Seismic Behaviour of Strap-Braced LWS Structures: Shake Table Testing and Numerical Modelling

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Abstract. Nowadays Lightweight Steel (LWS) Constructions made with Cold-Formed Steel (CFS) profiles are currently used in seismic area, especially for residential buildings. The increasing use of these systems is accompanied by a large development of the research in this field. Indeed, the University of Naples “Federico II” has recently started an important collaboration with Lamieredil S.p.A. Company in order to investigate the seismic behaviour of strap-braced CFS systems through a series of shake-table tests, performed on two reduced-scale strap-braced structures. In addition, 3D numerical models were developed in OpenSees environment in order to simulate the dynamic/earthquake response of the whole structure, considering also the effect of the two different floor typologies. The present paper illustrates the main results of the shake-table testing and numerical modelling.

1. Introduction
In the last decade, many national and international research activities were undertaken in order to evaluate the seismic behavior of structures. In particular, the University of Naples "Federico II" dedicated specific researches to steel structures [1-13] and lightweight steel (LWS) constructions made with Cold-Formed Steel (CFS) profiles [14-20], focusing the attention on many issues. In this framework, an important cooperation between the University of Naples “Federico II” and Lamieredil S.p.A. company started in the last years. In particular, after monotonic and cyclic tests on full scale shear walls, carried out in past studies, the dynamic response of CFS strap-braced structures was analyzed by performing shake table tests on two reduced-scale (1:3) three-storeys CFS mock-ups. Two prototypes differ for the floor typology: (1) composite steel-concrete floors (CONCRETE solution) or (2) floors made of wood-based panels (OSB). In order to represent the experimental seismic response, a specific task has been devoted to develop 3D numerical models with OpenSees [19] software. Firstly, the numerical models of CFS strap-braced stud walls were calibrated on the base of the available experimental results, i.e. quasi-static cyclic tests on single walls [22], then numerical models of whole prototype were developed in order to simulate the non-linear dynamic response. The paper provides a description of shake table tests and numerical modelling of CFS strap-braced structures. Moreover, the experimental data and the numerical modelling results are compared and discussed in order to validate the developed models.

2. Shake-table testing
Two reduced scale mock-ups (figure 1) tested on the shake-table which representative of three-story and two bay systems, in which each storey was composed of three CFS strap-braced stud walls placed in one direction only. The two tested mock-ups differed with each other with respect to the floor typology, which was either composite steel-concrete floors (CONCRETE solution) or floors made of
wood-based panels (OSB solution). Mock-ups tested on the shake table were scaled representations of the seismic force resisting zones of the case studies buildings, which were rectangular in plan with dimensions of 12.2 x 18.1 m and each storey 3 m high. The scaling procedure consists of fixing three independent scaling factors, which were length ($S_L=1/3$), stress ($S_\sigma=1$) and acceleration ($S_a=1$), while scaling factors of other parameters (dependent) are defined according to the similitude laws [23]. As a result of scaling process, the mock-ups had a rectangular plan, which covers an area of about 0.95 × 2.95 m, an inter-storey height of about 1.05 m with a total height of about 3.10 m and CFS strap-braced walls having dimension 800 x 900 mm (length x height). Response of the mock-ups was evaluated using the shake-table of Department of Structures for Engineering and Architecture, University of Naples “Federico II”. Dynamic properties of mock-ups in shorter direction were evaluated through dynamic identification tests under white noise signals. In order to analyze experimental data obtained, experimental modal analysis technique was used, thanks to ARTeMIS Modal [24] software. Earthquake response was evaluated through shaking the mock-ups under natural ground motion record from 2016 Central Italy earthquake (Norcia station). The earthquake record was scaled at various intensities to replicate various ground motion hazard levels. Each test under an earthquake record was preceded by the random vibration test to track the progress of damage in structures through variation in natural period and damping ratios. Table 1 shows the scaling factors for earthquake records for both mock-ups.

![Mock-ups](image)

**Figure 1.** Reduced scale tested mock-ups.

**Table 1.** Scaling factors for earthquake records for both mock-ups.

| Mock-up        | Test label | Time history scaling factor          |
|----------------|------------|--------------------------------------|
| CONCRETE solution | TH1, TH2, TH3, TH4, TH5, TH6, TH7 | 9%, 12%, 38%, 49%, 100%, 120%, 150% |
| OSB solution   | TH1, TH2, TH3, TH4, TH5, TH6, TH7 | 11%, 16%, 48%, 62%, 100%, 120%, 150% |

### 3. Numerical modelling

![Model options and parameters](image)

**Figure 2.** Modelling options and parameters used

OpenSees software [21] with a goal to better understand the contribution of several factors in the experimental seismic response of specimens and to evaluate the better options for modelling. The primary seismic force resisting system of the mock-ups, i.e. strap-braced stud walls was modelled using a pair of diagonal truss elements representing the straps of walls [25]. To simulate their hysteretic response, for the diagonal truss elements Pinching4 material [26] was used. In particular,
test data [22] was used to calibrate the backbone envelope and the parameters governing the cyclic behaviour in Pinching4 material rule. Pinching4 material can be defined through the set of 39 parameters, which 16 define the backbone curve, calibrated on the basis of available cyclic test data [22], and 23 control the cyclic behaviour. Model capacity to capture the wall behaviour was evaluated by comparing the experimental and numerical force-displacement responses (figure 2a) and the dissipated energy under same loading protocol (figure 2b). The definition of the position of numerical backbone points for respective walls were same in both specimen models (OSB and CONCRETE). In detail, the first point was positioned at 50% of the experimental yielding strength, the second point corresponds to the elastic limit, the third point represents the experimental peak strength and fourth point was used to capture the post peak behaviour, setting the displacement as maximum experimental displacement achieved and finding the corresponding force with an energy balance. Moreover, experimental results showed a vertical drop of strength after the peak. However, this steep drop of strength may lead to convergence problems, consequently post peak branch of the backbone envelope was aligned with little positive slope downwards using the fourth point. Four points of backbone envelope are listed in table 2. Additionally, among parameters controlling cyclic behaviour, the parameter governing ratio (rDisp) between the displacement of the reloading point and maximum positive displacement of preceding cycles was taken as 0.1, 0.05 and 0.1 for W1, W2 and W3 walls, respectively. Rest of the parameters required to define material rule for Pinching4 material were taken as 0. Model capacity to capture the single wall behaviour is evaluated by comparing the experimental [22] and numerical responses. Figure 2a and 2b shows the comparison of force-displacement response and cumulative energy dissipation for W3 wall configuration. Connections present at the ends of each wall in specimens to transmit the axial forces from studs to floors or foundation were modelled with two parallel Zerolength elements (figure 2c). The first Zerolength element was modelled with uniaxial elastic material representing the stiffness of hold down connection only, while the second element was assigned with Elastic-perfectly plastic gap material with a zero-gap representing the compression stiffness offered by foundation or floor. Uniaxial elastic material in case of the CONCRETE specimen had a stiffness of 20, 10 and 7.5 KN/mm for W1, W2 and W3 walls, respectively, whereas in case of OSB specimen it had a stiffness of 20, 20 and 15 KN/mm for W1, W2 and W3 walls, respectively. Elastic perfectly-plastic gap material had a stiffness of 20 KN/mm for foundation, while for floors, stiffness was negligible in comparison with the foundation. The yield strength used for Elastic perfectly-plastic gap material was also extracted experimentally [22]. Floor was modelled by means of rigid beam-column elements in both CONCRETE and OSB solutions. Since the motion of shake table was uni-directional in x direction and the motion in the y direction was restrained, also in the model the motion in other degree of freedom was restrained. Seismic mass of the specimen, evaluated considering differences between CONCRETE and OSB solution, was distributed at the top end nodes of walls at each floor. Chord studs were modelled using truss elements having Uniaxial elastic material. Rayleigh damping was set to 5%, after a sensitivity analysis.

| MATERIAL- ELEMENT | WALL | CONCRETE | OSB |
|-------------------|------|----------|-----|
| Pinching4 material - backbone envelope (stress in MPa, strain in mm/mm) | W1   | (61379; 0.00053), (306895; 0.00877), (46439; 0.00695), (232194; 0.00911) | (283316, 0.01007), (245516, 0.00368) |
|                   | W2   | (31607, 0.000643), (158033, 0.00959), (185755, 0.00391) | (222780, 0.010483) |
|                   | W3   | (61379; 0.00053), (306895; 0.00877), (46439; 0.00695), (232194; 0.00911) | (283316, 0.01007), (245516, 0.00368) |
| Elastic uniaxial material - zero length elements stiffness (N/mm) | W1   | 20000 | 20000 |
|                   | W2   | 10000 | 20000 |
|                   | W3   | 7500 | 15000 |
| Elastic-perfectly plastic gap material - zero length elements stiffness (N/mm) | W1   | 20000 | 20000 |
|                   | W2   | 1 | 1 |
|                   | W3   | 1 | 1 |

Table 2. Truss and Zerolength element properties.
4. Experimental vs numerical results

To check the validity of models, experimental and numerical dynamic properties obtained were compared in terms of fundamental period and modal shapes, whereas the seismic behaviour was compared in terms of peak roof drift ratio, defined as the ratio between the maximum top displacement and the height of the structure, and peak inter-storey drift ratio, defined as the ratio between the relative translational displacement difference between two consecutive floors and the storey height.

The numerical models were subjected to modal analysis in order to find the fundamental periods of first (T1), second (T2) and third (T3) vibration modes and modal shapes, whereas experimental periods and modal shapes were determined via experimental modal analysis performed with ARTeMIS Modal software [24]. The experimental results showed that for the CONCRETE prototype the first period of vibration before Earthquake tests was 0.43 s, whereas for the OSB solution it was 0.50 s. The numerical fundamental periods are 0.40 s and 0.50 s for CONCRETE and OSB solution, respectively. The periods of vibration corresponding to the second and third mode were also evaluated and compared. Experimental and numerical values are summarized in Table 3. The vibration modes obtained are compared and represented in Figure 3. It is clear that numerical models give a good prediction of dynamic properties of prototypes for the first period of vibration (experimental-to-numerical ratio of 1.00 and 1.08 for OSB and CONCRETE solution, respectively), whereas the second and third periods of vibration are underestimated by numerical models (experimental-to-numerical ratio in the range 1.25 to 1.43).

|       | CONCRETE |       | OSB  |       |       |
|-------|----------|-------|------|-------|-------|
|       | EXP  | NUM  | EXP/NUM | EXP  | NUM  | EXP/NUM |
| T1 [s] | 0.43 | 0.40 | 1.08 | 0.50 | 0.50 | 1.00 |
| T2 [s] | 0.15 | 0.12 | 1.25 | 0.18 | 0.13 | 1.38 |
| T3 [s] | 0.10 | 0.07 | 1.43 | 0.11 | 0.08 | 1.38 |

Figure 3. Comparison of experimental and numerical modal shapes (OpenSees).

Non-linear time history analysis was performed to evaluate the numerical dynamic behaviour of specimens and to make a comparison with the experimental response. As already introduced, in the experimental activity the earthquakes were applied in sequence to the specimens with increasing scale factors. In order to consider the seismic sequence effects numerical models were subjected to the same strong multiple earthquakes really reproduced by the shake table. In this way, also the signal distortion (differences between signal really reproduced by the shake table and commanded signal) is considered by numerical models. Regarding the seismic response, it can be noticed that the maximum experimental inter-storey drifts were recorded for the input with maximum intensity (scaling factor of 150%) at the 3rd level: 3.49% for CONCRETE solution and 2.21% for OSB solution, whereas the maximum numerical drifts were 1.78% for CONCRETE recorded at 2nd level and 1.33% for OSB recorded at 3rd level. A comparison between experimental and numerical results in terms of peak inter-storey drift (PIDR) is given in Figure 4. The experimental-to-numerical ratio is in the range 0.80 to 2.08 for CONCRETE specimen and 0.65 to 2.06 for OSB specimen. Models are able to capture
PIDR for low inputs, whereas they do not show good predictions for higher inputs. The reason behind the inaccurate response at high intensity earthquakes could be the accumulation of damage and rocking of stories of specimen, which could not be properly captured by numerical models and would be the focus of future studies. Moreover, a comparison between experimental and numerical results in terms of peak roof drift ratio (PRDR) is shown in figure 5.

![Figure 4](image1.png)

**Figure 4.** Comparison between experimental and numerical peak inter-storey drift ratio (PIDR).

![Figure 5](image2.png)

**Figure 5.** Comparison between experimental and numerical peak roof drift ratio (PRDR).

### 5. Conclusions

The paper illustrates the shake-table tests carried out at University of Naples “Federico II” on two three-storeys prototypes in reduced scale (1:3), as a part of cooperation with Lamieredil S.p.A. company. Two mock-ups, which differ only for the type of floors (composite steel-concrete and wood-based panels) were designed and opportunely scaled to be compatible with the available shaking table. The two mock-ups were subjected to white noise signal for dynamic identification and earthquakes for evaluation of seismic behaviour. Moreover, 3D numerical models for both structures were developed in OpenSees environment to represent the experimental response. A comparison between experimental and numerical results was done in term of fundamental periods, mode shapes, peak inter-storey drift and peak roof drift achieved during the time history analysis. The models were able enough to capture the dynamic properties of mock-ups, such as fundamental period and vibration modes of the buildings. Experimental and numerical peak inter-storey drifts (PIDR) and roof drifts (PRDR) were compared and the experimental to numerical ratios were in the range 0.80 to 2.08 (PIDR) and 0.80 to 1.27 (PRDR) for CONCRETE specimen and 0.65 to 2.06 (PIDR) and 0.91 to 1.41 (PRDR) for OSB specimen.
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