Seismic Performance of Sheathed Cold-formed Shear Walls

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Abstract

The paper presents and discusses the results of a research on the seismic behaviour of cold-formed steel stud shear walls, sheathed with wood-based (oriented strand board) and gypsum-based (wallboard) panels. Within this activity, this paper provides the outcomes of the results of experimental (capacity evaluation) and theoretical (demand evaluation) phases of the research. Moreover, a contribution is given for the evaluation of the strength reduction factor of this structural typology.

Introduction

The design of building structures according to standard design philosophy is based on force-reduction factors that, exploiting the structure own ductility, avoid collapse, safeguard human lives and allow a relatively less expensive structural design. In case of light-gauge cold-formed steel framed structures, the building seismic weight is significantly smaller, allowing the design to be carried out with relatively low values of the force reduction factors. This event is particularly favourable, because of the relatively small ductility of this type of structures.

The study presented in the current paper is the core of a research effort being carried out at the University of Naples "Federico II". The paper provides the outcomes of the results of experimental (capacity evaluation) and theoretical (demand evaluation) phases of the research. Moreover, a contribution is given for the evaluation of the strength reduction factor for this structural typology.

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Experimental results

Physical tests have been designed starting from a typical one-family one-story dwelling. The plan dimensions of the house were about 7 x 11m, while its total height about the ground level was about 6m. The structure was a stick-built construction in which both horizontal (roof and floors) and vertical (walls) diaphragms were cold-formed frames sheathed with structural panels.

The experimental program was based on two nominally identical wall sub-assemblages (Della Corte et al. 2003, Fiorino 2003, Fiorino et al. 2004). One sub-assemblage was tested under monotonic loading, the other was instead subjected to a purposely developed cyclic loading history. The generic stud shear wall sub-assemblage is shown in Figure 1.

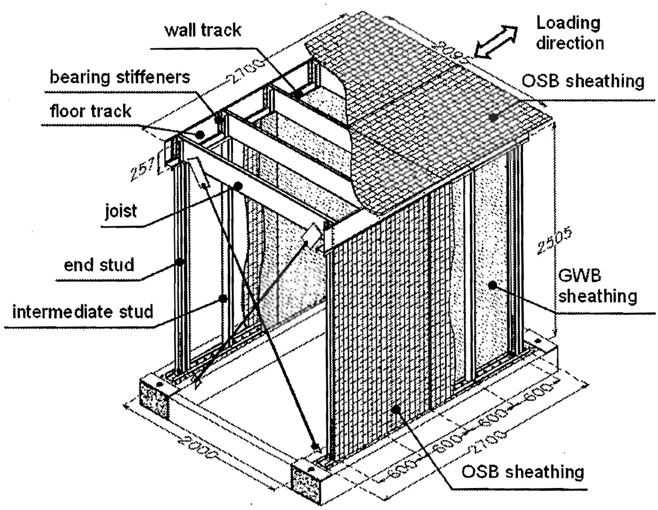


Figure 1. Global 3D view of the tested prototype.

The generic wall framing, which was 2400mm long and 2500mm height, consisted of single top and bottom tracks, single intermediate studs and double back-to-back end studs, spaced 600mm on centre. The floor framing consisted of joists spaced 600mm on centre, with single span of 2000mm. The foundation was simulated by two 280x380mm (depth x width) rectangular concrete beams. The walls were connected to the foundation by intermediate shear anchors and purposely-designed steel hold-down connectors placed in correspondence of the end studs. The main details of the specimen components are reported in Table 1.

All the specimen components (members, panels and connections) were designed according to capacity design principles, in such a way to promote the development of the full shear strength of sheathing-to-wall framing connections.
Two types of load were applied: gravity and racking loads. A gravity load of 45kN was applied on the floor of the prototype. Racking loads were applied to the floor panels by means of two programmable servo-hydraulic actuators.
Fourteen potentiometers were used for measuring displacements of the specimens during tests, as shown in Figure 2. In particular, five potentiometers (w1 through w5) were installed for each wall (Fig. 2a); one potentiometer (f1) was installed for each foundation beam (Fig. 2a); and two potentiometers (d1, d2) were installed on the specimen floor (Fig. 2b). The load was measured through the actuators' load cells. Figure 3 shows a global view and some details of specimen and testing apparatus.
Two load regimes were applied: monotonic and cyclic. In the monotonic regime, the specimen was loaded up to a displacement of 150mm. In the cyclic test, the specimen was subjected to a specific loading sequence based on the results of a numerical study on the probable deformation histories the structure would be subjected to, as better illustrated in the next Section.
The following symbols will be used in the following: V_1 and V_2 forces measured by actuators a1 and a2, respectively; d_1 and d_2 displacements measured by actuators a1 and a2, respectively; L_t=4800mm total length of walls.

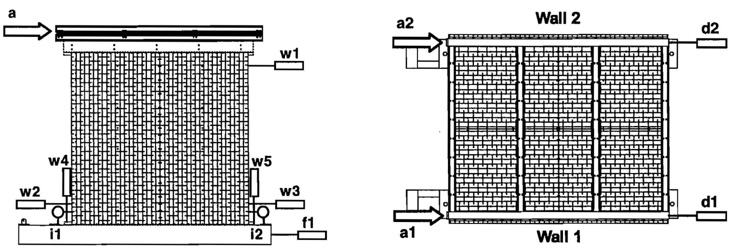
Table 1. Full-scale specimen materials and construction data.

Cold formed steel members		
Steel grade	FeE350G (S350GD+Z/ZF) hot dipped galvanized (zinc coated) steel	
Wall members	Studs	C (lipped channel section) 100x50x10x1.00mm (web depth x flange size x lip size x thickness)
	Tracks	U (unlipped channel section) 100x40x1.00mm (web depth x flange size x thickness)
Floor members	Joists	C 260x40x10x1.50mm
	tracks	U 260x40x1.00mm
	Bearing stiffeners	C 100x50x10x1.00mm
Sheathings		
Wall sheathings	Interior	1200x2500x12.5mm (width x height x thickness)
	Exterior	1250x2500x9.0mm Type 3 OSB
Floor sheathing		1250x2500x18.0mm Type 3 OSB
Frame-to-foundation connections		
Hold-down connector		Purposely-designed welded steel hold-down
Hold-down anchors		HIT-RE 500 with HIS-N(8.8) M20 adhesive-bonded anchors
Shear anchors		HST M8 mechanical anchors spaced at 100mm
Steel-to-steel connections		
CFS members		4.2x13mm (diameter x lenght) modified truss head self drilling screws
CFS members-to-hold down connector		6mm diameter bolts
Steel-to-Sheathing connections		
Walls	Interior	3.5x25mm bugle head self drilling screws spaced at 150mm at the perimeter and at 300mm in the field
	Exterior	4.2x25mm flat head self drilling screws spaced at 150mm at the perimeter and at 300mm in the field
Floor		4.2x32mm flat head self drilling screws spaced at 150mm for sheathing-to-track connections and at 250mm for sheathing-to-joist connections

The global behaviour of the sub-assemblage may be synthesized by means of the relationship between the unit shear resistance v=$(V_1 + V_2)/L_t$ and the mean lateral displacement $d=(d_1+d_2)/2$. The v-d response curve is shown in Figure 4. During the monotonic test different behaviours were identified:

- For a lateral displacement equal to 10mm, tilting of the screws in the oriented strand board (OSB) sheathing-to-frame connections started, while bearing of the gypsum wallboard (GWB) panels begun.
- For a lateral displacement equal to 36mm (maximum shear resistance), tilting of screws in the OSB connections, as well as bearing in the GWB panels, were evident (see Figures 5 a and c).
- For a lateral displacement equal to 80mm, in both the OSB and GWB-to-frame connections screw heads initiated to pull through the sheathings and when the lateral displacement was equal to 130mm the screw heads completely pulled through the sheathings (see Figures 5 b and d). As a consequence, the sheathings were completely unzipped along the panel edges.

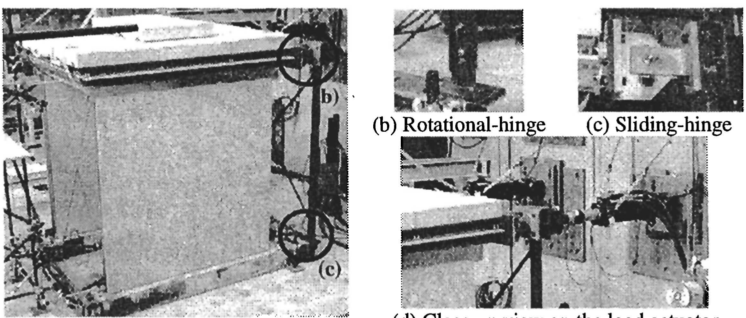
For all displacement levels, the wall framing deformed into a parallelogram and the sheathings had rigid body rotation (Fig. 5e).



(a) Walls (b) Floor

actuator potentiometer inclinometer

Figure 2. Instrument arrangements.



(a) Global 3D view (b) Rotational-hinge (c) Sliding-hinge (d) Close up view on the load actuator

Figure 3. Global view and some details of specimen and testing apparatus.

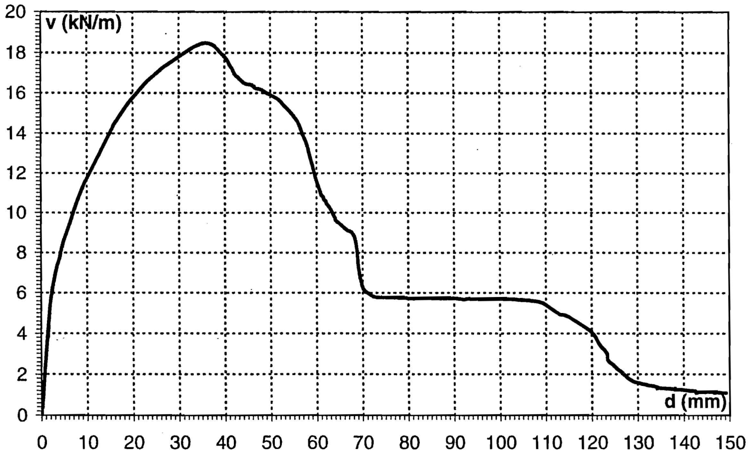


Figure 4. v vs. *d* curve for monotonic test.

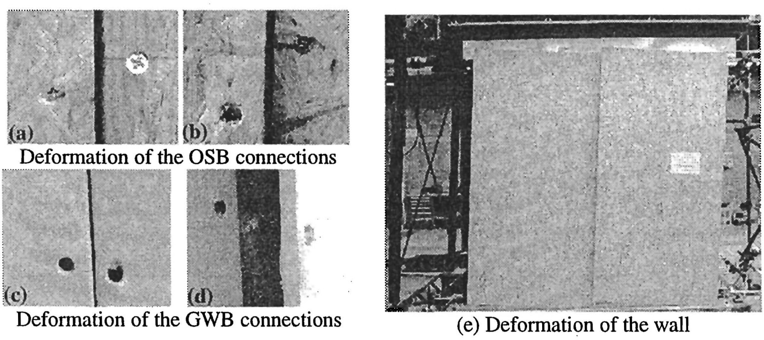


Figure 5. Specimen condition during the cyclic test.

The global cyclic response in terms of unit shear resistance (v) vs. mean displacement (*d*) curve is shown in Figure 6. In this Figure, v_{MAX+1} represents the maximum (positive) unit shear measured during the whole loading history; v_{MAX+3} represents the positive unit shear measured at the third cycle of displacement corresponding to v_{MAX+1}; v_{MAX-1} and v_{MAX-3} are the analogous quantities measured in the opposite direction of loading.

During the cyclic test, for lateral displacements less than the ones corresponding to the maximum shear resistance, the behaviour of OSB sheathing-to-frame connections resulted from a combination of the tilting of the screws and the screw heads pulling through the OSB sheathings. The response of the GWB sheathing-to-frame connections was characterized by a combination of the

bearing of the GWB panels and the screws' heads pulling through the GWB panels.
For lateral displacements larger that one corresponding to the maximum shear resistance, heads of the end screws completely pulled through the sheathings in the upper half of the walls or, in some cases, the screws caused the rupture of the sheathing edges. For these displacement levels the deformation of the wall framing still had the shape of a parallelogram, while due to the rupture of sheathing-to-frame connections, the rotation of the sheathings was limited.
Strength degradation after the achievement of the peak strength was more pronounced in the cyclic loading test, with respect to the monotonic case. This is well evidenced by the comparison between the unstable part of response in the monotonic and cyclic regimes of loading (Fig. 6).
More details about these experimental results can be found in Landolfo et al. (2004).

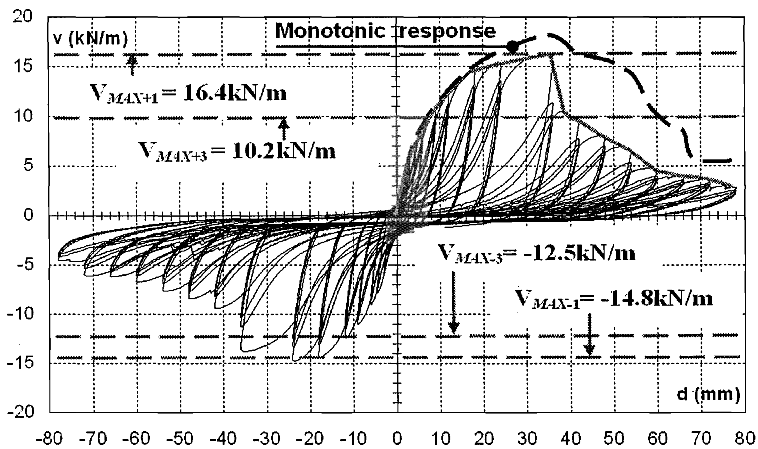


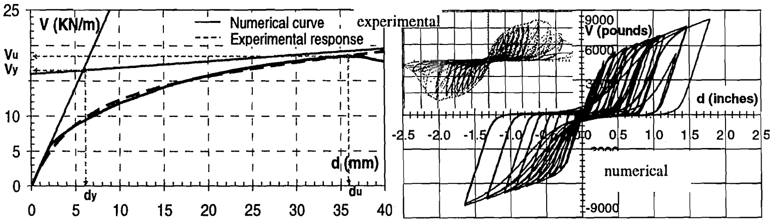
Figure 6. v vs. *d* curve for cyclic test.

Numerical results

An experiment can provide only information on capacities, but, because of the strong interrelation between capacity and demand, due consideration must be given to seismic demand issues (Krawinkler 1996).
A fully nonlinear with pinching mathematical model of the hysteretic response has been adopted in this numerical study (Della Corte et al. 2000). The model has been calibrated using both the experimental results obtained in the current study and existing experimental cyclic tests (Serrette et al. 1996a, b; Serrette et

al. 1997; COLA–UCI 2001). Figure 7a illustrates the comparison between the experimental monotonic lateral load-displacement relationship and the numerical model simulation, while Figure 7b illustrates the adopted hysteretic model simulating one of the selected cyclic tests. Only the stable part of the response has been simulated.

Analyses were carried out using 26 far field records from Central Italy and adopting the incremental dynamic analysis (IDA) procedure (FEMA 350 2001). Figure 8 shows the obtained IDA curves. In this Figure the elastic (5% damped) spectral acceleration ($S_{a,e}$) is plotted versus the maximum required inter-story drift angle (d/h). The ultimate value of the inter-story drift angle (d_u/h) and the design value of the elastic spectral acceleration ($S_{a,e,d}$) are also reported in Figure 8.



(a) Simulating of the monotonic response (b) Simulating of a selected cyclic test

Figure 7. Model calibration

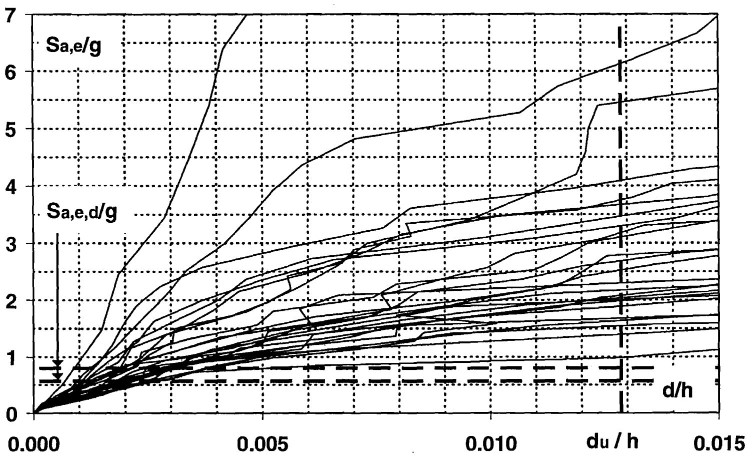


Figure 8. IDA curves.

The following parameters are introduced in the following discussion:
- $S_{a,e,d}$: design value (10% probability of being exceeded in 50 years) of the elastic (5% damped) spectral acceleration;
- $S_{a,e,u}$: the elastic (5% damped) spectral acceleration corresponding to the ultimate value of the inter-story drift angle;

Besides, the following displacement-controlled limit states are defined:
- Damage-limiting limit state: it is the attainment of a limiting value of the inter-story drift angle beyond which plastic deformations are so large to produce appreciable damage to the structure. This limiting value of the inter-story drift angle is set equal to 0.0035, on an empirical basis.
- Collapse limit state: it is the attainment of a limiting value of the inter-story drift angle beyond which the residual safety of the structure against collapse is assumed negligible. This limiting inter-story drift angle is set equal to 0.013, which corresponds to the attainment of the maximum lateral strength on the static pushover curve.

On the basis of the obtained numerical data, the following comments can be made:
- Under the design earthquake intensity ($S_{a,e,d}$), damage is negligible, the maximum inter-story drift angle demand being 0.33%<0.35%.
- The average $S_{a,e,u}/S_{a,e,d}$ ratio is relatively large (($S_{a,e,u}/S_{a,e,d})_{\mathrm{av}}$=5.4), but dispersion of data is also large. The minimum value of the ratio is 1.7, which results acceptable according to modern code suggestions for very rare earthquakes (prEN 1998-1 2003, ATC-40 1996).

The obtained numerical results have also been used for selecting an appropriate loading history for cyclic testing. It has been based on the following seismic demand parameters:
- Maximum normalised displacement (ductility): $\mu_{max}=(d/d_y)_{max}$
- Number of plastic deformation excursions: N_p
- Sum of normalised plastic deformation ranges: $\overline{\mu}=\Sigma\Delta d_{p,i}/d_y$
- Average over maximum plastic deformation range ratio: $\rho_p=|\Sigma\Delta d_p|_{\mathrm{av}}/|\Sigma\Delta d_p|_{\mathrm{max}}$

For a given value of μ_{max}, the parameters N_p and $\overline{\mu}$ give a measure of the cumulative damage effects produced by the earthquake. The value of ρ_p gives, instead, information about the distribution of the plastic deformation ranges.

Starting from the monotonic pushover physical test carried out in this research, the maximum normalised displacement capacity has been fixed equal to 6 ($\mu_{max,c}$=6). Then, peak ground accelerations of considered natural records have been artificially scaled up to values corresponding to the attainment of a

ductility demand equal to 6. Results of these analyses are summarised in Figures 9a through 9d. Figures 9b and 9c show the required number of inelastic excursions (N_p) and the required sum of normalised plastic deformation ranges ($\overline{\mu}$). Figure 9d illustrates instead the computed values of the ratio (ρ_p) between the average and the maximum plastic deformation ranges.
More details on the seismic demand numerical study can be found in Della Corte et al. (2004).

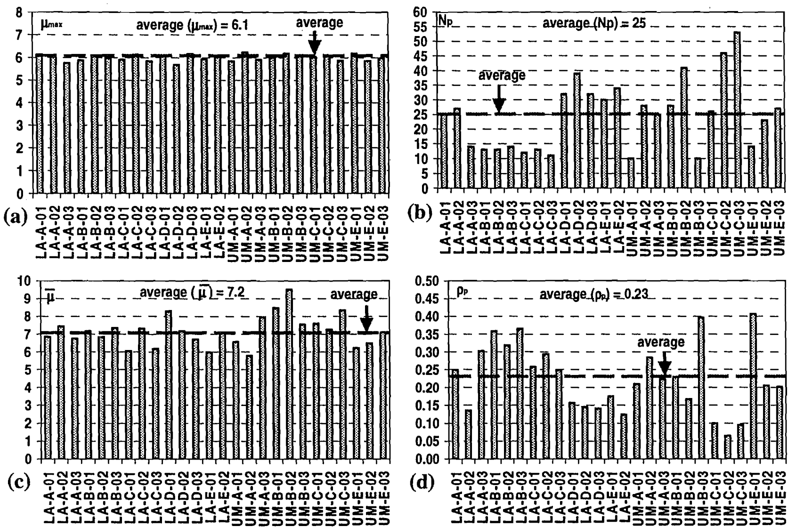


Figure 9. Some characteristics of the inelastic deformation demand.

Average values of N_p, $\overline{\mu}$ and ρ_p have been adopted as the basis for deriving the cyclic loading history to be applied in the physical test. Values of N_p, $\overline{\mu}$ and ρ_p characterizing the first part of the loading history ($\mu<\mu_{max,c}$) have been derived on a trial-and-error basis, by searching the best possible matching of the average values derived from the numerical analysis of demand, under the constraint to have a loading protocol similar to that suggested by ATC-24 (1992). The remaining part of the loading history ($\mu>\mu_{max,c}$) has been defined strictly following the ATC-24 (1992) suggestion. This subdivision between the ranges $\mu<\mu_{max,c}$ and $\mu>\mu_{max,c}$ derives from the limitations of the numerical model, which is able to simulate only the stable part of the physical response.

Strength reduction factors

Current standard design philosophy is based on the strength-reduction factor ("behaviour factor" (q) using the European terminology or "response modification factor" (R), according to the USA terminology) taking into account the structural ductility. In particular, the strength-reduction factor is usually subdivided into ductility factor (q_μ or R_μ) and over-strength factor (q_Ω or R_Ω), and expressed as follows:

$$q = q_\mu q_\Omega \qquad \text{or} \qquad R = R_\mu R_\Omega \tag{1}$$

In Zhao & Rogers (2002) a method for the evaluation of the response from quasi-static reversed cyclic test results is presented. Following this method, the experimental backbone curve based on the highest strength hysteretic response is considered. In particular, the backbone curve is schematised through an equivalent bilinear elasto-plastic curve in which the elastic response has the same initial stiffness of the envelope curve and the plateau (plastic response) intersects the peak load in the backbone curve. The ductility factor (R_μ) is then evaluated through the following equations:

$$R_\mu = \mu \ \text{ for } \ T \geq 0.5\text{ s} \quad \text{or} \quad R_\mu = \sqrt{2\mu - 1} \ \text{ for } \ T < 0.5\text{ s} \tag{2}$$

where T is the fundamental period of vibration and the ductility demand factor (μ) can be determined as $\mu=\Delta_{max}/\Delta_y$, in which Δ_y and Δ_{max} are evaluated from the bilinear idealization (Δ_y is the displacement corresponding to the intersection of the two segments and Δ_{max} is the displacement corresponding to the peak load). These relationships are well known expressions of the equal displacement and equal energy approximations, respectively. The over-strength factor (R_Ω) is defined as the ratio of nominal (F_n) and first significant yield (F_s) strengths:

$$R_\Omega = F_n \,/\, F_s \tag{3}$$

where the nominal strength (F_n) is obtained considering the stable loops (decreasing of strength in successive cycles of a given displacement amplitude less than 5%) and the yield strength (F_s) is defined as the value of lateral load for which the ideal elastic response deviates significantly from the backbone curve. In particular, in Zhao & Rogers (2002) an over-strength factor (R_Ω) equal to 1.82 is assumed, based on the design provisions of Uniform Building Code (ICBO, 1997).

Based on the cyclic response of the specimen tested in this research and considering the Zhao & Rogers' methodology, ductility factors (R_μ) of 2.66 and 2.46 are obtained, considering the positive and negative cycles, respectively. It is

interesting to observe that these results are very close to those obtained by Zhao & Rogers (2002) with reference to walls having similar characteristics (specimens AISI-OSB1, -OSB2, -E1, -E2 and Group 14). In fact, the average value of R_μ for the tests examined by Zhao and Rogers is 2.6, which is very close to the average value obtained using the experimental test results presented in this paper ((2.66+2.46)/2=2.56).
As far as the evaluation of the over-strength factor is concerned, from the cyclic test results obtained in the current research, over-strength factors (R_Ω) of 2.69 and 2.45 are obtained, considering the positive and negative cycles, respectively. Consequently, the resulting average R-value, according to formula (1), is R=2.56 × 2.57≅6.6. This value appears to be too much large if compared with the ratios $S_{a,e,u}/S_{a,e,d}$ obtained by the dynamic inelastic analysis results previously presented (minimum value of $S_{a,e,u}/S_{a,e,d}$ equal to 1.7). Then, this (static) methodology seems to be unconservative.

Conclusions

Some results of a research program aiming to study the seismic performance of cold-formed steel stud shear walls and being carried out at the University of Naples "Federico II" have been presented and discussed throughout the current paper.
The obtained results allow the following conclusions to be drawn:

- All the components of this structural system can be designed according to capacity design principles, imposing collapse in the shear walls' sheathing-to-frame connections (most ductile collapse mechanism), without significant increase of the cost.
- In the monotonic test, the collapse mechanism was invariant during the increasing lateral displacement, whilst in the cyclic test some modifications (more brittle collapse mechanism) occurred after that the peak lateral load was achieved. These modifications produced strength degradation, after attainment of the peak load, in the cyclic test stronger than in the monotonic test.
- The horizontal diaphragm can adequately transfer the horizontal loads to the vertical shear walls, without any appreciable damage.
- The maximum inter-story drift angle demand, under the whole set of considered acceleration records and for the design value of the spectral acceleration, was equal to 0.33%, which is smaller than the damage-limiting value (0.35%) coming from the experimental tests carried out.
- The minimum value of the ratio between the ultimate elastic spectral acceleration and its design value was equal to 1.7. This value satisfy the

minimum requirement of several different seismic codes (e.g. Eurocode 8, ATC-40) for very rare earthquakes.

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