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Lateral Distortional Buckling Behaviour of a New Cold-formed Hollow Flange

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Lateral Response of Sheathed Cold-Formed Shear Walls: An Analytical Approach

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Abstract

An analytical approach allowing for a reliable evaluation of load vs. deflection response curve is a useful tool when the nonlinear response of a structural system needs to be evaluated. In this paper a method for predicting the nonlinear shear vs. top wall displacement relationship of sheathed cold-formed shear walls is proposed. The method relies on the availability of screw connection test results. The comparison of analytical results with available wall test results shows that the proposed approach can provide good prediction of both strength and wall deflection.

Introduction

Cold-formed/light gauge steel buildings typically use panel sheathing fastened to steel stud framing to provide an adequate lateral force resisting system. Reliability of the shear response evaluation of these systems is critical to the accuracy of response prediction under horizontal actions.

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 used one. In fact, nominal shear strength design values given by building codes (UBC 1997, IBC

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2003) are based on experimental test results. This approach is the most expensive and it 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 sheathed cold-formed shear walls. On the contrary, a large number of methods developed for the analysis of sheathed wood 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 methods for wood-framed walls is reasonable also in the case of steel-framed walls.

The possibility to give predictions of 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 method allowing the prediction of the whole nonlinear shear vs. top wall displacement relationship is proposed.

Existing methods for deflection prediction

The following analytical methods for deflection prediction of sheathed wood-framed shear walls are considered in this study: McCutchenon (1985), Easley et al. (1982), ENV 1995-1-1 (1993), Finnish timber Code RIL 120-2001 (Hieta and Kesti 2002). All these methods have common assumptions, which can be synthesised as follows: (a) local failure of sheathing-to-wall framing connections governs the global collapse mode; (b) studs and tracks are rigid and hinged to each other; (c) panels are rigid or panels shear strain only is considered; (d) relative displacements between the sheathing and framing are small compared with the panel size; (e) the edges of the panel are free to rotate without

interference from adjacent sheathings and the foundation or other stories; (f) the wall is fully anchored to the foundation or lower storey.
Moreover, each method formulates additional hypotheses concerning assumptions especially on the wall deformation, force distribution and connection load-deflection relationship.

Using the same energy approach illustrated in Tuomi and McCutcheon (1978), McCutchenon (1985) presented a general method for the evaluation of the racking deformation of wood shear walls for moderate load levels (design load levels). This approach takes into account the nonlinear behavior of connections by schematizing its load vs. displacement response curve through a power function. As a result, the racking response of the wall was also defined by a power curve. By comparing theoretical and test results the writer concluded that the estimation of the racking response was accurate up to moderate levels of deformation, but the method underestimated displacements at higher loads.

Based on experimental results of 8 wall tests, Easley et al. (1982) proposed a nonlinear formula for wall deformation assessment. In particular, the writers assumed a linear force distribution for the connections. In addition, they assumed that: all the fasteners are identical and studs and nails are symmetrically located about the sheathing centre line. An empirical four-parameters response curve was adopted for simulating the response in the nonlinear range. From comparison between experimental and analytical results the writers concluded that the results obtained applying the proposed formulas were accurate enough for engineering practice.

Eurocode 5 (ENV 1995-1-1 1993) suggests an “elastic method” to predict the lateral deformation of wood shear walls (Kallsner and Lam 1995). In this method, the following assumptions are made: sheathings are rigid; the central points of the frame and the sheathings have the same displacement, only relative rotation exists; the load-deflection curve of sheathing-to-wall framing connections is linear. Based on the minimum potential energy principle, the writers found a relationship for the evaluation of total horizontal displacement.

In the Finnish timber Code RIL 120-2001 (Hieta and Kesti 2002) a formula for the evaluation of the horizontal deflection is given. Differently for other mentioned methods, the Finnish calculation method allows the deflection of a shear wall to be calculated when the uplift of the wall corner is not prevented.

As far as the numerical models is concerned, significant studies were developed by Foschi (1977), Dolan (1989), Dolan and Foschi (1991), and White and Dolan (1995). Foschi (1977) presented a numerical modeling procedure, named SADT, for the structural analysis of wood diaphragms. The model considers four structural components: the sheathing, which was assumed as an elastic and orthotropic material subjected to two dimensional state of stress; framing members, which were idealized as linear beam elements; framing connections, which were schematized as nonlinear springs; sheathing-to-frame connections, which were modeled with two-degree of freedom springs whose load vs. displacement response curve is schematized through an exponential function. Based on the comparison with test results, the author showed that the model was accurate in the deformation and ultimate load prediction.

Based on experimental results of 42 wall tests, different models capable of predicting the behavior of wood shear walls were presented by Dolan (1989), Dolan and Foschi (1991), White and Dolan (1995). The FREWALL model (Dolan 1989) consisted in a closed form analytical model developed to predict the dynamic response of walls subjected to harmonic excitations. The models SHWALL and DYNWALL (Dolan and Foschi 1991) are finite element models dedicated to predict the static and dynamic response of walls subjected to earthquakes, respectively. These finite element models are based on the SADT model, upgraded as follows: the adjacent panels can contact each other; the effect of bearing and gap formation between framing members is included. A further improvement is represented by the program WAlSEIZ (White and Dolan 1995), in which the following modifications were included: reduction of degrees of freedom in the plate and connection elements to reduce the analysis time; capability for performing both static and dynamic analysis; capability for calculating forces and stress; possibility for analyzing larger walls with and without openings. By comparing numerical and test results, the authors concluded that the program WAlSEIZ predicted the maximum strength of a wall subjected to monotonic loading to within 2%, and correlated well with dynamic test results.

The proposed method

Based on results of experimental tests on two, nominally identical, cold-formed stick-built house sub-assemblages, a method for prediction of load vs. deflection response curve of sheathed steel-framed shear walls is proposed and illustrated hereafter. The method is based on the observation of the deformation pattern during the tests. In addition to the basic assumptions of the illustrated analytical models for wood shear walls, the following hypotheses are made in the proposed approach: (a) the wall framing deforms into a parallelogram and the relative frame-to-panel displacements are determined based on a rigid body rotation of panels; (b) only shear deformation of the sheathings is considered by adopting the equation for shear deformation of a thin, edge-loaded, plate; (c) 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.

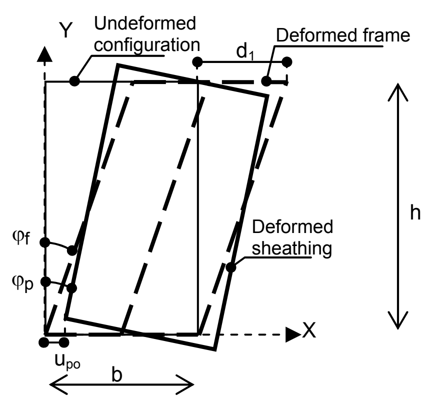

Figure 1. Assumed deformation of shear wall.

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

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

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:

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

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:

$$F_{x,i}=k_{x,i}u_i\ ,\ F_{y,i}=k_{y,i}v_i \quad (3)$$

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):

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

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

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

in which:

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

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

$K_e = k_{xe} n_e$; $S_e = k_{xe} n_e h$; $I_e = k_{xe} n_e h^2$.

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. The numerical procedure, whose main phases are shown in Figure 2, is presented in the following.

For a generic step s and a generic iteration j, assigned a top wall displacement increment ($\Delta d_1^{(s)} = \Delta\varphi_t^{(s)} h$), the increment of horizontal external force per unit of length ($\Delta V^{(s,j)}$) and the increment of rotation ($\Delta\varphi_p^{(s,j)}$) and translation ($\Delta u_{p0}^{(s,j)}$) of the panel can be evaluated by Equation 4 through 6, which can be summarized as follows:

$$\Delta V^{(s,j)} = f_1\left(\Delta\varphi_t^{(s)},..,k_{x,i}^{(s,j-1)},..,k_{y,i}^{(s,j-1)},..\right) \tag{8}$$

$$\Delta\varphi_p{}^{(s,j)} = f_2\left(\Delta V^{(s,j)},..,k_{x,i}^{(s,j-1)},..,k_{y,i}^{(s,j-1)},..\right) \quad (9)$$

$$\Delta u_{p0}{}^{(s,j)} = f_3\left(\Delta V^{(s,j)},..,k_{x,i}^{(s,j-1)},..,k_{y,i}^{(s,j-1)},..\right) \quad (10)$$

where $k_{x,i}^{(s,j-1)}$ and $k_{y,i}^{(s,j-1)}$ (with i ranging from 1 to n) are the stiffness of the generic connection i along X and Y directions, respectively. These stiffnesses are obtained from iteration j-1.

The increment of relative displacements between the framing member and panel are obtained from Equations 1 and 2:

$$\Delta u_i^{(s,j)} = \left(\Delta\varphi_p^{(s,j)} - \Delta\varphi_t^{(s)}\right)y_i - \Delta u_{p0}^{(s,j)}\ ;\ \Delta v_i^{(s,j)} = \Delta\varphi_p^{(s,j)}\left(b/2 - x_i\right) \quad (11)$$

while the increments of force components of connections can be found by Equations 3:

$$\Delta F_{x,i}^{(s,j)} = k_{x,i}^{(s,j-1)}\Delta u_i^{(s,j)},\ \Delta F_{y,i}^{(s,j)} = k_{y,i}^{(s,j-1)}\Delta v_i^{(s,j)} \quad (12)$$

Finally, force components of connections can be also derived from the connection load-displacement curve:

$$\overline{F}_{x,i}^{(s,j)} = g\left(u_i^{s,j}\right),\ \overline{F}_{y,i}^{(s,j)} = g\left(v_i^{s,j}\right) \quad (13)$$

At this stage, assuming that in the previous step s-1 l iterations have been carried out, the following check is made:

$$\overline{F}_{x,i}^{(s,j)} - F_{x,i}^{(s,j)} = \overline{F}_{x,i}^{(s,j)} - \left(F_{x,i}^{(s-1,l)} + \Delta F_{x,i}^{(s,j)}\right) \le \varepsilon \quad (14)$$

$$\overline{F}_{y,i}^{(s,j)} - F_{y,i}^{(s,j)} = \overline{F}_{y,i}^{(s,j)} - \left(F_{y,i}^{(s-1,l)} + \Delta F_{y,i}^{(s,j)}\right) \le \varepsilon \quad (15)$$

where $F_{x,i}^{(s-1,l)}$ and $F_{y,i}^{(s-1,l)}$ are the force components of connections found in the step s-1; ε is the iteration convergence tolerance.

If Equations 14 and 15 are true for i ranging from 1 to n, then the solution converges at iteration j, else the iteration j+1 is carried out assuming the following stiffnesses:

$$k_{x,i}^{(s,j)} = \frac{\overline{F}_{x,i}^{(s,j)} - F_{x,i}^{(s-1,l)}}{\Delta u_i^{(s,j)}},\ k_{y,i}^{(s,j)} = \frac{\overline{F}_{y,i}^{(s,j)} - F_{y,i}^{(s-1,l)}}{\Delta v_i^{(s,j)}} \quad (16)$$

The load vs. deflection response curve of the wall framing obtained with this procedure is based on the assumption that the panel has rigid body rotation. The deflection due to the shear deformation of the panel can be added as illustrated in the case of connections with linear response (see Equation 7).

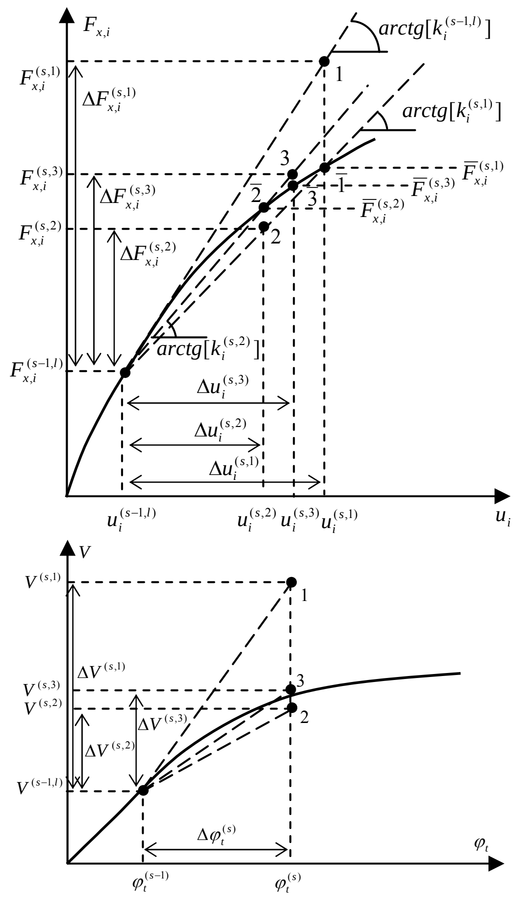

Figure 2. Numerical procedure.

Model calibration

A preliminary calibration of the load vs. deflection response curve prediction obtained applying the proposed method 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. 2006).

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: peak strength, is the maximum recorded load;
- s_p: peak displacement, is the displacement corresponding to F_u;
- $k_p=F_u/s_p$: peak secant stiffness
- $F_e=0.4F_u$: conventional elastic strength;
- s_e: conventional elastic displacement, is the displacement corresponding to F_e;
- $k_e=F_e/s_e$: conventional elastic secant stiffness;
- s_u: conventional ultimate displacement, is the 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. 3):

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

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

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 behavior (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:
  - conventional elastic point (s_e, F_e);
  - peak point (s_p, F_u);
  - 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 (A_1=A_2) for 0≤ s ≤s_p.

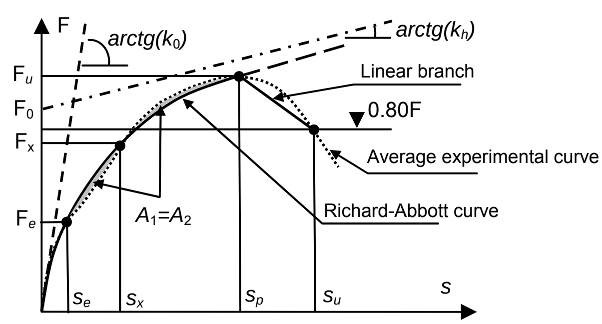

Figure 3. Analytical schematization of connection *F*-*s* response curve.

Because experimental results on walls are relevant to monotonic load condition in which displacements were 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 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 preliminar study (Fig. 4), 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.

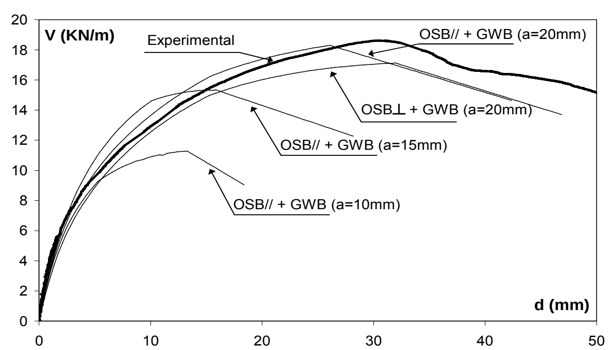

Figure 4. Calibration of the proposed method.

Comparison with existing methods

To assess the reliability of the proposed method, 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 (ENV 1995-1-1 (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:
- for evaluating the shear response of walls the contribution of OSB and GWB panels are added;
- the shear modulus of elasticity of the sheathings is 1400MPa for OSB panels and 750MPa for GWB panels;
- 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 5, in which also experimental connection responses are reported. Figure 6 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 analytical method gives a result which seems accurate enough in comparison with the experimental response. In particular, the proposed method 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 6 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.

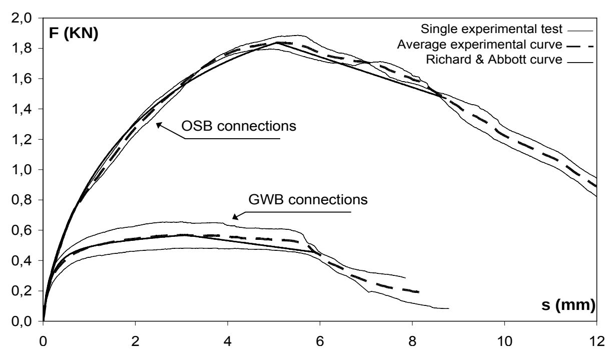

Figure 5. Load-displacement curves of sheathing-to-frame connections.

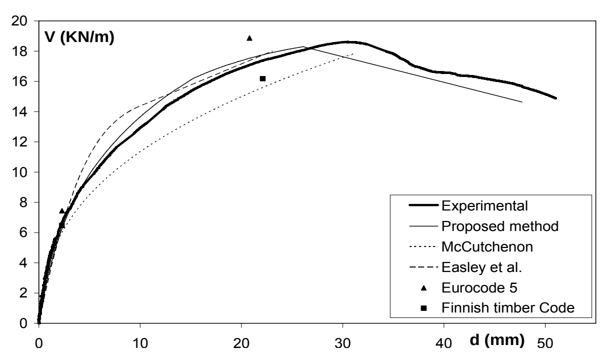

Figure 6. Load vs. deflection curve of examined wall: Experimental vs. analytical response.

To assess the reliability of strength prediction, the comparison between the predicted-to-test peak load ratios ($\rho_V = V_{u,ana}/V_{u,exp}$) obtained by considering all applied methods reveals that the best strength prediction is given by Eurocode 5 (ρ_V=1.01) and the proposed method (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 analytical 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 method ($\rho_{de} = d_{e,ana}/d_{e,exp}$=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 ($\rho_{dp} = d_{p,ana}/d_{p,exp}$=1.03). An almost accurate result is obtained also with the proposed method (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 $0.80V_u$ on the post-peak branch of response) is concerned, its predictions it is possible only with the proposed method. In fact, only this method 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).

Conclusions

From comparisons between available experimental results and predictions obtained with both existing analytical approaches and the proposed method, the following conclusions can be drawn:

- Proposed and existing analytical methods provide suitable prediction of the shear strength. All methods give prediction with an error less than 5%, except for the Finnish timber Code's method.
- The analytical prediction of wall deflection is not accurate as strength prediction (scatters larger than 15%, exception made for the McCutchenon's method for the prediction of peak deformation, Easley et al.'s method for elastic deformation prediction and the proposed method for both).
- On the whole, the proposed method 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 method is able to capture also the post-peak branch of response and the obtained results are slightly conservative (error equal to -7%). Being based on limited experimental data on connections and walls, this conclusion should be considered as a preliminary outcome, which should be confirmed through a comparison with other test results.

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