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ASSESSMENT OF PUSHOVER RESPONSE CURVE FOR STEEL FRAME / PANEL SHEAR WALLS

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ABSTRACT

Methods allowing a reliable evaluation of the pushover response curve are useful tools when the nonlinear static procedure is used for seismic analysis. In this paper, a model able to predict the nonlinear shear vs. top wall displacement relationship for steel frame/panel shear walls on the basis of screw connection test results is firstly proposed. In addition, a comparison with available wall test results is presented, in order to check the reliability of the numerical predictions. Finally, the application of the proposed model for evaluating the seismic response of a case study house is discussed.

Introduction

Cold-formed/light gauge steel buildings typically use panel sheathing fastened to steel stud framing (steel frame / panel shear walls) to provide an adequate lateral force resisting system. Reliability of the shear response evaluation of these systems is critical to the accuracy and efficiency of seismic analysis and design of these types of structures.

Different approaches exist to calculate the shear response of sheathed cold-formed shear walls: experimental, numerical and analytical methodologies. The experimental approach is based on full scale tests carried out on typical walls and it is the most common approach. In fact, nominal shear strength design values given by building codes (UBC 1997, IBC 2003) are based on experimental test results. This approach is the most expensive and can be used only when the wall characteristics (geometry and materials) are within the range of experimental results. To overcome limitations of the experimental approach, finite element methods may be utilized to evaluate the shear response of sheathed cold-formed shear walls. Numerical models are calibrated on available experimental results and they are used to simulate the structural performance of walls having characteristics different from tested walls.

No literature is available regarding analytical approaches specifically developed for steel frame / panel shear walls. On the contrary, a large number of methods developed for the analysis of wood frame / panel shear walls is available. Because the global response of steel-framed and wood-framed walls sheathed with panels under shear loads is qualitatively very similar, the application of existing analytical models for wood-framed walls is reasonable also in the case of steel-framed walls.

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The possibility of giving predictions for only strength and deflection, without furnishing a reliable evaluation of the whole load vs. deflection response curve, represents the main limitation of existing analytical approaches, especially when a nonlinear static procedure is selected for seismic analysis of the construction. As an attempt to overcome this limitation, a model allowing the prediction of the whole nonlinear shear vs. top wall displacement relationship is proposed. In particular, after a general review of existing analytical and numerical methods for wood-framed shear walls, the paper presents the proposed model and illustrates the comparisons between available experimental results and those obtained with existing approach and proposed model. Finally, the seismic performance assessment of a case study house performed through a nonlinear static analysis is presented as a possible application of the proposed model.

Model Derivation

Based on results of experimental tests on two, nominally identical, cold-formed stick-built house sub-assemblages, a model for the prediction of load vs. deflection response curve of sheathed steel-framed shear walls is proposed and illustrated hereafter. The model is based on the observation of the deformation pattern during the tests. In particular, the following hypotheses are made in the proposed approach: (a) local failure of sheathing-to-frame connections governs the global collapse mode; (b) studs and tracks are rigid and hinged to each other; (c) the wall framing deforms into a parallelogram and the relative frame-to-panel displacements are determined based on a rigid body rotation of panels; (d) the edges of the panel are free to rotate without interference from adjacent sheathings and the foundation or other stories; (e) the wall is fully anchored to the foundation or lower storey; (f) relative displacements between the sheathing and framing are small compared with the panel size; (g) only shear deformation of the sheathings is considered by adopting the equation for shear deformation of a thin, edge-loaded, plate; (h) load-displacement curves of the sheathing-to-frame connections are schematized by using the relationship proposed by Richard and Abbott (1975). The assumed deformation of a sheathed cold-formed shear wall is shown in Figure 1.

For a sheathing-to-frame connection *i*, the relative displacements between the framing member and the panel are given by the following relationships:

$$u_i = u_{f,i} - u_{p,i} = (\varphi_p - \varphi_f) y_i - u_{p0} \qquad v_i = v_{f,i} - v_{p,i} = \varphi_p (b/2 - x_i) \tag{1}$$

where u_i and v_i are the relative displacement components of the framing members to the panel along X and Y directions, respectively; $u_{f,i}$ and $v_{f,i}$ are the displacement components of the framing members along X and Y directions, respectively; $u_{p,i}$ and $v_{p,i}$ are the displacement components of the panel along X and Y directions, respectively; φ_f and φ_p are the rotations (defined positive as anticlockwise rotations) of the frame and panel, respectively; u_{p0} is the translation of the panel along X direction; h and b are the height and width of the wall, respectively; x_i and y_i are the coordinates along X and Y directions, respectively.
From equilibrium considerations involving moment equilibrium and horizontal force equilibrium for the panel, and horizontal force equilibrium for the top track, the following formulas can be obtained:

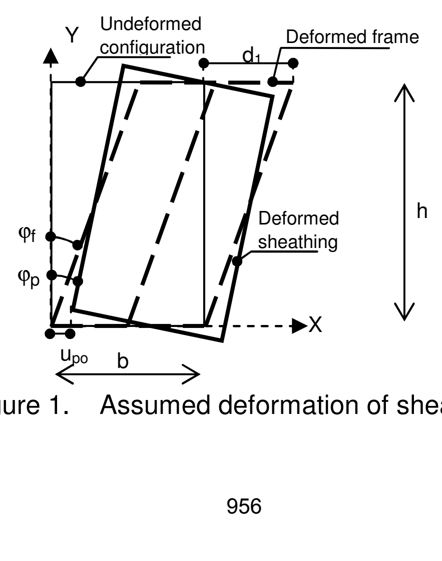

Figure 1. Assumed deformation of shear wall.

$$\sum_{i=1}^{n}\left(-F_{x,i}y_i+F_{y,i}x_i\right)=0 \qquad \sum_{i=1}^{n}F_{x,i}=0 \qquad V\,b-F_{x,e}n_e-\frac{1}{h}\sum_{i=1}^{m}F_{x,i}y_i=0 \tag{2}$$

where $F_{x,i}$ and $F_{y,i}$ are the force components of sheathing-to-frame connections along X and Y directions, respectively; $F_{x,e}$ is the force component of sheathing-to-top track connections along the X direction, which is constant according to the considered hypotheses; V is the horizontal external force per unit of length; n is the total number of sheathing-to-frame connections; m is the number of fasteners connecting the sheathing to studs; n_e is the number of fasteners connecting the sheathing to the top track.

The force components of sheathing-to-frame connections can be expressed as functions of relative displacements between the steel framing members and panel by:

$$F_{x,i}=k_{x,i}u_i \qquad F_{y,i}=k_{y,i}v_i \tag{3}$$

where $k_{x,i}$ and $k_{y,i}$ are the stiffnesses of sheathing-to-frame connections for displacement along X and Y directions, respectively.

Using Equations 1 through 3, the parameters describing the deformation of the wall (φ_f, φ_p, u_{p0}) are expressed as function of the wall geometry, stiffnesses of sheathing-to-frame connections ($k_{x,i}$, $k_{y,i}$), and horizontal external force per unit of length (V):

$$\varphi_f=\frac{2bh\left[K_xI_x-\left(S_x\right)^2-\dfrac{bK_xS_y}{2}+K_xI_y\right]V}{(S_{x,m}S_x-I_{x,m}K_x+S_eS_x-I_eK_x)(2I_y-bS_y)} \tag{4}$$

$$\varphi_p=\frac{-2bh[K_xI_x-(S_x)^2]V}{[(I_{x,m}+I_e)K_x-(S_{x,m}+S_e)S_x](2I_y-bS_y)} \tag{5}$$

$$u_{p0}=\frac{bhS_xV}{[I_{x,m}+I_e]K_x-[S_{x,m}+S_e]S_x} \tag{6}$$

in which:

$$K_x=\sum_{i=1}^{n}k_{x,i} \qquad S_x=\sum_{i=1}^{n}k_{x,i}y_i \qquad I_x=\sum_{i=1}^{n}k_{x,i}\left(y_i\right)^2$$

$$S_y=\sum_{i=1}^{n}k_{y,i}x_i \qquad I_y=\sum_{i=1}^{n}k_{y,i}\left(x_i\right)^2 \qquad S_{x,m}=\sum_{i=1}^{m}k_{x,i}y_i \qquad I_{x,m}=\sum_{i=1}^{m}k_{x,i}\left(y_i\right)^2$$

$$K_e=k_{xe}n_e \qquad S_e=k_{xe}n_eh \qquad I_e=k_{xe}n_eh^2$$

When for sheathing-to-frame connections a linear load-displacement response is assumed ($k_{x,i}$ and $k_{y,i}$ are constant values), Equation 4 gives a closed-form solution and the top wall displacement (d) can be evaluated as follows:

$$d=d_1+d_2=\varphi_f h+\frac{h}{Gt}V \tag{7}$$

where $d_1=\varphi_f\,h$ is the displacement obtained by assuming that the panel has rigid body rotation (see Fig. 1); $d_2=hV/Gt$ is the displacement obtained by considering only shear deformation of the panel; φ_f is calculated from Equation 4; G is the shear modulus of elasticity of the panel material; t is the panel thickness.
When a nonlinear load-displacement curve is adopted for sheathing-to-frame connections, Equations 1

through 6 can be written in differential format and can be used in a numerical step-by-step procedure which allows to obtain the load vs. deflection response curve of the wall. More details on the numerical procedure are given in Fiorino et al. (2006a,b).

Model Calibration

A preliminary calibration of the load vs. deflection response curve prediction, obtained by applying the proposed model, has been carried out considering experimental results of full scale tests on walls (Landolfo et al. 2006a, Della Corte et al. 2006) and shear tests on connections (Landolfo et al. 2006b, Fiorino et al. 2006c).

Full scale tests were carried out on two specimens representative of a typical steel stick-built structure sheathed with panels. In particular, the generic wall was 240cm long and 250cm height, consisted of a cold-formed frame sheathed with 9mm thick oriented strand board (OSB) external panels and 12.5mm thick gypsum wallboard (GWB) internal panels. Both panels were attached to the frame with screw connections spaced at 15cm at the perimeter and 30cm in the field.

Shear tests have been carried out on 62 screw connections between panels and cold-formed steel members nominally identical to those used in full scale tests on walls. In particular, the generic connection specimen consisted of two single panels attached to the opposite flanges of stud profiles in such a way that 6 screws were tested for each specimen. Three different values of the loaded edge distance (*a*) were adopted (10mm, 15mm, 20mm) and, in case of OSB specimens two different sheathing orientations were examined (strand orientation parallel and perpendicular to the load direction).

In order to discuss the analytical representation of load-displacement curves of sheathing-to-frame connections, the following definitions are introduced: F_u is the peak strength (maximum recorded load); s_p is the peak displacement (displacement corresponding to F_u); $k_p=F_u$ / s_p is the peak secant stiffness; $F_e=0.4F_u$ is the conventional elastic strength; s_e is the conventional elastic displacement (displacement corresponding to F_e); $k_e=F_e$ / s_e is the conventional elastic secant stiffness; s_u is the conventional ultimate displacement (displacement corresponding to a load equal to $0.80F_u$ on the post-peak branch of response).

The load-displacement (*F*-*s*) curve of a generic connection has been analytically expressed as follows (Fig. 2):

for $s \leq s_p$:
$$F(s)=\frac{(k_0-k_h)s}{\left[1+\left|\frac{(k_0-k_h)s}{F_0}\right|^{\alpha}\right]^{\frac{1}{\alpha}}}+k_h s$$
Richard and Abbott (1975) curve (17)

for $s_p < s \leq s_u$:
$$F(s)=\frac{0.2F_u}{s_u-s_p}(s_p-s)+F_u$$
linear branch (18)

where: k_0 is the initial stiffness; k_h is the slope of the straight line (hardening line) asymptote of the assumed *F*-*s* curve; F_0 is the intersection between the hardening line and the s = 0 axis; α is a shape parameter regulating the sharpness of transition from the elastic to the plastic behaviour (for α large enough a bilinear response is obtained).

The values of the parameters k_0, k_h, F_0, and α have been defined considering the following conditions: k_0 is equal to the initial stiffness of experimental average curve; k_h, F_0, and α are determined imposing that the analytical curve intersects the experimental average curve at the following three points: (1) conventional elastic point (s_e, F_e); (2) peak point (s_p, F_u); (3) a point (s_x, F_x), with $s_e<s_x<s_p$, defined in such a way to minimize the difference between the areas under the analytical and experimental curves (A1=A2) for $0\leq s \leq s_p$.

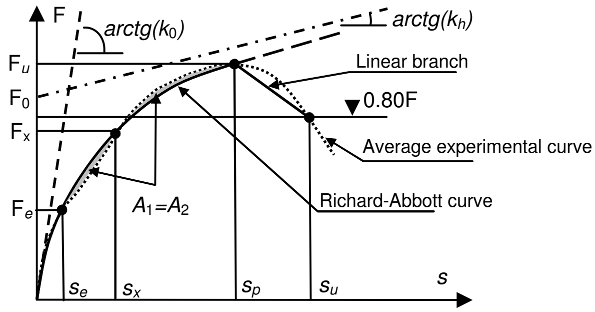

Figure 2. Analytical schematization of connection F-s response curve.

Because experimental results on walls are relevant to monotonic load conditions, in which displacements are applied at a rate less than or equal to 0.20mm/s, connection tests carried out in quasi-static monotonic tension loading regime have been considered only. When a sheathed cold-formed shear wall is subjected to shear loads, the wall framing deforms into a parallelogram and the deformation of the panels is mainly due to a rigid body rotation. Therefore, the amplitude and direction of relative frame-to-panel slips are dependent on the connection. As a consequence, the loading edge distance and the sheathing orientation (in the case of OSB panels) are not univocally defined. For this reason, the selection of the loaded edge distance (a) and OSB sheathing orientation have been defined by means of a preliminary study carried out considering all examined loaded edge distances (10, 15 and 20mm) and OSB sheathing orientations (parallel (//) and perpendicular (⊥) to the load direction) as examined in connection shear tests. Based on results of the preliminary study (Fig. 3), only specimens having a=20mm and OSB panels with strand orientation parallel to the load direction have been considered because in this case the best agreement between experimental and analytical response was obtained.

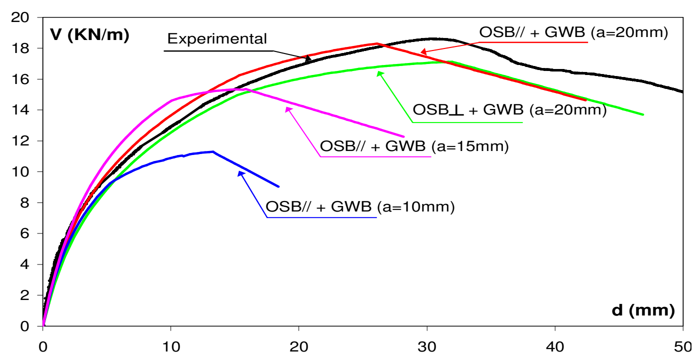

Figure 3. Calibration of the proposed model.

Preliminary Model Validation

To assess the reliability of the proposed model, the results obtained from its application in the case of full scale tests on steel-framed walls carried out by Landolfo et al. (2006a) has been compared with the results obtained applying some existing methods able to predict the deflection (Eurocode 5 (CEN 1993), Finnish timber Code RIL 120-2001 (Hieta and Kesti 2002)) or load-displacement curve (McCutchenon (1985), Easley et al. (1982)) of wood-framed walls.

For all examined methods the following assumptions have been made: (1) for evaluating the shear response of walls the contribution of OSB and GWB panels are added; (2) the shear modulus of elasticity of the sheathings is 1400MPa for OSB panels and 750MPa for GWB panels; (3) shear tests on connections having a=20mm and OSB panels with strand orientation parallel to the load direction are considered for establishing the connection parameters. In addition, for the application of the Eurocode 5 and Finnish timber Code's methods the following stiffnesses are obtained: k_e equal to 1.08 and 1.79

kN/mm for OSB and GWB, respectively and k_p equal to 0.36 and 0.18 kN/mm for OSB and GWB, respectively. The power functions used in the McCutchenon's method are determined imposing that the power curve intersects the experimental average curve at the conventional elastic point (s_e, F_e) and peak point (s_p, F_u). The four-parameters connection response curves adopted in the Easley et al.'s method are determined analogously to that described for the Richard and Abbott curve (imposing that the analytical curve intersects the experimental curve at the point (s_e, F_e), (s_p, F_u), and (s_x, F_x)).

Adopted analytical response curves are shown in Figure 4, in which also experimental connection responses are reported. Figure 5 shows the comparison between experimental response and analytical responses in terms of unit shear load (V) vs. deflection (d) curves. From this Figure, it can be noticed that the proposed model gives a result which seems accurate enough in comparison with the experimental response. In particular, the proposed model gives a good prediction of strength, while it slightly overestimates the displacements for d<4mm and underestimates the displacements for d>4mm. As far as the comparison between the proposed and considered existing methods is concerned, Figure 5 shows that the McCutchenon and Finnish timber Code's methods underestimate the shear capacity, the Eurocode 5 gives results that overestimate the shear capacity and in the case of Easley et al.'s method the shear capacity is underestimated for d<2.5mm and overestimated for d>2.5mm.

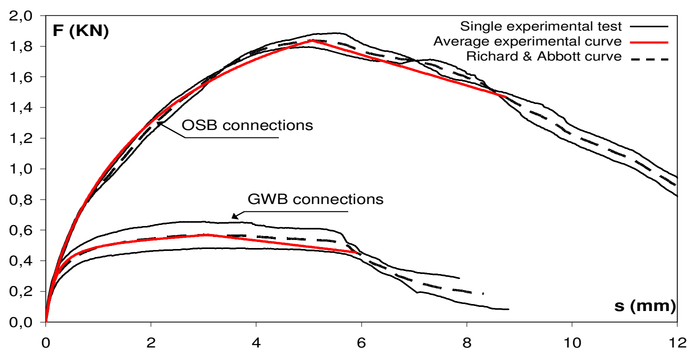

Figure 4. Load-displacement curves of sheathing-to-frame connections.

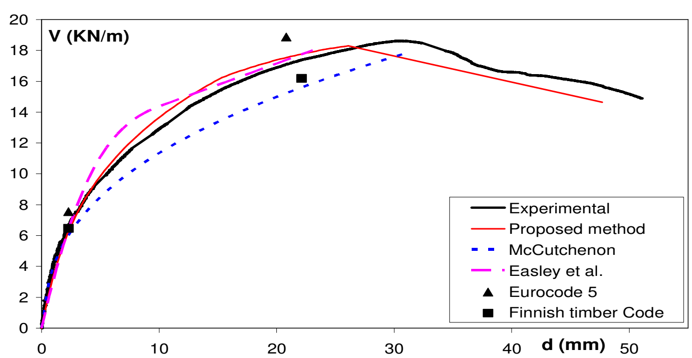

Figure 5. Load vs. deflection curve of examined wall: Experimental vs. analytical response.

To assess the reliability of strength prediction, the comparison between the predicted-to-test peak load ratios (ρ_V = $V_{u,ana}$ / $V_{u,exp}$) obtained by considering all applied methods reveals that the best strength prediction is given by Eurocode 5 (ρ_V = 1.01) and the proposed model (ρ_V = 0.98). The worst prediction is given by the Finnish timber Code (ρ_V = 0.87), while the other methods provide good and similar results (ρ_V equal to 0.97 and 0.96 for the Easley et al. and McCutchenon methods, respectively).

For evaluating the reliability of the deflection prediction, a comparison between the deflection measured during testing at the conventional elastic load ($d_{e,exp}$) and at the peak load ($d_{p,exp}$) and those predicted using the considered methods ($d_{e,ana}$ and $d_{p,ana}$) has been carried out. In particular, the conventional elastic deflection (d_e) for a wall has been defined analogously to the conventional elastic displacement (s_e) of a connection (deflection measured when a load equal to 40% of the peak load is applied).

The comparison, illustrated in terms of predicted-to-test ratios, reveals that the in the case of elastic deflection, the best predictions are given by the proposed model (ρ_{de}=$d_{e,ana}$ / $d_{e,exp}$=1.02). Good results are given also by Easley et al.'s methods (ρ_{de}=0.92), while the other methods provide less accurate results (ρ_{de}=1.17 for the McCutchenon's method and ρ_{de}=0.79 for both Finnish timber Code and Eurocode 5). In the case of peak deflection prediction, the McCutchenon's method gives the best result (ρ_{dp}=$d_{p,ana}$ / $d_{p,exp}$=1.03). An almost accurate result is obtained also with the proposed model (ρ_{dp}=0.86), while worse predictions are given by Easley et al., Finnish timber Code and Eurocode 5's methods (ρ_{dp} equal to 0.76, 0.73 and 0.68, respectively). As far as the evaluation of the conventional ultimate displacement (d_u: displacement corresponding to a load equal to 0.80 V_u on the post-peak branch of response) is concerned, its predictions are only possible with the proposed model. In fact, only this model is able to capture the post-peak branch of response and the obtained results are slightly conservative ($d_{u,ana}$ / $d_{u,exp}$=0.93).

Nonlinear Static Seismic Analysis Using the Proposed Model: A Case Study

As a complementary task, the seismic performance level of a case study house has been evaluated through a nonlinear static seismic analysis, in which the pushover curve has been obtained using the proposed model. The examined house is a model representative of a typical 2-storey, one-family dwelling having cold-formed steel walls sheathed with OSB panels. In particular, the load bearing walls consisted of cold-formed frames sheathed on both sides with 9mm thick OSB panels having screw sheathing-to-frame connections spaced at 15cm at the perimeter and 30cm in the field.

For the purpose of seismic analysis, the global force-displacement response curve of the house (base shear vs. 2nd floor lateral displacement curve) has been obtained from the load-deflection response curve of the single wall, which has been determined by using the proposed model. In particular, the single wall response curve obtained with the model has been successively modified to take into account the additional deflections due to the wall base rotation and floor deformation (Fig. 6).

The displacement demand has been estimated using the equal displacement-equal energy approach. This approach requires an idealized bilinear force-displacement curve; therefore an elastic-plastic curve has been obtained by using the equivalent energy elastic-plastic method as recommended by Branston (2004). In addition, since the load vs. deflection response curve of the wall has been derived by considering the average load vs. displacement curve of sheathing-to-frame connections, then the resistance of the average idealized force-displacement curve ($V_{y,av}$) has been reduced in such way to obtain the design idealized force-displacement curve ($V_{y,d}$= $V_{y,k}$ / γ_m, in which $V_{y,k}$ = 0.75 $V_{y,av}$ is the characteristic resistance, and γ_m = 1.25 is the partial factor for resistance) (Fig. 7). Therefore, the parameters defining the idealized bilinear force-displacement curve per unit length of wall are: design resistance $V_{y,d}$ = 22.6 kN/m, stiffness K_e = 1.19 kN/mm/m, ultimate displacement d_u = 68.2 mm.

For the dynamic characterization, the equivalent SDOF system has been generated by using a first mode response having an inverted triangular linear shape (Fig. 7). The masses for both first and second floors have been assumed in such way to have a mass per unit length of wall (m) ranging from 50 to 150 kg/m. Starting from these values, the modal participation factor is Γ = 1.25, while the modal mass coefficient (m^*) ranges from 0.75 kg/m (for m = 50 kg/m) to 2.25 kg/m (for m = 150 kg/m), and the equivalent effective fundamental period (T^*) ranges from 0.16 s (for m = 50 kg/m) to 0.27 s (for m = 150 kg/m).

The elastic acceleration demand spectra provided by Eurocode 8 (CEN 2005) and adopted also by the new Italian Seismic Code (OPCM 2005) have been assumed. In particular, the Italian Seismic Code defines soil types A, B, C, D, and E according to the Eurocode 8 classification, but only three spectra are adopted grouping the soil types B, C and E under one spectrum type. Values of the peak design ground acceleration (a_g) on type A soil (for earthquakes having 10% chance of being exceeded over 50 years – average return period 474 years) are 0.35g, 0.25g, and 0.15g for seismic zones having low, medium and high intensity, respectively. For earthquakes having 2% and 50% probability of exceedance in 50 years (average return periods of 2475 and 72 years, respectively) the demand spectra are multiplied by 1.5 and 0.4, respectively. Parameters describing the elastic response spectrum are reported in Table 1, in which: S is the soil factor; T_B, and T_C are the limits of the constant spectral acceleration branch, respectively; T_D is the value defining the beginning of the constant displacement response range of the spectrum.

A relative viscous damping ratio equal to 5% has been adopted, being this value based on the basis of literature data (Kawai et al. 1999, Dubina et al. 2006).

The main objective of the performed nonlinear static seismic analysis is to estimate the performance level of the case study house under earthquakes having different hazard levels (earthquakes having 50%, “frequent”, 10%, “rare”, and 2%, “very rare”, probability of exceedance in 50 years have been considered). For each selected seismic hazard level, different soil conditions (“rock”, “soft”, and “very soft” soil) and seismic intensities (a_g equal to 0.35g, 0.25g, and 0.15g) have been assumed. The range of performance levels, expressed in terms of inter-story drift values (d / h) have been assumed as follows: (a) $(d / h)_{IO}$ = 0.25% for the “immediate occupancy” (IO) level; (b) $(d / h)_{LS}$ = 1.00% for the “life safety” (LS) level; and (c) $(d / h)_{CP}$ = 1.28% for the “collapse prevention” (NC) level. In particular, the drift limit values corresponding to IO and LS levels have been set in accordance with the provision of FEMA-356 (FEMA 2000) for wood stud walls, while the limit value corresponding to CP level has been assumed on the basis of the ultimate displacement of the global force-displacement response curve estimated with proposed model ($(d / h)_{CP}$ = d_u / h = 69.2 / 5400 = 1.28%).

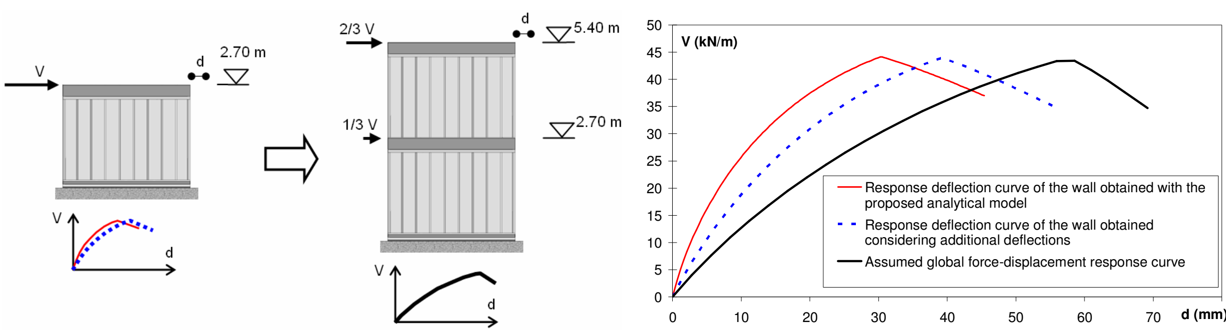

Figure 6. From the response curve of the wall to the global response curve of the.

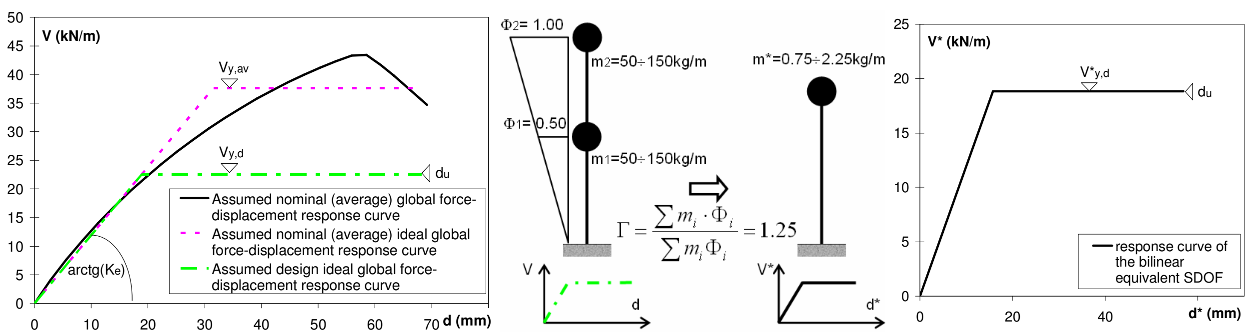

Figure 7. From the global response curve to the response curve of to the equivalent SDOF system.

Table 1. Values of the parameters describing the elastic response spectrum (OPCM 2005).

Ground type	***S***	T_B	T_C	T_D
A (Rock soil)	1.0	0.15	0.40	2.0
B-C-E (Soft soil)	1.25	0.15	0.50	2.0
D (Very soft soil)	1.35	0.20	0.80	2.0

The results of the performance level assessment are shown in Figure 8 in terms of performance objectives matrix (seismic performance levels vs. earthquake hazard level). In particular, for each examined earthquake hazard level, seismic intensity zone, and soil condition, this Figure shows the range of the inter-story drift values (d / h) achieved varying the mass (m). From the obtained results, it can be observed that generally the performance exceeds the "enhanced objective" defined by the basic safety objective (FEMA 2000). In fact, the performance objectives matrix shows that for "low" intensity zones it is possible to associate the IO level with "frequent" and "rare" earthquakes and the LS level with "very rare" earthquakes. Instead, in the case of "medium" and "high" intensity zones it is possible to associate the IO level with "frequent" earthquakes and the LS level with "rare" and "very rare" earthquakes, except in the case of "high" intensity zone and "very soft" soil, in which for "very rare" earthquakes the drift limit value corresponding to CP is exceeded for the higher values of mass.

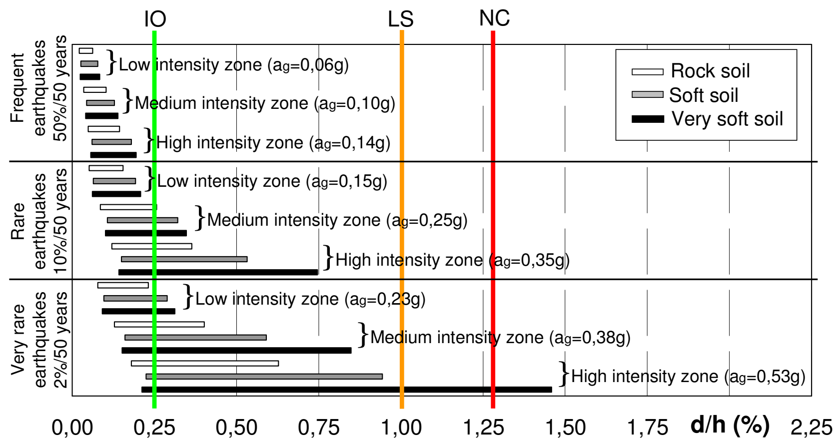

Figure 8. Performance objectives matrix.

Conclusions

In this paper, a model able to predict the nonlinear shear vs. top wall displacement relationship for steel frame/panel shear walls on the basis of screw connection test results has been presented. From a comparison between available experimental results and the numerical predictions obtained with both existing analytical approaches and the proposed model, the following conclusions can be drawn:

- The proposed and existing methods provide suitable prediction of the wall shear strength. All methods give a prediction with a scatter less than 5%, except for the Finnish timber Code's method.
- The analytical prediction of wall deflection is generally less accurate than the strength prediction (scatters larger than 15%).
- On the whole, the proposed model seems to give good results. In particular, in the examined case, strength, elastic and peak deflections are predicted with an error of -2%, +2% and -14%, respectively. In addition, the proposed model is also able to capture the softening branch of the structural response and the obtained results are slightly conservative (error equal to -7%). Being based on limited experimental data on connections and walls, these conclusions have to be considered as a preliminary outcome, which should be confirmed through a comparison with other test results.

The seismic performance assessment of a case study house performed through a nonlinear static analysis has been also presented as a possible application of the proposed model. The results of this application have shown that low rise residential buildings having steel frame / panel shear walls can be designed in such way to satisfy the main seismic performance objectives, as usually defined.

References

Branston, A. E., 2004. Development of a design methodology for steel frame / wood panel shear walls, *MSc. Thesis*, McGill University, Montreal, Canada.

Della Corte, G., Fiorino, L., Landolfo, R., 2006. Seismic behavior of sheathed cold-formed structures: numerical study. *J. Struct. Eng.* 132(4), 558-569.

Dubina, D., Fülöp, L. A., Aldea, A., Demetriu,,S., Nagy, Zs., 2006. Seismic performance of cold-formed steel framed houses, Proc. of the *5th International Conference on Behaviour of Steel Structures in Seismic Areas*, Yokohama, Taylor & Francis Group, 429-435.

Easley, J. T., Foomani, M., Dodds, R. H., 1982. Formulas for wood shear walls, *J. Struct. Division* 105(11), 2460-2478.

European Committee for Standardization (CEN), 1993. *Eurocode 5 - Design of Timber Structures – Part 1-1: General Rules and Rules for Buildings*, ENV 1995-1-1, Bruxelles.

European Committee for Standardization (CEN), 2005. *Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings*, EN 1998-1, Bruxelles.

Federal Emergency Management Agency (FEMA), 2000. *Prestandard and commentary for the seismic rehabilitation of buildings*, Rep. No. 350, Washington, D.C.

Fiorino, L., Della Corte, G., Landolfo, R., 2006a. Seismic Response of Steel Frame / Panel Shear Walls: Modelling based on Screw Connection Tests, Proc. of the *5th International Conference on Behaviour of Steel Structures in Seismic Areas*, Yokohama. Taylor & Francis Group, 503-510.

Fiorino, L., Landolfo, R., Della Corte, G., 2006b. Lateral Response of Sheathed Cold-Formed Shear Walls: An Analytical Approach. Proc. of the *18th International Specialty Conference on Cold-formed Steel Structures*, Orlando, Accepted for publication.

Fiorino, L, Landolfo, R., Della Corte, G., 2006c. Experimental tests on typical screw connections for cold-formed steel housing. Engineering Structures, in press, doi:10.1016/j.engstruct.2006.09.006

Hieta, J. and Kesti, J., 2002. *Design recommendations for shearwalls braced with sheathings*, Teräsrakenteiden tutkimus- ja kehityspäivät 13.-14.6.2002, Mikkeeli, Finnish Constructional Steelwork association.

International Building Code (IBC), 2000. *International Building Code: 2000*, Falls Church.

Kawai, Y., Kanno, R., Uno, N., Sakumoto, Y., 1999. Seismic resistance and design of steel-framed houses. *Nippon Steel Technical Report* 79, 6-16.

Landolfo, R., Fiorino, L., Della Corte, G., 2006a. Seismic behaviour of sheathed cold-formed steel structures physical tests, *J. Struct. Eng.* 132(4), 570-581.

Landolfo, R., Fiorino, L., Della Corte, G., 2006b. Shear behavior of connections between cold-formed steel profiles and wood or gypsum –based panels: an experimental investigation. Proc. of the *ASCE Structures Congress 2006*, St. Louis. Published by the American Society of Civil Engineers.

McCutcheon, W. J., 1985. Racking deformation in Wood Shear Walls, *J. Struct. Eng.* 111(2), 257-269.

OPCM, 2005. *Primi elementi in materia di criteri generali per la classificazione sismica del territorio nazionale e di normative tecniche per le costruzioni in zona sismica*. Ordinanza della Presidenza del Consiglio dei Ministri No.3431/2005, Rome.

Richard, R. M. and Abbott, B. J., 1975. Versatile elastic-plastic stress-strain formula. *J. Eng. Mech. Div.* 101(4), 511-515.

Uniform Building Code (UBC), 1997. *Uniform Building Code: Volume 2*, International Conference of Building Officials, Whittier, CA.