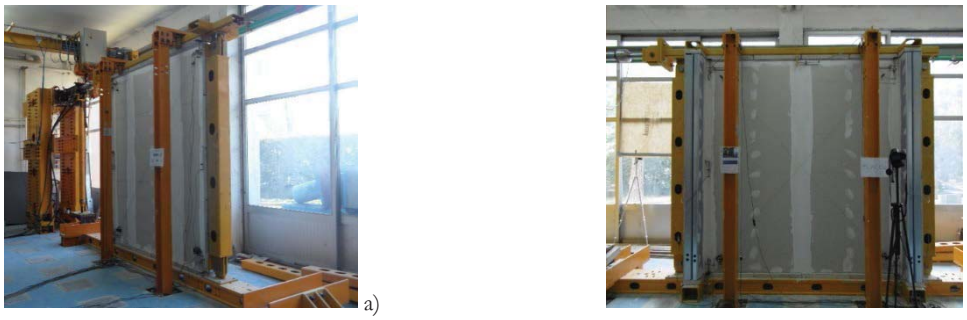


Figure 9. In-plane quasi-static reversed cyclic tests: a) test on Component 1; b) test on Component 2

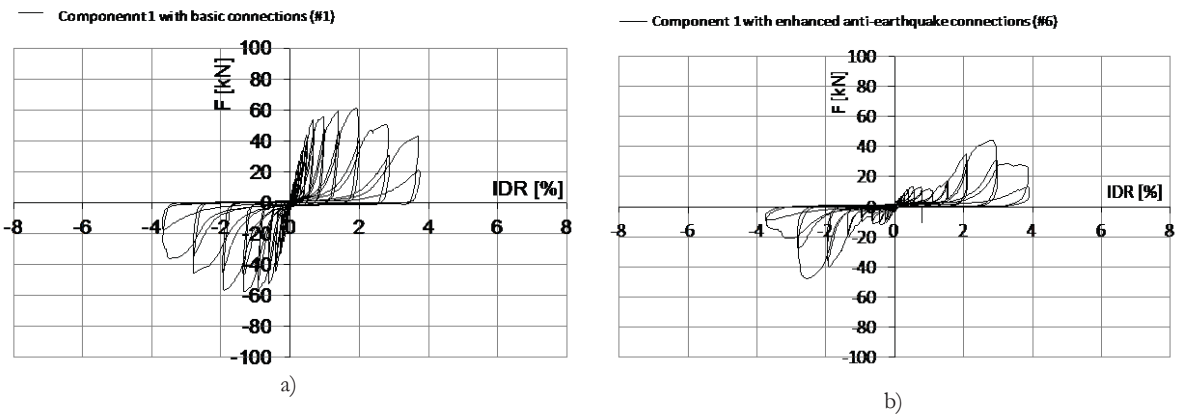


Figure 10. Response curve for in-plane quasi-static reversed cyclic test on: a) #1 specimen; b) #6 specimen

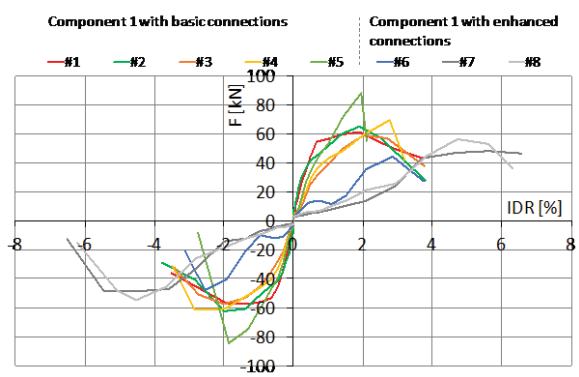


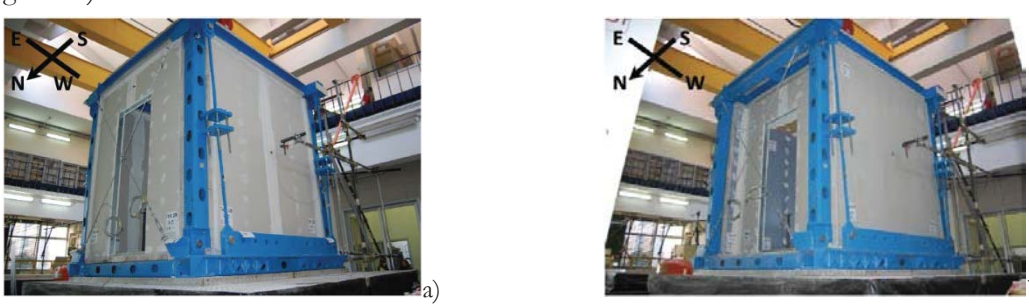
Figure 11. Backbone curves for Components 1

5. SHAKE TABLE TESTS

Shake table tests were performed for evaluating experimentally the dynamic properties, dynamic amplification and seismic fragilities of partitions, façades and ceilings. Tests were carried out by means of the shaking-table available at the University of Naples "Federico II". The test set-up (Figure 12), representative of a reinforced concrete bare structure, was made of a bottom and a top steel beam grid connected by four columns. The lateral structural restraint systems in the shaking direction (E-W direction) was an eccentric bracing system, in which diagonal members were pretensioned truss elements with

rectangular cross section, whereas in N-S direction the test set-up was braced by means of X-bracings made of steel cables. Shake table tests were carried out on two assemblages of different components: (1) Assembly 1 composed by four partitions (Components 1) placed in both E-W and N-S directions; (2) Assembly 2 consisting of two partitions (Components 2) placed in N-S direction, two façades (Components 3) placed in E-W direction and one ceiling (Component 4). For both Assemblages 1 and 2, the solutions with basic and enhanced anti-earthquake connections were investigated. A total number of 3 and 2 prototypes were tested on Assemblages 1 and 2, respectively. The seismic input was a unidirectional acceleration time history artificially defined to match the Required Response Spectrum (RRS) provided by ICBO-AC156 [2000] code acting along the E-W direction. The selected input time history was applied with different scaling factors (SFs) in the range from 5% to 120%, corresponding to a maximum horizontal flexible acceleration ($A_{FLEX,H}$ in ICBO-AC156, which represents the maximum spectral acceleration) ranging from 0.08 g to 1.92 g, i.e. SF=100% corresponds to a_g=0.4g, SDS=1.0g, and $A_{FLEX,H}$=1.6g. Dynamic identification tests were carried out before and after each input by applying a white noise signal.

Results show that the non-structural components provided an increment of the fundamental vibration frequency of the bare structure in case of Assembly 1 and Assembly 2. The Assemblages 1 and 2 with enhanced-connections reached higher values of fundamental vibration frequency than the Assemblages 1 and 2 with basic-connections. The presence of non-structural components also increased the damping ratio of the bare structure. The type of partition or façade-to-surrounding connections influenced the damping ratio and, in fact, Assembly 1 and 2 with enhanced anti-earthquake connections reached higher values of damping ratio respect to the values recorded for Assembly 1 and 2 with basic connections. The dynamic amplification of tested components can be evaluated by comparing the peak component acceleration (PCA) and the peak bare structure acceleration (PBA) measured by accelerometers installed on components and bare structure, respectively (Figure 13). The examination of test results points out that the dynamic amplification increased as PBA increased, due to the reduction of stiffness caused by the increment of damages of components. The acceleration amplification for in-plane response was in the range from 1 to 4 for Component 1 (partitions) and from 1 to 3 for Component 3 (façades). Regarding the effect of the partition or façade-to-surrounding connection types, Assembly 1 and 2 with enhanced anti-earthquake connections revealed a more flexible behaviour with higher values of the in-plane acceleration amplification (up to 4 and to 3 for Component 1 – partitions - and Component 3 – façades -, respectively) respect to Assemblages 1 and 2 with basic connections with values up to 2 for both Component 1 (partitions) and Component 3 (façades). Furthermore, the acceleration amplification for out-of-plane response of Component 1 and 2 (partitions) and in-plane response of Component 4 (ceiling) were in the range from 1 to 2 (Figure 14).



a) b)

Figure 12. Shake table tests: a) test on assembly 1; b) test on assembly 2

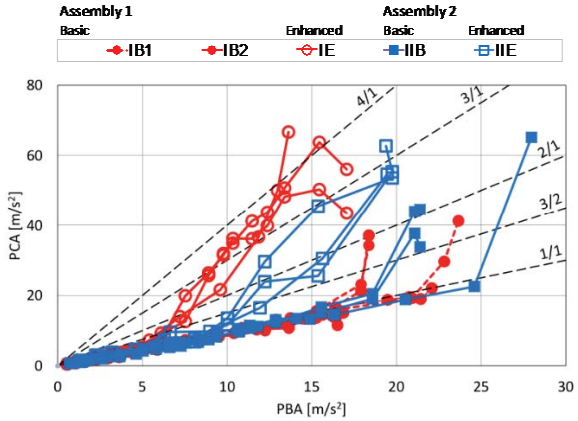


Figure 13. In-plane dynamic amplification for Components 1 and 3

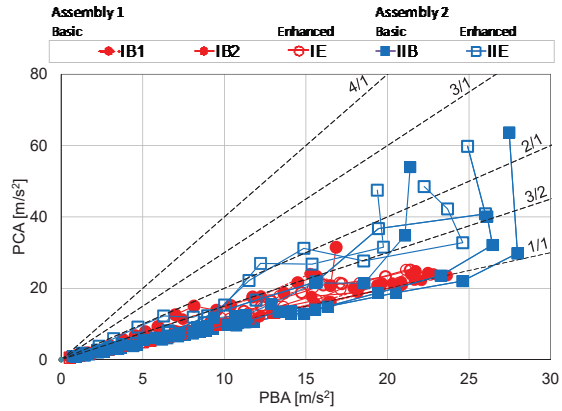
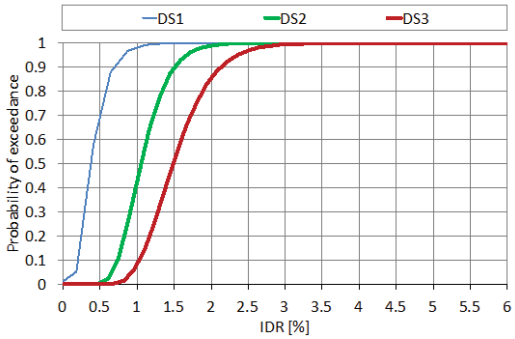


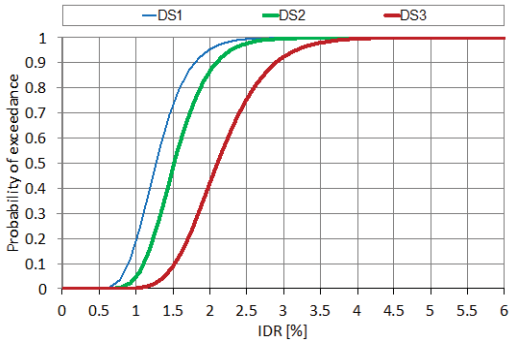
Figure 14. Out-of-plane dynamic amplification for Components 1 and 2

6. SEISMIC FRAGILITY EVALUATION FOR IN-PLANE RESPONSE

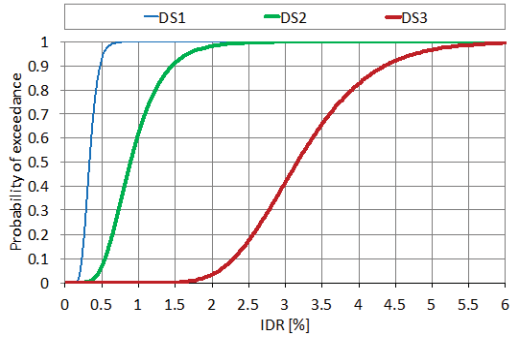
The seismic fragility evaluation was performed by elaborating test results obtained by both in-plane quasi-static reversed cyclic tests and shake table tests. In particular, the proposed fragility curves refer to the in-plane seismic response of Components 1, 2 and 3. A procedure articulated in 5 steps was adopted for developing fragility curves. Firstly (step 1), three damage limit states (DSs) were evaluated on the base of a large database of tests [Retamales *et al.,* 2013]. The DSs were defined according to the observed damage level and the required repair action as following: DS1, which is characterized by superficial damage and it requires minimum repair with plaster, tape and paint; DS2, which is characterized by local damage of sheathing boards and/or steel frame and it requires the replacement of few elements (boards and/or local repair of steel profiles); DS3, which is characterized by severe damage and it requires the replacement of significant parts or whole non-structural component. Then (step 2), the damage phenomena were observed during the experimentation by means of visual inspection and classified. Subsequently (step 3), the damage phenomena were correlated to the DSs. Therefore (step 4), because the in-plane behaviour is primarily governed by IDRs, the damage phenomena were associated to the IDR levels at which each phenomenon started. Finally (step 5), the seismic fragility assessment of the tested components was performed by elaborating test results for developing fragility curves. Fragility curves were evaluated according to the method 'A' indicated by Porter et al. [2007]. Because the behaviour of tested components was particularly affected by component typology (Components 1, 2 and 3), partition or façade-to-surrounding connections (basic vs. enhanced anti-earthquake connections) and loading protocol (quasi-static cyclic vs dynamic shake table), all other variations were neglected and the fragility data were collected in eight Groups: (A) Components 1 with basic connections subjected to quasi-static loading; (B) Components 1 with enhanced connections subjected to quasi-static loading; (C) Components 1 with basic connections subjected to dynamic loading; (D) Components 1 with enhanced connections subjected to dynamic loading; (E) Components 2 with basic connections subjected to quasi-static loading; (F) Components 2 with enhanced connections subjected to quasi-static loading; (G) Components 3 with basic connections subjected to dynamic loading; (H) Components 3 with enhanced connections subjected to dynamic loading. Figure 15 shows the fragility functions for the selected Groups.



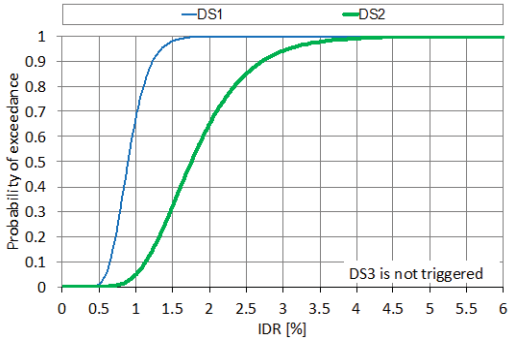
A. Components 1(1) with basic connections subjected to quasi-static loading



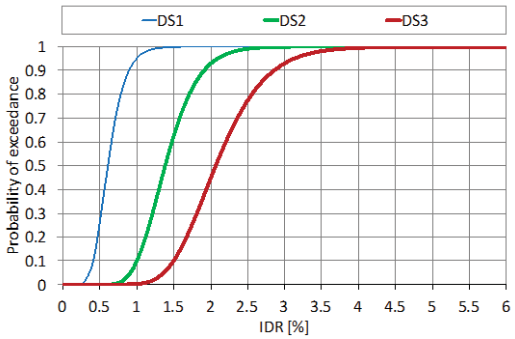
B. Components 1(1) with enhanced connections subjected to quasi-static loading



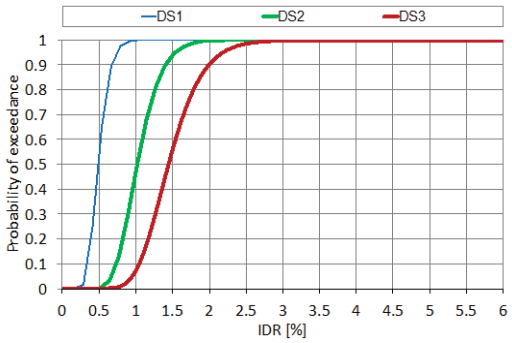
C. Components 1(1) with basic connections subjected to dynamic loading



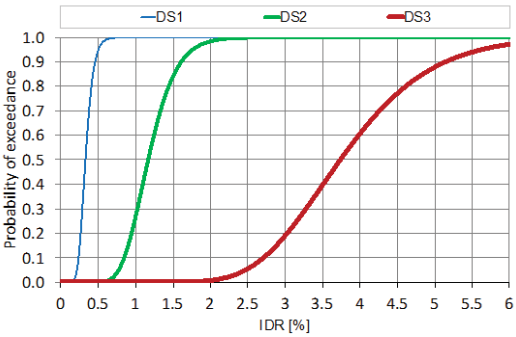
D. Components 1(1) with enhanced connections subjected to dynamic loading



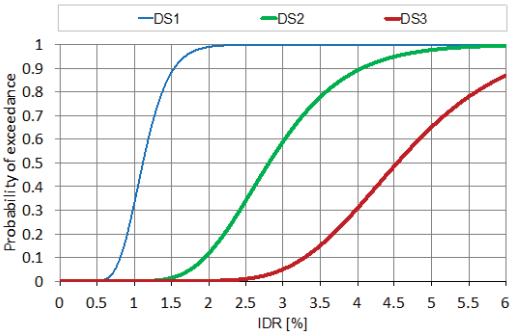
E. Components 2(2) with basic connections subjected to quasi-static loading



F. Components 2(2) with enhanced connections subjected to quasi-static loading



G. Components 3(3) with basic connections subjected to dynamic loading



H. Components 3(3) with enhanced connections subjected to dynamic loading

(1) Component 1 representing of partitions infilled in a surrounding structure and enclosed by structural elements on all sides (i.e. floors or beams and columns); (2) Component 2 representing of partitions enclosed by structural elements at the top and bottom (i.e. floors or beams) and connected at their ends to transversal façades (return walls); (3) Component 3 representing of façades infilled in a surrounding structure and enclosed by structural elements on all sides (i.e. floors or beams and columns).

Figure 15. Fragility curves

Partition-to-surrounding connections influence the in-plane response in term of seismic vulnerabilities, and enhanced connections showed a better behaviour for partitions (Components 1) and façades (Components 3), except for partitions with return walls (Components 2), for which basic connection revealed a better seismic response. The results show that façades had a lower seismic vulnerability than partitions. Considering a reasonable limit for the probability of exceedance equal to 5%, and assuming the most onerous results between quasi-static cyclic and shake table test results, the IDR limits provided by Eurocode 8 Part 1 have been attributed to three groups of components and components with high (with an IDR limit of 0.75% for DS3), intermediate (with an IDR limits of 0.75 and 1.00% for DS2 and DS3) and low (with an IDR limit of 1.00% for DS2) seismic fragility were identified.

7. CONCLUSIONS

An extended experimental research was performed with the main goal to characterize the seismic behaviour of architectural non-structural LWS drywall components, i.e. partitions, façades and ceilings. As far as the out-of-plane behaviour of partitions (Components 1) is concerned, the main findings showed that the responses in terms of strength, stiffness, fundamental vibration frequency and damping ratio were strongly affected by stud spacing. The out-of-plane dynamic response of partitions in terms of acceleration amplification was not affected by the connection type and partitions with both basic and enhanced connections showed values in the range from 1 to 2. As far as the in-plane behaviour is concerned, results reveal that the responses of partitions (Components 1 and 2) and façades (Components 3) were not affected by stud spacing, whereas the partition or façade-to-surrounding connections played an important role on the seismic performance. Because the enhanced-connections had minor interaction with surrounding elements, they revealed a more flexible behaviour with higher values of the in-plane acceleration amplification respect to basic connections. Finally, high, intermediate and low fragility components can be classified by taking into account the IDR limits provided by Eurocode 8 Part 1.

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