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Seismic analysis of sheathing-braced cold-formed steel structures

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ABSTRACT

The seismic behavior of sheathed cold-formed steel (SCFS) structures is characterized by the lateral response of shear walls. Basically, if cold-formed steel (CFS) structures are designed according to the "sheathing-design" methodology, then the seismic behavior of shear walls is strongly influenced by the sheathing-to-frame connections response, characterized by a remarkable nonlinear response and a strong pinching of hysteresis loops. In this paper the results of an extensive parametric non linear dynamic analysis, carried out on one story buildings by means of incremental dynamic analysis (IDA), using an ad hoc model of the hysteresis response of SCFS shear walls, are presented. An extended number of wall configurations has been considered investigating several parameters such as sheathing panel typology, wall geometry, external screw spacing, seismic weight and soil type. Based on IDA results, three behavior factors have been defined, which take into account overstrength, ductility and both overstrength and ductility, respectively. Finally, a design nomograph for the seismic design of single-storey SCFS frame structures developed on the basis of non-linear dynamic analysis results is presented. This last aims to complete a proposal of a design methodology, already presented by the author in the last years.

1. Introduction

The seismic behavior of cold-formed steel (CFS) structures sheathed with panels is influenced by the response of shear walls, which are characterized by a highly non-linear structural response. In order to assess the seismic performance of sheathed cold-formed steel (SCFS) structures, several experimental and/or numerical research programs have been carried out on different wall configurations. On the basis of results provided by tests and analysis, different seismic design parameters, as behavior factor (q, using the European terminology) or seismic force modification factor (R, according to the USA terminology) [10,11,2] and interstory drift limits [16,2,6] have been identified, and interesting seismic design methods have been proposed [16]. In addition, other studies, focused on the dynamic characteristics of this type of structures [16,12,6], have identified typical values of vibration period and damping ratio.

Monotonic and cyclic tests on walls sheathed with trapezoidal steel sheets or oriented strand board (OSB) panels with or without openings were carried out by Fülöp and Dubina [10]. On the basis of this experimental result they developed a numerical model and performed incremental dynamic analysis. Therefore, the authors define three different behavior factors [11]: q_1, corresponding to the overstrength, q_2, corresponding to the ductility and $q_3 = q_1 \cdot q_2$, which takes into account both effects. The behavior factor q_1 ranges from 2.2 to 2.6, q_2 ranges from 1.4 to 1.6 and q_3 is in the range 3.6 through 3.7. In particular, in the case of walls sheathed with OSB panels and without openings the authors found the following average behavior factors: $q_1 = 2.7$, $q_2 = 1.4$ and $q_3 = 3.7$. Boudreault et al. [2] evaluated the seismic force modification factors (R) due to the ductility (R_d) and overstrength (R_o) as defined by the National Building Code of Canada NBCC [19]. The proposed values $R_d = 2.5$, $R_o = 1.7$, and $R = R_o \cdot R_d = 4.3$ were obtained by analyzing the results of monotonic and cyclic tests on different configurations of walls sheathed with plywood and OSB panels [1,3,21]; Rokas, 2006). The values obtained on experimental basis were verified through the results of non-linear dynamic analysis (time-history) carried out on two representative buildings [2]. Values of seismic force reduction factor (R-factor) for CFS buildings are also provided in a number of codes, as the Uniform Building Code [23], the International Building Code [15] and the FEMA 450 [7]. The value of R-factor provided by the UBC is 5.5, for buildings with less than three floors and walls sheathed with wood-based panels, and 4.5 for other types of walls. The IBC provides a value of reduction factor R (response modification coefficient) equal to 6.5 for shear walls sheathed with wood panels or steel sheets, and 2 for other types of sheathings. Finally, a R-factor equal to 6.5 is given in the FEMA 450 for walls with shear panels.

A design methodology based on direct evaluation of seismic capacity in terms of displacements (interstory drift) is proposed

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by Kawai et al. [16]. To support this methodology, the authors performed dynamic analysis on a typical SCFS house produced by Kaza Club and approved by the Building Center of Japan, in which the walls are sheathed with plywood and plasterboard panels. In the analysis, the authors assumed a damping ratio equal to 6%. This value was selected in the range of experimental results in such a way to account the presence of all structural and non-structural components including finishing. For the case study house, the walls had a total length significantly larger than that obtained on the basis of proposed methodology. As a result, the interstory drift obtained from dynamic analysis was 0.33% (1/300 rad), which highlights a good response under severe earthquakes. Obviously, the interstory drift increases when the total walls length decreases and in the case of walls with a total length equal to that obtained by the proposed method, the displacement demand became equal to 1.67% (1/60 rad), which was less than 2%, which represents the limit (target) value suggested by Kawai et al. [16] for severe earthquakes in order to ensure the building safety.

As for as the interstory drift limits (d/h) definition is concerned, based on test results on steel sheet-to-steel sheet screw connections, for CFS walls sheathed with trapezoidal CFS sheets Dubina [6] suggested different interstory drift limits, corresponding to different performance levels: 0.3%, 1.5% and 2.5% for fully operational, partially operational and ultimate limit states, respectively. Boudreault et al. [2] performed numerical dynamic analysis on a two-story house and a three-story commercial building, both of them having lateral resisting systems made of CFS walls sheathed with OSB panels. These constructions were designed by considering the values of shear wall resistance and force modification factor obtained on the basis of experimental results founded by the authors. The analysis results showed that the maximum interstory drift under earthquake ground motion records scaled to 2% in 50 years probability of exceedence were obtained for the first floor and were equal to 1.40% and 1.62% for the house and commercial building, respectively. These values did not exceed the limit values (1.91–2.23%) defined on the basis of test results [1] and the limit value (2.5%) given in the National building code of Canada [19].

Dynamic vibration tests on a real two-stories CFS framed house sheathed with plywood panels were carried out by Kawai et al. [16]. The tests were performed in different construction stages in order to take into account the contribution of different constructional components, including the finishing. Three different tests (microtremor test, sweep test and free vibration tests) were carried out and three levels of exiting action were used. The obtained fundamental periods of vibration range from 0.128 s to 0.156 s, while damping ratio values range from 2.5% to 7.0%. The authors noticed that as progress of construction increases the period of vibration decreases and the damping increases, while an increasing of the exiting action magnitude produces an increasing of both period of vibration and damping. Single walls and an one-story house prototype were tested on a shacking table by Gad et al. [12]. The tests on the prototype were carried out under different house configurations: (1) X-bracing; (2) cement-based sheathing panels and interior finishing; (3) X-bracing, sheathing panels and interior finishing; (4) X-bracing, sheathing panels, interior finishing and brick exterior finishing. The authors concluded that the fundamental period of vibration (0.225–0.260 s) decreases and the damping ratio (4.2–10.3%) increases with the progress of construction, while the presence of the brick exterior finishing does not produce significant variations in structural dynamic response. The dynamic response of a real two-stories with attic CFS framed house sheathed with OSB panels in three different construction stages were evaluated by Dubina [6] through on-site tests. Fundamental periods of vibration (0.101–0.546 s) and damping ratios (1.2–4.2%) measured for different construction stages were compared with those obtained from the results of numerical analysis. The comparison showed a significant damping effect of the finishing, which contribute positively to the seismic dissipation capacity.

This paper presents the results of a very extensive parametric non-linear dynamic analysis performed on SCFS structural systems (Sections 2 and 3). On the basis of analysis results, an evaluation of behavior factors has been carried out (Section 4) and a seismic design procedure based on a specific nomograph is proposed (Section 5). Finally, the seismic design of a one-story with attic house is illustrated as study case (Section 6).

2. Monotonic response evaluation

2.1. Model for monotonic response prediction

In this paper the analytical-numerical model proposed by the authors [18], that allows the evaluation of the monotonic lateral response of a SCFS shear wall, has been used. The method is based on the most common assumptions made for other available approaches, such as full anchorage between frame and foundation or other stories and sheathing-to-frame interaction characterized by a "rigid body" behavior. The model allows the evaluation of each wall deflection contribution, from which the total wall deflection can be obtained by adding the deformation due to bending (d_F,), sheathing fasteners ($d_{F–S}$), sheathing panels (d_S), and anchorages ($d_{F–F}$): $d = d_F + d_{F–S} + d_S + d_{F–F}$.

The model has been calibrated on the basis of monotonic tests on full scale specimens [17] carried out on CFS shear walls sheathed with OSB and gypsum board (GWB) and tests performed on sheathing connections [8] that are nominally identical to that used for the walls. Further details about hypothesis and applicability of the model can be found in Fiorino et al. [9].

2.2. Parametric study

In order to study the behavior of different wall configurations, the model has been applied to simulate the monotonic response of 72 different wall configurations. All the walls are made of 100 × 50 × 10 mm (outside-to-outside web depth × outside-to-outside flange size × outside-to-outside lip size) lipped channel studs, spaced at 600 mm and sheathed with GWB panels on both sides (G + G) or GWB on one side and OSB panels on the other side (G + O). The thicknesses of GWB and OSB panels are 12.5 mm and 9.0 mm, respectively. For connecting sheathing panels and frame 4.2 × 25 mm (diameter × length) flat head self drilling screws have been considered for OSB and 3.5 × 25 mm bugle head for GWB. The fasteners have spacing equal to 300 mm in the field, while different external screw spacing have been investigated. Hold down devices type Simpson Strong-Tie [22] and chemical anchorage (type HIT-RE 500 with HIS-N 8.8 by Hilti [13]) have been considered at each end of the wall, while mechanical shear anchors (type HST M8 by Hilti) have been hypothesized. Sheathing panel typology, wall geometry (height h and length l) and external screw spacing (s) have been varied as summarized in Table 1. For each wall configuration obtained by combining the parameters given in Table 1, the stud thickness (t_F) and hold-down device typology have been selected in such a way to promote the sheathing fasteners collapse, as shown in Table 2. The monotonic response of the walls has been

Table 1
Variables assumed in the parametric study.

Sheathing panel typology	GWB + GWB (G + G), GWB + OSB (G + O)
Wall height (h) (mm)	2400, 2700, 3000
Wall length (l) (mm)	1200, 2400, 9600
External screw spacing (s) (mm)	50, 75, 100, 150

Table 2
Selected stud thicknesses and hold down devices.

Wall typology (s and h in mm)	Hold down	Stud (t_F) (mm)	K_a (kN/mm)
G + G; s = 75, 100, 150 G + O; s = 150	S/HD 8B	1.0	28
G + O; s = 75; h = 2400, 2700 G + O; s = 100; h = 2400	S/HD 8B	1.5	28
G + G; s = 50 G + O; s = 100; h = 2700, 3000	S/HD 10B	1.0	27
G + O; s = 50; h = 2400	S/HD 10B	1.5	27
G + O; s = 50; h = 2700, 3000	S/HD 15B	1.5	31
G + O; s = 75; h = 3000	S/HD 8S	1.5	29

obtained by considering a shear modulus (G) of 1400 MPa for OSB panels and 750 MPa for GWB panels. In addition, a Young modulus (E) for the steel equal to 200,000 MPa has been adopted, while the assumed values of hold down device axial stiffness (K_a) are given in Table 2. All results obtained through the application of the model to 72 wall configurations are shown in Fig. 1 in terms of horizontal force (H) vs. lateral deflection (d) curves.

3. Evaluation of cyclic response

3.1. Cycle response modeling

In this study an upgraded version of the cyclic model proposed in Della Corte et al. [5], which takes into account of strength degradation, has been used. According to this approach, in order to model the cycle lateral response of shear walls in terms of horizontal force (H) vs. lateral deflection (d), the definition of three limit curves, together with the definition of transition and strength degradation laws are needed (Fig. 2). The first curve (monotonic curve) represents the monotonic response, while the other curves represent the upper bound cyclic (UBC) curve and the lower bound cyclic (LBC) curve to all possible H–d values obtained in the cyclic response. For each curve a 6 independent parameters Richard–Abbott type law is assumed, with the following parameters for the monotonic curve: K_0 is the initial stiffness; H_0 is the intersection between the hardening line and the $d = 0$ axis; K_h is the slope of the hardening line; n is the shape parameter regulating the sharpness of transition from the elastic to the plastic behavior; d_p is the peak deflection; K_d is the post-peak stiffness; d_u is the conventional ultimate deflection corresponding to a load equal to 0.80 time the peak horizontal force (H_p) on the post-peak branch of response curve (dependent parameter). The same parameters are needed for the definition of UBC curve (K_{0U}, H_{0U}, K_{hU}, n_U, d_{pU}, K_{dU}, d_{uU}) and LBC curve (K_{0L}, H_{0L}, K_{hL}, n_L, d_{pL}, K_{dL}, d_{uL}).

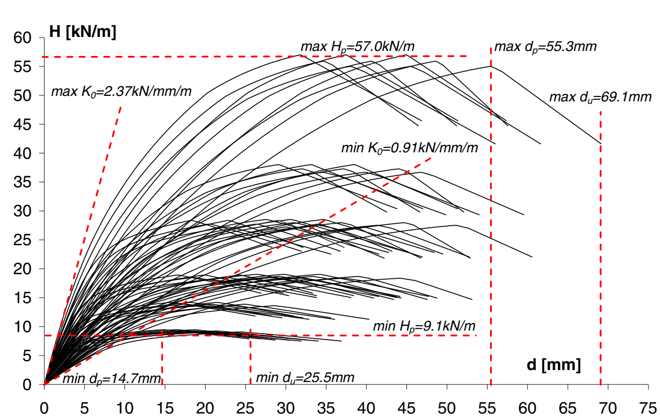

Fig. 1. H–d response curve for all wall configurations.

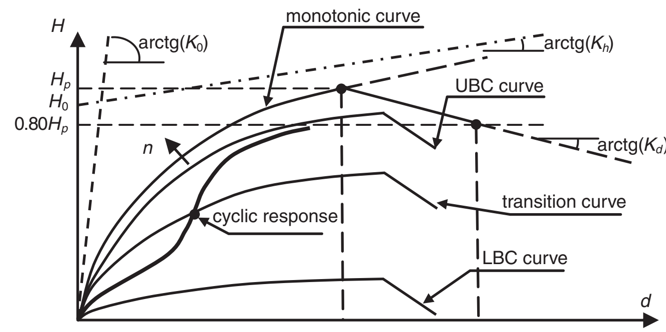

Fig. 2. Model of the cycle lateral response of SCFS shear walls.

The model assumes that the UBC and LBC curves can be derived from the monotonic response curve on the basis of these assumptions:

$$X_U = U_X \cdot X \tag{1}$$

$$X_L = L_X \cdot X \tag{2}$$

where X, X_U, and X_L represent the generic independent parameters defining the monotonic, UBC and LBC curve, respectively (i.e. $X = K_0$, $X_U = K_{0U}$ and $X_L = K_{0L}$); U_X and L_X are the relevant constant (i.e. $U_X = U_{K0}$ and $L_X = L_{K0}$, thus $K_{0U} = U_{K0}\ K_0$ and $K_{0L} = L_{K0}\ K_0$).

Other assumption of the proposed methodology is that the generic point of a cyclic loading branch belongs to a Richard–Abbott type curve (transition curve), whose the relevant independent parameters K_{0t}, H_{0t}, K_{ht}, n_{pt}, d_{pt}, K_{dt} are given by the linear convex combination of analogous parameters of the LBC and UBC curves:

$$X_t = X_L + (X_U - X_L)t \tag{3}$$

where X_t represents the generic independent parameter defining the transition curve (i.e. $X_t = K_{0t}$, thus $K_{0t} = K_{0L} + (K_{0U} - K_{0L})t$), and t defines the transition law from the LBC to the UBC curve:

$$t = \left[\frac{[d/(\lambda(|d_0| + d_{\max}))]}{[d/(\lambda(|d_0| + d_{\max}))]^{t_1} + 1}\right]^{t_2} \quad \text{with } 0 \leqslant t \leqslant 1 \tag{4}$$

in which d_0 is the deflection corresponding to the initial point of the current excursion and d_{max} is the maximum deflection value reached in all previous loading history in the direction to be described. Therefore, t can be defined by assigning t_1, t_2 and λ.

The strength degradation is taken into account following the methodology proposed by Park and Ang [20]:

$$H_{0,red} = H_0(1 - D_F) \tag{5}$$

where $H_{0,red}$ is the reduced value of H_0 in the current cycle excursion and D_F represent the parameter accounting for the strength reduction.

As results, the proposed model depends on 22 independent parameters (K_0, H_0, K_h, n, d_p, K_d, U_{K0}, U_{H0}, U_{Kh}, U_n, U_{dp}, U_{Kd}, L_{K0}, L_{H0}, L_{Kh}, L_n, L_{dp}, L_{Kd}, t_1, t_2, λ, D_F).

The calibration of the parameters describing the whole cyclic response has been carried out on the basis of available results of experimental monotonic and cyclic full scale tests [17]. The values of the monotonic curve parameters (K_0, H_0, K_h, n, d_p, K_d) have been obtained in such a way that this curve matches the experimental monotonic response curve, while the UBC curve parameters (U_{K0}, U_{H0}, U_{Kh}, U_n, U_{dp}, U_{Kd}) have been defined in order to match the envelope of the experimental cyclic response curve. Finally, the values of the LBC curve (L_{K0}, L_{H0}, L_{Kh}, L_n, L_{dp}, L_{Kd}), transition (t_1, t_2, and λ) and strength degradation (D_F) parameters have been defined in such a way that the numerical cyclic response would be as much as possible similar to the experimental cyclic response in terms of

H–d curve and dissipated energy. The parameters values obtained as result of calibration are given in Table 3, while the comparison in terms of response curve and dissipated energy are shown in Figs. 3 and 4, respectively.

3.2. Case study

One-story buildings have been considered as case studies. They refer to a stick built constructions in which both floors and walls are realized with CFS framing sheathed with structural panels. In particular, in order to obtain a large range of solutions, a schematic plan has been considered with wall length (L) variable between 3 and 7 m (Fig. 5) and lengths of full height (resisting) wall segment (l) in the range $l = 0.4{\cdot}L$ through $l = 0.7{\cdot}L$. Unit weights ranging from 0.4 to 1.5 kN/m^2 and from 0.3 to 1.2 kN/m^2 have been considered for floors and walls, respectively (Table 4).

Moreover, the building has been considered without and with attic. In the first case, a variable live load of 2.0 kN/m^2 has been considered and, in the latter case, a snow variable load ranging from 0.60 to 1.20 kN/m^2 has been added. The seismic weights have been defined according to the following relationship:

$$\sum G_{Ki} + \sum_j (0.3 \cdot Q_{Kj}) \tag{6}$$

where G_{ki} are the characteristic value of permanent actions and Q_{ki} are the variable loads. With this conditions, a seismic weight for unit wall length ranging between 2 and 38 kN/m has been obtained. Therefore, 7 seismic weights per unit wall length have been considered (10, 15, 20, 25, 30, 35 and 40 kN/m) and have been applied to the 72 wall configurations defined previously in the parametric study in such a way to obtain a total number of $7 \times 72 = 504$ cases.

3.3. Ground motion

In order to develop a non-linear dynamic seismic analysis, the seismic inputs have been selected in such a way that they could cover all the soil typologies classified by Eurocode 8 [4]. In particular, Eurocode 8 provides five different soil types A, B, C, D, and E, but in this study only three spectra are adopted grouping the soil types B, C and E under one spectrum type. Therefore, 21 earthquake records have been selected from the ESD (European Strong-motion Database, www.isesd.cv.ic.ac.uk). For each soil type 7 accelerograms have been considered so that the shape of the average elastic response spectrum is close as much as possible to the shape of the corresponding Eurocode 8 elastic acceleration spectrum [14], as shown in Fig. 6a, b and c. As results, a total number of $504 \times 21 = 10584$ cases have been obtained. The selected earthquakes include records from different European and Mediterranean regions. For these earthquakes the Richmond magnitude range from 5.8 to 7.6. In order to match the design spectra the natural accelerograms have been scaled by the peak ground acceleration (PGA).

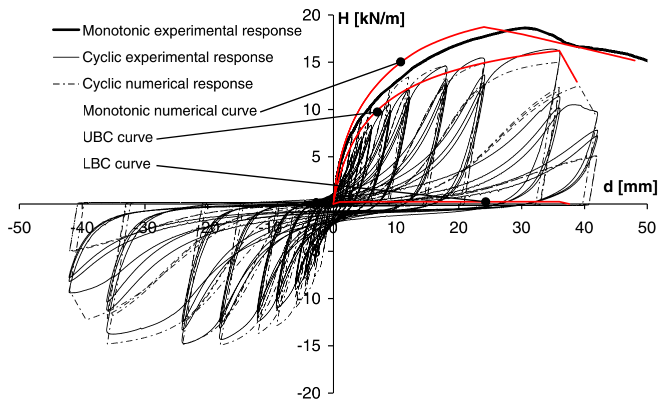

Fig. 3. Calibration results in terms of force vs. displacement response.

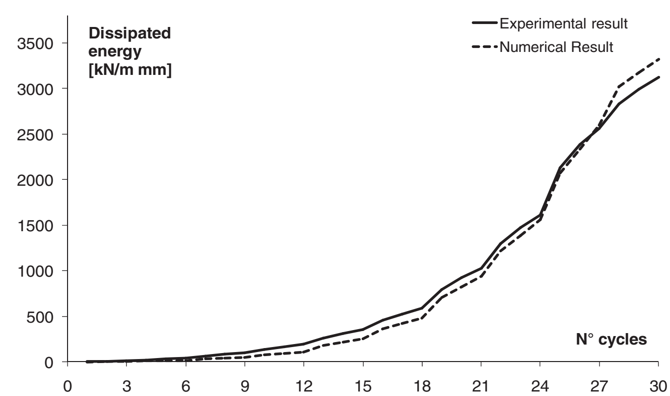

Fig. 4. Calibration results in terms of dissipated energy.

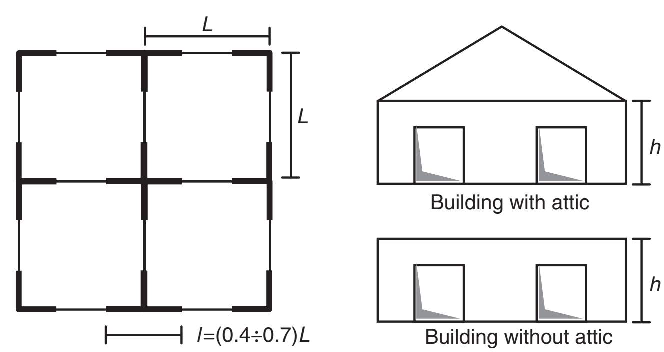

Fig. 5. Case study: a schematic stick built construction.

3.4. Wall response modeling

Each wall defined previously in the parametric study has been schematized as single degree of freedom structure, in which the hysteretic behavior under horizontal loads is described by the model presented in Section 3.1. In particular, for the generic wall

Table 3
Parameters defining the cyclic response, as result of calibration.

Monotonic curve parameters	$K_0 = 6.05$ kN/mm/m	$H_0 = 24$ kN/m	$K_h = 0.03$	$n = 0.78$	$d_p = 24$ mm	$K_d = -0.15$
UBC curve parameters	$U_{K0} = 0.67$	$U_{H0} = 0.88$	$U_{Kh} = 0.0$	$U_n = 1.0$	$U_{dp} = 1.5$,	$U_{Kd} = 7.0$
LBC curve parameters	$L_{K0} = 0.67$	$L_{H0} = 0.01$	$L_{Kh} = 0.0$	$L_n = 1.0$	$L_{dp} = 1.5$	$L_{Kd} = 1.0$
Transition parameters	$t_1 = 12$	$t_2 = 0.8$,	$\lambda = 0.9$			
Strength degradation parameter	$D_F = 0.10$					

Table 4
Unit weights.

Floor and roof	
Steel members	0.08–0.25 kN/m^2
OSB panels	0.10–0.15 kN/m^2
GWB panels	0.00–0.10 kN/m^2
Insulation	0.02–0.30 kN/m^2
Floor finishing	0.10–0.40 kN/m^2
False ceiling	0.10–0.30 kN/m^2
Total	0.40–1.50 kN/m^2
Wall	
Steel members	0.03–0.08 kN/m^2
External board – OSB	0.05–0.20 kN/m^2
Internal board – GWB	0.10–0.20 kN/m^2
Insulation	0.02–0.30 kN/m^2
Internal finishing	0.00–0.20 kN/m^2
External finishing	0.10–0.30 kN/m^2
Total	0.30–1.20 kN/m^2

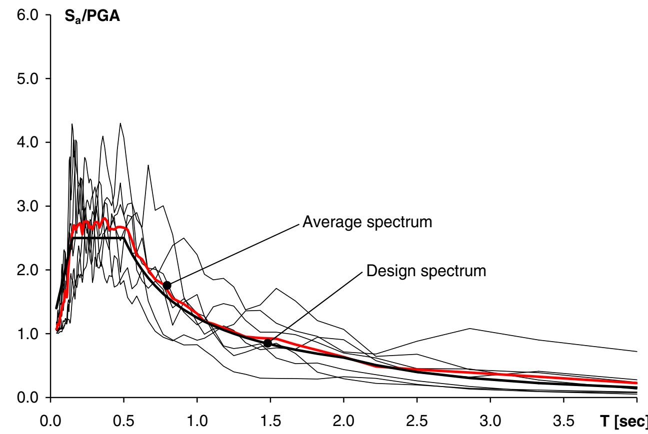

Fig. 6b. Eurocode 8 [4] elastic acceleration spectrum for soil type BCE.

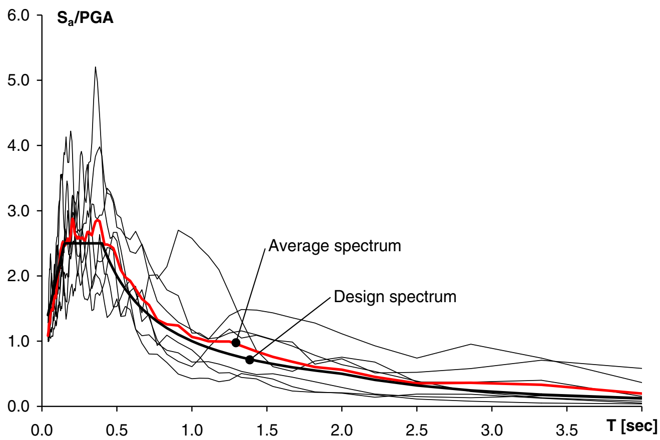

Fig. 6a. Eurocode 8 [4] elastic acceleration spectrum for soil type A.

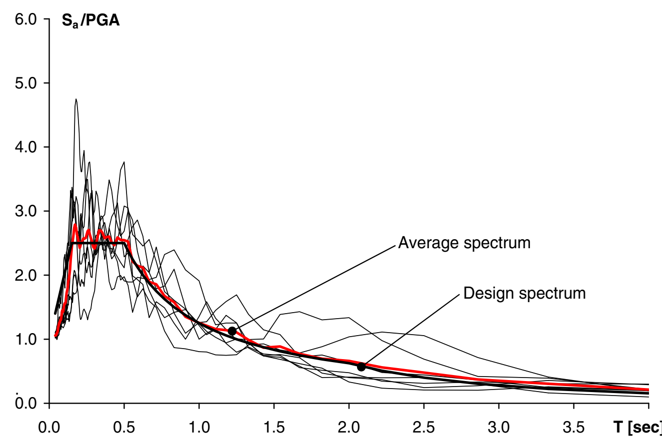

Fig. 6c. Eurocode 8 [4] elastic acceleration spectrum for soil type D.

configuration the parameters defining the cyclic response have been set in such a way that the monotonic curve parameters give a function matching the relevant monotonic curve obtained in the parametric study, while the other parameters (UBC curve, LBC curve transition and strength degradation parameters) have been set equal to the value given in Table 3. This assumption implicates that in all the wall configurations examined in the parametric study the cyclic load produces the same effects observed in the tests carried out by Landolfo et al. [17].

In order to account the second order effects, a vertical load equal to the 100% of the mass has been considered. The viscous damping ratio has been set equal to 5%, according to the experimental results obtained by Kawai et al. [16], Gad et al. [12] and Dubina [6]. The used procedure is the well known incremental dynamic analysis (IDA). Outputs of the analysis are the IDA curves, that are presented as adimentionalized elastic spectral acceleration ($S_{a,e}/g$, intensity measure of the seismic record) vs. maximum required interstory drift angle (d/h, performance parameter). In particular, for obtaining a single IDA curve, which represents the response for a specific wall condition (wall geometry and materials, seismic weight and soil type), each accellerogram has been scaled in the range from 0.05 to 1.95 by considering 50 values. Therefore, a total number of 10,584 × 50 = 529,200 single non-linear dynamic analysis has been performed. Fig. 7 shows typical IDA curves obtained in the present study by changing the accelerograms for a 2400 mm high and 1200 mm long wall, sheathed with GWB panels on both sides, having external screw spacing of 50 mm and a seismic weight of 15 kN/m.

4. Behavior factor evaluation

4.1. General assumptions

Based on the results of performed dynamic analyses, the non-linear capacity of SCFS shear walls, in terms of behavior factor, has been assessed. The numerical results have been interpreted by considering three different limit displacements on the generic response curve (Fig. 8): the peak (d_p) and ultimate (d_u) displacements, as defined in Section 3.1, and the yielding displacement of the idealized bilinear curve (d_y) created according to an equivalent energy elastic-plastic approach [20], following the recommendations given by Branston et al. [3].

For each IDA curve the seismic intensity measures $S_{a,y}$, $S_{a,p}$ and $S_{a,u}$ corresponding to the limit displacements d_y, d_p and d_u, respectively, have been evaluated and these spectral accelerations have been used to define three different behavior factors, as follows (Fig. 9):

$$q_1 = S_{a,p}/S_{a,y}, \quad q_2 = S_{a,u}/S_{a,p}, \quad q_3 = S_{a,u}/S_{a,y} \tag{7}$$

in which q_1 takes into account the overstrength, q_2 takes into account the ductility and $q_3 = q_1 \cdot q_2$ takes into account both overstrength and ductility.

In order to obtain an assessment of the behavior factors q_1, q_2 and q_3 on the basis of significant IDA results, only the IDA curves representing realistic design conditions have been selected. The selection has been performed by comparing the displacement demand (d_D) obtained for a given earthquake hazard level and different seismic intensity zones (i.e. peak ground acceleration on soil

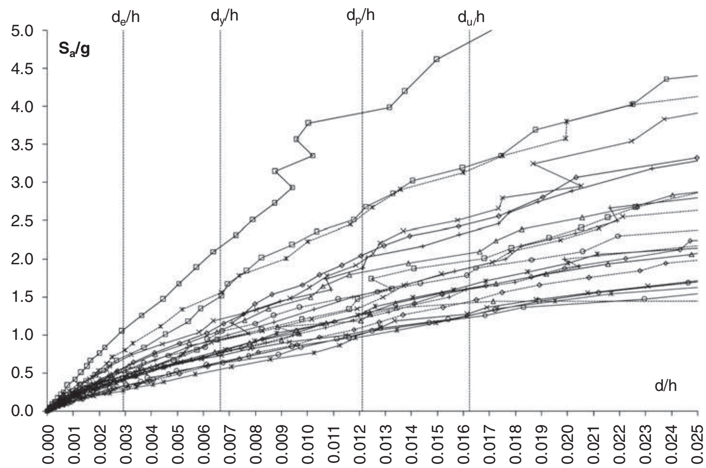

Fig. 7. Typical IDA curves.

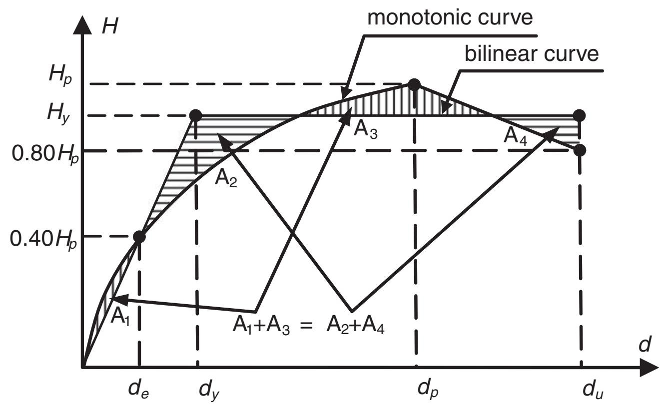

Fig. 8. Monotonic response curve of a SCFS shear wall: definition of the idealized bilinear curve.

type A (a_g) for a 10% probability of exceedance in 50 years equal to 0.15, 0.25 and 0.35 g for low, medium and high seismic intensity zone, respectively) with the relevant displacement capacity (d_C) (i.e. conventional ultimate displacement, $d_C = d_u$). As result of this comparison, a performance coefficient ($p = d_D/d_C$) has been defined and only the IDA curves for which p satisfies specified target conditions (i.e. $0.5 \leqslant p \leqslant 1.0$) have been chosen. In particular, the selection of IDA curves has been performed by adopting two different approaches: ''Classical'' and ''Multi-performance'', in which the difference consists on the required performance objectives, as discussed in following Sections.

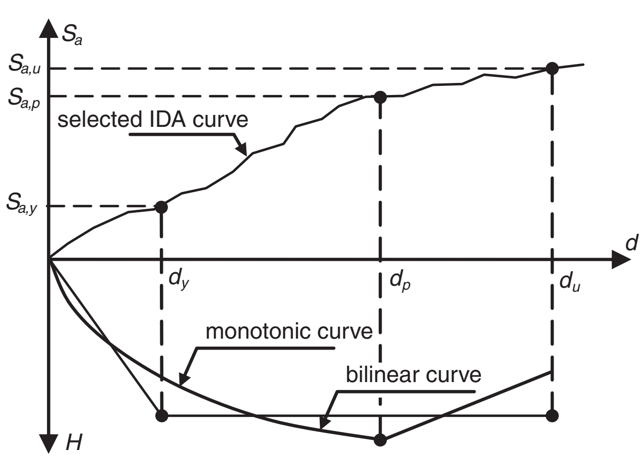

Fig. 9. Definition of behavior factors.

4.2. ''Classic'' approach

Nowadays, generally the seismic design according to current standards is performed by means force-based approaches, in which in the case of the ultimate limit state the structural inelastic response and overstrength are taken into account by reducing the seismic force, that usually corresponds to an hazard seismic level equal to 10% probability of exceedance in 50 years (10%/50). Therefore, the evaluation of behavior factors should be carried out on the basis of seismic demands required by records representative of 10%/50 earthquakes, typically by relating this hazard level to the life safety structural performance (Table 5). As result, these approaches allow significant structural damage under 10%/50 earthquakes, by neglecting the seismic performance evaluation in the case of seismic events having lower probability of exceedance.

Following this approach, one required performance objective only has been selected, in which the displacement demand (d_D) has been obtained under 10%/50 earthquakes, with a_g equal to 0.15 g, 0.25 g, and 0.35 g for low (LO), medium (ME) and high (HI) seismic intensity zone, respectively, and by adopting a displacement capacity corresponding to the ultimate displacement ($d_C = d_u$). For each IDA curve for which the performance coefficient $p = d_D/d_C$ is in the range from 0.5 to 1.0 (Fig. 10), the behavior factors q_1, q_2 and q_3 have been evaluated.

The obtained average, standard deviation and coefficient of variation values of behavior factors for G + G, G + O and all walls (G + G and G + O together) are shown in Table 6. The results show that q_1 is about 2.4 for all types of wall configurations, q_2 is about 1.3, even if in case of walls sheathed with G + O the value decreases to about 1.2, and q_3 is about 3.0 for all walls, while for G + G and G + O configurations is about 3.2 and 2.9, respectively. As for as the dispersion of the data is concerned, the coefficient of variation ranges from 0.10 to 0.32, which corresponds to moderately scattered results.

On the basis of results shown in Table 6 a behavior factor $q = q_3 = 3.0$ is proposed for the ''Classic'' seismic design under 10%/50 earthquakes.

4.3. ''Multi-performance'' approach

As attempt to overcome the limitations of as called ''Classic'' approach, in this paper is proposed a methodology in which the

Table 5
Performance objective matrix: "Classic" vs. "Multi-performance" approach.

		Performance levels		
		IO	LS	CP
Earthquake hazard level	50%50	[a,b]		
	10%50		[a,b]	
	2%50			[b]

[a] Goal achieved by "Classic" approach.
[b] Goal achieved by "Multi-performance" approach.

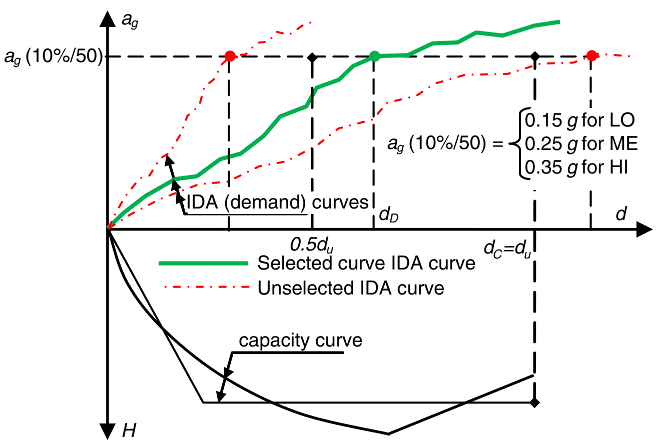

Fig. 10. CA: selection of IDA curves for the "classic" approach.

different behavior factors defined in Section 4.1 are related to different seismic performance levels, in such a way to obtain a "Multi-performance" seismic design. The peculiarity of the proposed approach is to give the possibility to achieve an "enhanced objective" (FEMA 2003), consisting of the following goals (Table 5): (1) immediate occupancy (IO) performance level for earthquakes having 50% probability of exceedance in 50 years (50%/50), that is ground motions with mean return period of about 75 years or so called "frequent" earthquakes; (2) life safety (LS) performance level for earthquakes having 10% probability of exceedance in 50 years (10%/50), that is ground motions with mean return period of about 500 years or so called "rare" earthquakes; (3) collapse prevention (CP) performance level for earthquake with 2% probability of exceedance in 50 years (2%/50), that is ground motions with mean return period of about 2500 years or so called "very rare" earthquakes. Therefore, if the displacement capacities of the generic SCFS shear wall associated to the different performance levels are assumed as follows: yield displacement for IO ($d_{C,IO} = d_y$), peak displacement for LS ($d_{C,LS} = d_p$) and ultimate displacement for CP ($d_{C,CP} = d_u$); then a seismic design which allows an adequate damage control for all selected earthquake hazard levels should be

Table 6
Behavior factors for the "Classic" approach.

Wall configuration		q_1	q_2	q_3
G + G	Average	2.37	1.32	3.16
	St. Dev.	0.53	0.20	1.02
	C.o.V.	0.22	0.15	0.32
G + O	Average	2.40	1.20	2.87
	St. Dev.	0.41	0.12	0.58
	C.o.V.	0.17	0.10	0.20
All types (G + G and G + O)	Average	2.38	1.27	3.04
	St. Dev.	0.49	0.19	0.88
	C.o.V.	0.20	0.15	0.29

reached. In fact, for IO performance level the assumed displacement capacities correspond to the interstory drift limits (d/h) ranging from 0.13% to 0.97%, which are less than those (1.5%) given by Dubina [6]. For LS and CP performance levels the assumed limits of d/h range from 0.61% to 1.87% and from 0.73% to 2.44%, respectively, which are generally more conservative values respect to those suggested by different authors: 2.5% for Dubina [6], 2.0% for Kawai et al. [16], and from 1.91 to 2.23 for Blais [1].

The possibility of damage containment based on the assumed interstory drift limits is also supported by available experimental experiences, which confirm a damage of sheathing connections negligible for d/h less than 1.0% (Fig. 11a), which became tolerable for d/h in the range 1.0% through 2.5% (Fig. 11b and c).

According to this approach, for each selected seismic hazard level, three different seismic intensities have been assumed: a_g equal to 0.06 g, 0.10 g, and 0.14 g for "frequent" earthquakes; 0.15 g, 0.25 g, and 0.35 g for "rare" earthquakes; and 0.23 g, 0.38 g, and 0.53 g for "very rare" earthquakes. Hence, in order to choose realistic cases, only IDA curves satisfying the following criterion (Fig. 12) have been considered:

$$(p_{IO} = d_{D,IO}/d_{C,IO} \leqslant 1) \cap (0.5 \leqslant p_{LS} = d_{D,LS}/d_{C,LS} \leqslant 1.0) \cap (p_{CP} = d_{D,CP}/d_{C,CP} \leqslant 1) \qquad (8)$$

The obtained average, standard deviation and coefficient of variation values of the behavior factors q_1, q_2 and q_3 are shown in Table 7. From the obtained results, it can be observed that q_1 is about 2.2, 2.4 and 2.3 for walls sheathed with G + G, G + O and all walls, respectively; q_2 is about 1.3 considering all types of wall configurations, while it is about 1.4 and 1.2 for G + G and G + O configurations, respectively; q_3 is about 3.1, 2.9 and 3.0 for walls sheathed with G + G, G + O and all wall typologies. The results in terms of dispersion are very similar to those observed in case of the "Classic" approach, with a coefficient of variation in the range from 0.10 to 0.27.

According to the proposed Multi-performance approach and by considering the obtained results, a behavior factor $q = q_2 = 2.0$ is proposed in the case of "rare" (10%/50) earthquakes, while its value should be assumed equal to $q = q_3 = 3.0$ for "very rare" (2%/50) earthquakes.

4.4. Comparison with literature results and code prescriptions

Considering the behavior factors defined on the basis of nonlinear dynamic analysis results using "Classic" and "Multi-performance" approaches, they range between 2.2 and 2.4 for q_1 (overstrength) between 1.2 and 1.4 for q_2 (ductility) and from 2.9 to 3.2 for q_3 (both overstrength and ductility).

The values of overstrength related factor (q_1) achieved in this study are very similar to those (q_1 = 2.2–2.6) proposed by Fülöp and Dubina [11], while they are larger than the corresponding

(a) $d/h \approx 0.6\%$ (b) $d/h \approx 1.4\%$ (c) $d/h \approx 2.0\%$

Fig. 11. Experimental observation of the sheathing fastener damage during a monotonic test.

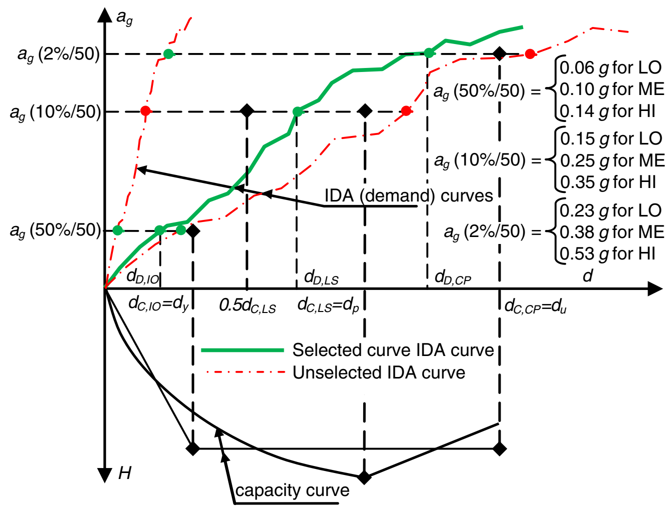

Fig. 12. Selection of IDA curves for the "Multi-performance" approach.

result (R_o = 1.7) given by Boudreault et al. [2]. For the values of the ductility related factor (q_2), they are slightly smaller than the results (q_2 = 1.4–1.6) obtained by Fülöp and Dubina [11] and quite smaller than the factor (R_d = 2.5) proposed by Boudreault et al. [2]. As consequence, the results obtained in terms of global behavior (q_3), which represents the product of q_1 and q_2 are smaller than those achieved by Fülöp and Dubina [11] (q_3 = 3.6–3.7) and Boudreault et al. [2] ($R_o \times R_d$ = 4.3). Finally, considering the global behavior factor (q_3), the comparison with the prescriptions of applicable Codes shows that the value proposed in this paper is in the very large range of those given by the Uniform Building Code [23], the International Building Code [15] and the FEMA 450 [7], where the indication closer than the q_3 proposed values seems to be that (4.5 for other types of walls) given by the Uniform Building Code [23]. potrebbe andare: "the closest values seems to be those provided by the Uniform Building Code (q_3 = 4.5 for other types of walls)" ?

5. A seismic design procedure based on non-linear dynamic analysis results

A seismic design methodology for SCFS housing that allows the screws spacing and all the shear walls components to be obtained on the basis of linear dynamic or non-linear static seismic analysis was presented by authors in Fiorino et al. [9]. As an attempt to complete this methodology, a design nomograph for the seismic design of single-storey SCFS frame structures developed on the basis of non-linear dynamic analysis results is presented in this paper. The proposed methodology refers to SCFS walls, without openings, in which the wall components are designed in such a way to promote the sheathing fastener failure. The design procedure can be summarized in three consecutive phases: (1) the selection of the wall geometry and some design parameters of wall components, which derives from architectural and technological considerations and vertical loads design; (2) the assessment of external screw spacing (s), which is the only design parameter that directly derives from seismic analysis results; (3) the evaluation of stud thickness, hold-down anchor diameter, and shear anchor spacing, which is carried out on the basis of "capacity design" criteria. More details about the hypothesis and limit of applicability can be found in Fiorino et al. [9].

The procedure for the evaluation of external screw spacing (phase 2) by means of non-linear dynamic (ND) analyses is described hereafter. As it is well known, when the non-linear dynamic procedure is selected for seismic analysis, the ductility and the overstrength of the structure are directly considered and the comparison between seismic capacity and demand can be obtained in terms of displacements (displacement-based approach):

$$d_C \geqslant d_D \tag{9}$$

where d_C and d_D are the seismic displacement capacity and demand, respectively. In particular, the displacement demand can be achieved by means of incremental dynamic analysis, in which the results can be given in terms of displacement demand (d_D) vs. peak ground acceleration (a_g) curve, which corresponds to the response of a fixed wall condition (wall geometry and materials, seismic weight and soil type). Instead, the displacement capacity corresponds to a given limit displacement (i.e. ultimate displacement d_u) on wall response curve, which can be expressed as a function of external screw spacing (s):

$$d_C = f_C(s) \tag{10}$$

Therefore, for a fixed wall condition, with exception of the external screw spacing (which is the design parameter), if the function $f_C(s)$ and the relevant IDA curves are known, then for each value of external screw spacing (s), both displacement demand (d_D) and capacity (d_C) can be evaluated and compared until Eq. (9) is satisfied. The design procedure can be represented graphically by means of the schematic "ND" nomograph shown in Fig. 13.

6. Case study

In the case study presented hereafter, a seismic design for a small structure is carried out by using the proposed ND nomograph following both "Classic" and "Multi-performance" approaches. The analyzed building is a typical one-family one-story dwelling with attic. The plan dimensions are 12 × 8 m, while the height is 6.7 m including a pitched roof with 100% slope. The structure is a stick-built construction in which floor, roof and walls are SCFS frames.

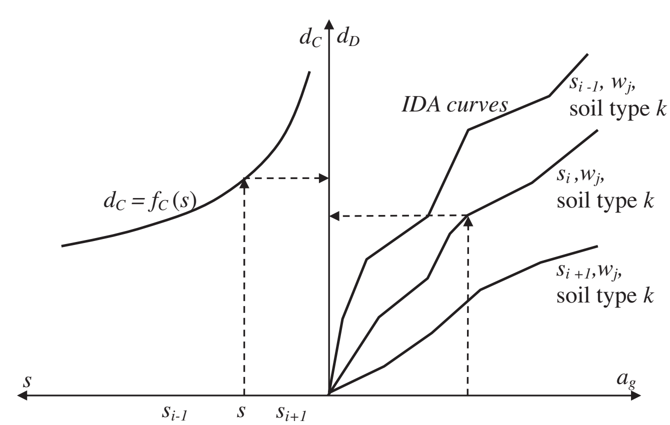

Fig. 13. Nomograph for ND procedure.

Table 7
Behavior factors for the "Multi-performance" approach.

Wall configuration		q_1	q_2	q_3
G + G	Average	2.23	1.35	3.05
	St. Dev.	0.44	0.17	0.83
	C.o.V.	0.20	0.13	0.27
G + O	Average	2.35	1.23	2.88
	St. Dev.	0.40	0.12	0.57
	C.o.V.	0.17	0.10	0.20
All types (G + G and G + O)	Average	2.29	1.29	2.96
	St. Dev.	0.43	0.16	0.71
	C.o.V.	0.19	0.12	0.24

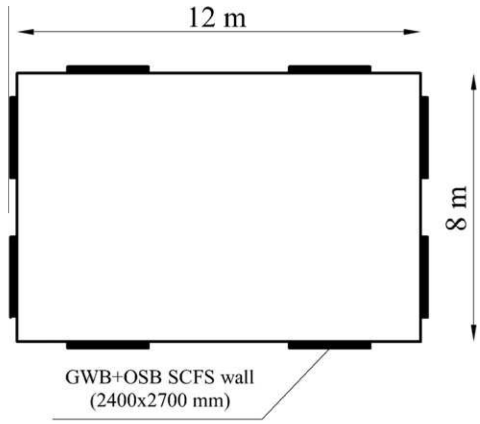

Fig. 14. Dwelling plan view.

Table 8
Design lateral strength for a G + O 2400 × 2700 mm wall.

External screw spacing (s) (mm)	Design lateral strength (H_C) (kN/m)
150	8.2
100	12.2
75	16.3
50	24.5

The SCFS walls are placed symmetrically along the house perimeter (Fig. 14) and represent the seismic resistant system. The walls are sheathed with G + O panels and they have dimensions 2400 × 2700 mm (length × height). The walls components are selected according to the data given in Section 2.2, while the external spacing of sheathing-to-frame connection represents the design parameter. The unit dead loads of the main elements are 0.60 kN/m^2 for the walls, 0.80 kN/m^2 for the floor and 0.85 kN/m^2 for the roof, while the live load is assumed equal to 2.0 kN/m^2. The resulting unit seismic weight evaluated by Eq. (6) results equal to 2.6 kN/m^2.

The house is located in a medium intensity seismic zone, therefore for "Classic" approach the assumed value of PGA (a_g) corresponding to 10%/50 earthquakes is 0.25 g, while for "Multi-performance approach" the PGA values are 0.10 g, 0.25 g and 0.38 g for 50%/50, 10%/50 and 2%/50 earthquakes, respectively. The assumed soil conditions are type D, according to Eurocode 8 [4] classification.

In a first phase, the seismic analysis has been carried out by adopting the following hypotheses: the ground motion acts in longitudinal direction; the floor is assumed as a rigid diaphragms; the possible torsional effects are neglected; the dynamic lateral behavior of the structure is represented by the dynamic lateral behavior of the walls; the "segment" method is used for describing the shear behavior of the SCFS walls, assuming that the sum of the length of the resistant wall segments in longitudinal direction is 9.6 m; the dynamic lateral behavior of the shear walls is described by a SDOF system. The seismic design, which consists mainly in the selection of the adequate external screw spacing is carried out by a linear seismic analysis, in which the design seismic force acting on a wall with unit length may be calculate by the following relationship:

$$\bar{H}_D = \frac{S_e(T)w}{q} \tag{11}$$

where the seismic weight per unit length (w) results equal to 26 kN/m and the behavior factor (q) is assumed equal to 3 for "Classic" approach, while for "Multi-performance" approach the assumed values are 1, 2 and 3 for IO, LS and CP levels, respectively. The normalized (respect to gravity acceleration) elastic spectral acceleration ($S_e(T)$) has been obtained as the maximum value obtained for natural vibration periods ranging between 0.19 s and 0.27 s, which have been evaluated on the basis of all possible lateral stiffness values for the selected wall configuration (lateral stiffness per unit length ranging from 1.4 to 2.7 kN/mm/m, depending on the external screw spacing). As result $S_e(T)$ equal to 0.33, 0.84 and 1.38 have been obtained for frequent, rare and very rare earthquake, respectively. Therefore, for "Classic" approach the design force ($\bar{H}_D$) results equal to 0.84 × 26/3 = 7.3 kN/m, while the values of the design forces ($\bar{H}_D$) for "Multi-performance" approach are 0.33 × 26/1 = 8.7 kN/m, 0.84 × 26/2 = 10.9 kN/m and 1.38 × 26/3 = 12.0 kN/m for frequent, rare and very rare earthquake, respectively.

The design wall lateral strength may be estimated by the following relationship:

$$\bar{H}_C = \frac{0.625\bar{H}_{y,av}}{\gamma_w} \tag{12}$$

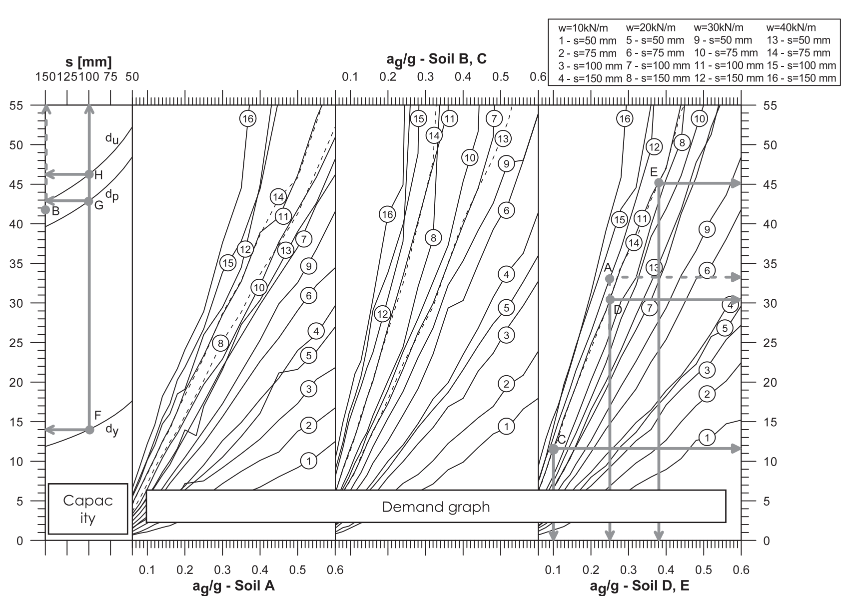

Fig. 15. Application of ND dynamic nomograph following the "Multi-performance" approach.

in which $\bar{H}_{y,av}$ is the conventional yielding strength on the idealized bilinear of monotonic average response curve obtained by the parametric analysis illustrated in Section 2, 0.625 is the factor which convert the average value of wall strength into the characteristic value and γ_w is a partial safety factor assumed equal to 1.25. The design values of wall lateral strength for different external screw spacings are shown in Table 8.

If the "Classic" approach is used, an external screw spacing equal to 150 mm is adequate ($\bar{H}_D/\bar{H}_C$) = 7.3/8.2 = 0.89). Instead, for "Multi-performance" approach 100 mm screw spacing is needed (max$\bar{H}_D/\bar{H}_C$ = 12.0/12.2 = 0.98). Therefore, this result confirms that in order to limit the wall damage, it is necessary to increase the wall strength by reducing the screw spacing.

In a second phase, the designed structure has been checked by means of the non-linear dynamic seismic procedure presented in Section 5 by using the ND nomograph shown in Fig. 15. In particular, each curve is derived by considering the average of the IDA curves obtained for a given wall condition. These demand curves are grouped by soil type in three different part of the graph. As mentioned before, the curves provide the displacement demand as function of seismic intensity (a_g/g). In addition, yield (d_y), peak (d_p) and ultimate displacement (d_u), as defined in Section 4.1, are assumed for seismic capacity and are given as a function of external screw spacing (s).

For the application of nomograph in the case of the "Classic" approach, the seismic displacement demand is estimated on the demand graph for type D soil considering the curve 12 (s = 150 mm, w = 30 kN/m). On this curve, for an a_g = 0.25 g, the displacement demand d_D is equal to 33 mm (point A). The point B on capacity graph represents the seismic displacement capacity ($d_C = d_u$ = 43 mm) corresponding to s = 150 mm. The point B is upper than point A, then the external spacing is adequate ($p = d_D/d_C$ = 0.77).

For "Multi-performance" approach, the displacement demand for each level is evaluated on the demand graph for type D soil by considering curve 11 (s = 100 mm, w = 30 kN/m). In particular, for a_g=0.10 g (IO) the displacement demand is 12 mm (point C), for a_g = 0.25 g (LS) the displacement demand is 31 mm (point D), while for a_g = 0.38 g (CP) the displacement demand is 45 mm (point E). On the capacity graph for s equal to 100 mm, the displacement capacity is: d_y = 14 mm for IO (point F), d_p = 43 mm for LS (point G) and d_u = 46 mm for CP (point H). The points F, G and H are higher than points C, D and E, respectively, then the external spacing is adequate and the performance coefficients are p(IO) = 0.86, p(LS) = 0.72 and p(CP) = 0.98.

7. Conclusions

In the last years a large number of research teams have been involved in the evaluation of seismic capacity of SCFS structures and seismic design parameters as behavior factors and interstory drift limits so as dynamic characteristics as vibration periods and damping ratios have been evaluated. In this paper the results of extensive parametric dynamic analyses on sheathed cold-formed steel (SCFS) shear walls, performed by a presented cyclic model able to predict the non linear force deflection response, have been presented. Based on the results of the analyses the seismic performance of SCFS shear walls, in terms of behavior factors has been assessed. In particular, three behavior factors related to overstrength (q_1 = 2), ductility (q_2 = 1.5) and both overstrength and ductility (q_3 = 3) have been defined.

In addition, two different seismic design approaches, in which the difference consists on the required performance objectives, have been proposed. For the first approach, named "Classic", in which 10%/50 hazard level is related to the life safety structural performance, a behavior factor $q = q_3$ = 3 is proposed to be used. For the second approach, named "Multi-performance", different behavior factors are related to different seismic performance levels, in such a way to achieve an "enhanced objective" by allowing an effective damage control. Under this assumption, a behavior factor equal to $q = q_2$ = 2 is proposed for 10%/50 hazard level, which is related to live safety structural performance; while a behavior factor of $q = q_3$ = 3 should be used for 2%/50 hazard level, which correspond to collapse prevention structural performance.

Finally, a seismic design procedure which allows the evaluation of the external screw spacing by non-linear dynamic analysis results has been presented. The procedure applicability is demonstrated by through the design of a typical one-story house located in a medium seismic intensity area. The case study results confirm that an "enhanced objective" design approach is feasible with a little reduction of screw spacing.

References

[1] Blais C. Testing and analysis of light gauge steel frame/9 mm OSB wood panel shear walls. Master's thesis. Montreal, Canada: Department of Civil Engineering and Applied Mechanics, McGill University; 2006.
[2] Boudreault FA, Blais C, Rogers CA. Seismic force modification factors for light-gauge steel-frame–wood structural panel shear walls. Can J Civ Eng 2007;3(1):56–65.
[3] Branston AE, Chen CY, Boudreault FA, Rogers CA. Testing of light-gauge steel-frame–wood structural panel shear walls. Can J Civ Eng 2006;33(5):561–72.
[4] CEN. EN 1998-1 – Eurocode 8: design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings. Bruxelles: European Committee for Standardization; 2004.
[5] Della Corte G, Landolfo R, Fiorino L. Seismic behavior of sheathed cold formed structures: numerical study. J Struct Eng ASCE 2006;132(4):558–69.
[6] Dubina D. Behaviour and performance of cold-formed steel-framed houses under seismic action. J Constr Steel Res 2008;64:896–913.
[7] FEMA 450. NEHRP recommended provision for seismic regulations for new buildings and other structures. Washington, DC: Federal Emergency Management Agency; 2003.
[8] Fiorino L, Della Corte G, Landolfo R. Experimental tests on typical screw connections for cold-formed steel housing. Eng Struct (Elsevier Science) 2007;29(8):1761–73.
[9] Fiorino L, Iuorio O, Landolfo R. Sheathed cold-formed steel housing: a seismic design procedure. Thin Walled Struct (Elsevier Science) 2009;47(8–9):919–30.
[10] Fülöp LA, Dubina D. Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading. Part I: Experimental research. Thin Walled Struct (Elsevier Science) 2004;42(2):321–38.
[11] Fülöp LA, Dubina D. Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading. Part II: Numerical modelling and performance analysis. Thin Walled Struct (Elsevier Science) 2004;42(2):339–49.
[12] Gad EF, Duffield CF, Hutchinson GL, Mansell DS, Stark G. Lateral performance of cold-formed steel-framed domestic structures. Eng Struct (Elsevier Science) 1999;21:83–95.
[13] Hilti. North America Product Technical Guide; 2005.
[14] Iervolino I, Maddaloni G, Cosenza E. Unscaled real record sets compliant with Eurocode 8. In: First European conference on earthquake engineering and seismology. Geneva, Switzerland; 2006.
[15] IBC. International Building Code (IBC). Country Club Hills, IL: International Code Council; 2003.
[16] Kawai Y, Kanno R, Uno N, Sakumoto Y. Seismic resistance and design of steel framed-houses. Nippon Steel Technical Report; 1999. p. 79.
[17] Landolfo R, Fiorino L, Della Corte G. Seismic behaviour of sheathed cold-formed structures: physical tests. J Struct Eng 2006;132(4):570–81.
[18] Landolfo R, Fiorino L, Iuorio O. A specific procedure for seismic design of cold-formed steel housing. Adv Steel Construct 2010;6(1):603–18.
[19] NRCC. National Building Code of Canada (NBCC). Ottawa, Ontario: National Research Council Canada; 2005.
[20] Park R. Evaluation of ductility of structures and structural assemblages from laboratory testing. Bull New Zealand Natl Soc Earthquake Eng 1989;22(3):155–66.
[21] Rokas D. Testing and evaluation of light gauge steel frame/ 9.5 mm CSP wood panel shear walls. Master's thesis. Montreal, Canada:Department of Civil Engineering and Applied Mechanics, McGill University;2006.
[22] Simpson Strong-Tie Company; 2007. www.strongtie.com.
[23] UBC. Uniform Building Code. Whittier, CA: International Conference of Building Officials; 1997.