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THE INFLUENCE OF THE ASPECT RATIO ON THE LATERAL RESPONSE OF SHEATHED COLD FORMED STEEL WALLS

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ABSTRACT

The influence of the aspect ratio on the lateral response of cold formed steel walls is analyzed by three design methodologies. In particular the prediction provided by the AISI Lateral Design, that is at the moment the main document for the design of CFS buildings under horizontal loads, is compared with the results obtained by applying the principles of mechanics and with those provided by non-linear finite element models. This paper presents and discusses in terms of strength and stiffness the validity of the different design methodologies in case of non conventional wall aspect ratios comparing the numerical results with available experimental data.

Introduction

The adoption of cold-formed steel (CFS) buildings is spreading all over the world. The growing structural confidence with this construction system is allowing also complex architectural requirements to be satisfied. Therefore, often unconventional dimensions in plan and elevation are adopted. Since, the seismic behaviour of these structures is strongly influenced by the lateral response of shear walls, the influence of different wall aspect ratios on the seismic response is a concern.

Different approaches are available to calculate the lateral response of sheathed CFS walls: tabulated, numerical and analytical methodologies. The tabulated approaches are based on the results of full scale tests on typical walls and their application is possible only when the wall characteristics (geometry and materials) are within the range of experimental results. In order to overcome the limitations of this approach, finite element methods may be used to evaluate the lateral response of CFS walls. Few analytical methods specifically developed for CFS structures exists, but they have not yet been included in any code.

This work compares the results in terms of strength and stiffness of three methodologies for the prediction of CFS lateral wall response. In particular, the tabulated approach provided by the AISI Lateral Design, that represents the main document for the design of CFS buildings under horizontal loads, is compared with the results given by the principles of mechanics and with those obtained by non-linear finite element models, specifically developed for walls having different aspect ratios.

Design based on the principles of mechanics

The lateral response of a SCFS shear wall can be evaluated by principles of mechanics considering the behavior of its structural components: sheathing-to-frame connection, sheathing panels, frame-to-foundation anchors and CFS frame (Landolfo et al., 2010, Fiorino et al., 2009). In this methodology the wall lateral resistance is given by the strength associated to the weakest failure mechanism of the walls components. Therefore, for each component can be defined the failure mechanism and the smallest associated strength value defines the wall resistance:

$$H = \min\,(H_{c,f}, H_{c,p}, H_{c,ha}, H_{c,s}) \qquad (1)$$

where H is the wall average resistance and H_f, H_p, H_{ha}, H_s are the wall average resistances associated to the failure mechanism of sheathing-to-frame connection, sheathing panel, frame-to-foundation anchors and steel frame, respectively. The resistance associated to the sheathing-to-frame connections can be evaluated by different methods, in this paper, the Hieta & Kesti (2002) for timber shear walls approach has been used, in which the lateral resistance of wall due to connection (H_f) is based on maximum connection force in the panel corner:

$$H_f = n \frac{F_V}{\gamma \cdot c} b \qquad (2)$$

where F_V is the strength of single connection between sheathing panel and steel frame, which can be experimentally determined; n is the number of panel connected to the frame; b is the panel width; c is the fastener spacing; γ is a coefficient which depends on the h/b ratio and h is the wall height. The failure of sheathing panel is generally due to shear and the corresponding wall resistance (H_p) is the lateral load which induces ultimate shear stress in the sheathing panel. In case of wood based panel the resistance can be obtained by the formula given by EN 1995-1-1 (2004):

$$H_p = n k_{mod} \cdot f_{p,V} t_p L \qquad (3)$$

where k_{mod} is the modification factor due to duration of load and moisture content assumed equal to 0.70; $f_{p,V}$ is the shear strength of panel material; t_p is the thickness of the panel and L is the wall length.

The wall steel frame is generally anchored to the foundation by hold-down devices placed at the end of the wall which resist to the uplift force due to the applied lateral load. Therefore, the resistance associated to the failure of frame-to-foundation anchors (H_s) is the lateral load which corresponds the axial resistance of this anchorage:

$$H_{ha} = \frac{N_{ha}}{h} L \tag{4}$$

where N_{ha} is the tension resistance of the anchorage. The steel frame failure under lateral load is usually governed by the buckling failure of the end stud in compression and the corresponding wall resistance (H_s) is given by:

$$H_s = \frac{N_s}{h} L \tag{5}$$

where N_s is the buckling resistance of the end stud.

According to this methodology, the lateral wall displacement can be obtained by adding the different deformation contribution of wall components individually calculated ($d = d_f + d_p + d_{ha} + d_s$). Therefore, the wall lateral stiffness can be evaluated by the following formula:

$$K = \frac{1}{\frac{1}{K_f} + \frac{1}{K_p} + \frac{1}{K_{ha}} + \frac{1}{K_s}} \tag{6}$$

where K_f, K_p, K_{ha}, and K_s are the stiffness contributions due to sheathing-to-frame connection, sheathing panel, frame-to-foundation anchors and steel frame, respectively.

The wall stiffness contribution of sheathing-to-frame connections can be evaluated by different formulations. In this paper, as well as for resistance, the relationship proposed by Hieta & Kesti (2002) has been used:

$$K_f = n \cdot \frac{k_{f-s}}{\beta \cdot c} \cdot \frac{b^3}{h^2} \tag{7}$$

where k_{f-s} is the stiffness of a single connection in shear, which can be obtained from experimental tests, and β is a coefficient which depends on the h/b ratio.

The wall stiffness due to the sheathing panels is obtained by considering the panels as a thin, edge-loaded, plate in shear:

$$K_p = n \cdot \frac{G \cdot t_p \cdot b}{h} \tag{8}$$

where G is the shear modulus of elasticity of the panel material.

The wall stiffness contribution due to hold-down devices is calculated from the following equation:

$$K_{ha} = \frac{k_{hd} \cdot L^2}{h^2} \tag{9}$$

where k_{hd} is the axial stiffness of the hold-down device given by manufacturers.

The wall stiffness due to the steel frame can be evaluated by considering it as a cantilever having a cross-section made of the only end studs:

$$K_s = \frac{3E \cdot A \cdot L^2}{2h^3} \tag{10}$$

where E is the Young's modulus of steel, A is the gross cross-sectional area of an end stud.

Design according to AISI lateral design recommendations

The AISI lateral design S213-07/S1-09 (AISI S213-07/S1-09, 2009) represents the main document for the design under lateral forces of buildings with CFS framing. In this standard sheathed CFS shear walls are classified in two categories: "Type I" and "Type II" shear walls. "Type I" shear walls are fully sheathed and are provided of hold-down anchors at each end of wall. The openings are permitted only if specific details to transfer the forces around the openings are provided. On the other hand, for "Type II" shear walls openings are permitted without particular details and the wall resistance is evaluated as the wall resistance without opening multiplied by an adjustment factor which depends on the opening shape.

The AISI lateral design provides in tables the resistance values for wind, seismic and other in-plane loads for walls with different types of sheathing and screw spacing. In particular, the nominal resistances (R_n) of walls sheathed on one side based on experimental test results are provided in tables (Table 1). The provided resistance values can be used only for walls consistent with fixed limitation such as maximum aspect ratio, stud thickness, steel grade and screw size. The tabulated nominal resistance values are valid for aspect ratios (h/L) up to 2, while, greater values, but not exceeding 4, can be used starting from nominal resistance values and multiplied by the reduction factor equal to $2L/h$. For walls with same type of sheathing on both sides, the nominal resistance is cumulative, while for walls sheathed with two different materials, the nominal resistance is either two times that of the sheathing with the smallest value or that of the strongest side. According to the code, the evaluation of wall deflection is based on a simple model corrected by empirical factors to account the inelastic behavior. The model assumes that the total deflection is the sum of four basic contributions: linear elastic cantilever bending, linear elastic sheathing shear, nonlinear lateral deflection due to fastener deformation and linear elastic lateral contribution of anchors deformation. The wall deflection can be calculated by the following equation:

$$d = \frac{2vh^3}{3EA\,b} + \omega_1\omega_2 \frac{vh}{\rho G t_{sheathing}} + \omega_1^{5/4}\omega_2\omega_3\omega_4\left(\frac{v}{\beta}\right)^2 + \frac{h}{b}\delta_v \qquad (11)$$

where s is the maximum spacing at the panel edges, $t_{sheathing}$ is the sheathing thickness, v is the lateral load per unit length acting on the wall, β is a coefficient depending on sheathing material, δ_v is the vertical deformation of anchors, ρ is a coefficient depending on sheathing material, ω_1 is equal to $s/152.4$ with s in mm, ω_2 is equal to $0.838/t_{stud}$ with the stud thickness t_{stud} in mm, ω_3 is a coefficient depending on aspect ratio (h/b), ω_4 is a coefficient depending on sheathing material. This equation cannot be used beyond the nominal resistance values provided by the code.

Assembly Description	Max. Aspect Ratio (h/w)	Fastener Spacing at Panel Edges[2] (mm)			Designation Thickness[5,6] of Stud, Track and Blocking (mils)	Required Sheathing Screw Size
		150	100	75		
9.5 mm CSP Sheathing	2:1[3]	8.5	11.8	14.2	43 (min.)	8
12.5 mm CSP Sheathing	2:1[3]	9.5	13.0	19.4	43 (min.)	8
12.5 mm DFP Sheathing	2:1[3]	11.6	17.2	22.1	43 (min.)	8
9 mm OSB 2R24/W24	2:1[3]	9.6	14.3	18.2	43 (min.)	8
11 mm OSB 1R24/2F16/W24	2:1[3]	9.9	14.6	18.5	43 (min.)	8

Table 1. Nominal wall resistance for SCFS walls sheathed with wood-based panels (AISI S213-07/S1-09, 2009)

Design based on finite element models

In order to overcome the limitation of tabulated design procedures, finite element models can be developed. On this purpose, non linear finite element models, that are able to reproduce the response in terms of strength and stiffness of available experimental tests on full scale walls have been carried out.

In particular, finite element models (FEM) have been developed and calibrated by using the SAP 2000 v. 14 software on the base of two full scale wall tests presented in Iuorio et al. (2012). Two identical 4.80 m long and 3.95 m height sheathed CFS walls have been tested under vertical and horizontal loads (Fig. 1). In particular, CFS frame have been made with 150×50×20×1.50 mm lipped channel studs spaced at 600 mm and sheathed with 9 mm thick OSB/3 panels on both side. The sheathing-to-frame connections have been realized with 4.2 mm flat head self-drilling screws spaced at 100 mm at panel edges and at 300 mm on the internal studs.

“Back-to-back” coupled studs have been placed at the wall ends and, at the same location, purposely designed hold-down devices made with S700 steel grade and anchored to the concrete foundation by HILTI (2008) HIT-RE 500+HAS-E(5.8)-M24 have been placed. Shear connections between the steel frame and the concrete foundation have been provided by HILTI (2008) HST-M8 anchors spaced at 200 mm. In order to prevent any acoustic noise transmission, an insulation pad has been placed between the bottom track and the concrete foundation. The walls have been subjected to vertical loads equal to 5.92 kN and 10.20 kN for the first and second test, respectively. lateral loads have been applied to the top of the walls by a double effect jack. The specimens were tested under two different loading protocols, characterized by a first cyclic history followed by a second monotonic sequence. In the first phase, cyclic displacements up to 9 and 13 mm in the first and second test have been respectively impressed to the walls. In the latter phase, the specimens were monotonically loaded up to the collapse condition. In both tests the collapse has been due to the sheathing-to-frame connection failure.

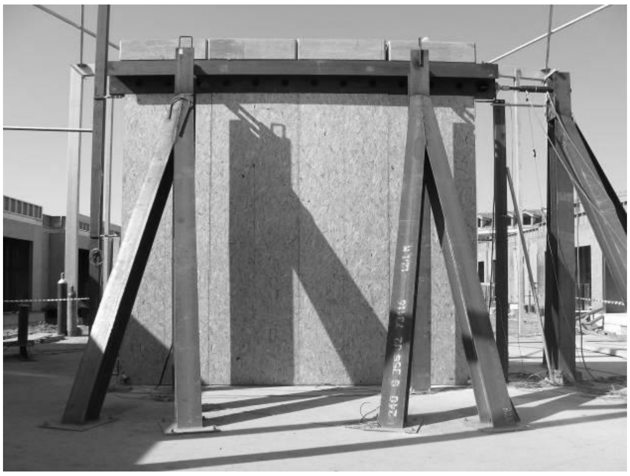


Figure 1: Full scale wall test.

As far as the model is concerned, the steel members have been modeled by frame elements, with linear elastic material having Young modulus equal to E=210000 MPa and Poisson’s ratio equal to ν_p=0.3. The mesh dimension of these elements is equal to 50 mm. The OSB panels have been modeled with thin shell elements having 50x50 mm rectangular mesh. A linear mechanical model has been assumed for the OSB panels characterized by a shear modulus G=1134 MPa and Poisson’s ratio ν_p=0,29. The sheathing-to-frame connections have been modeled by multilinear elastic links with a force-displacement relationship defined according to available experimental data and having peak strength and conventional elastic stiffness equal to 1.32 kN and 0.90 kN/mm, respectively (Fig. 2).

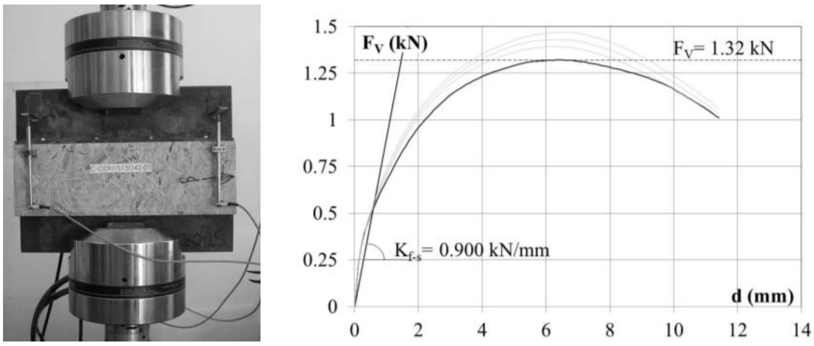


Figure 2: Adopted sheathing to frame connection curve.

The behavior of the wall-to-foundation anchors, in the described wall tests has been influenced by the presence of an acoustic insulation pad, that produced an unforeseen slip during the tests. Therefore, the hold down devices, located at the bottom track ends, have been simulated by elastic springs having vertical stiffness equal to 30000 N/mm and horizontal stiffness equal to 450 N/mm. The shear anchors, placed on the bottom wall track and spaced each 200mm, have been schematized by elastic springs in both horizontal and vertical directions having stiffness equal to 450 N/mm and 9000N/mm, respectively. The stiffness values of hold-down and shear anchors have been obtained starting from the horizontal and vertical displacements recorded by the LVDTs placed, during the tests, at the wall bottom. The geometrical and mechanical characteristics of all the structural components are reported in Figure 3 and in Table 2.

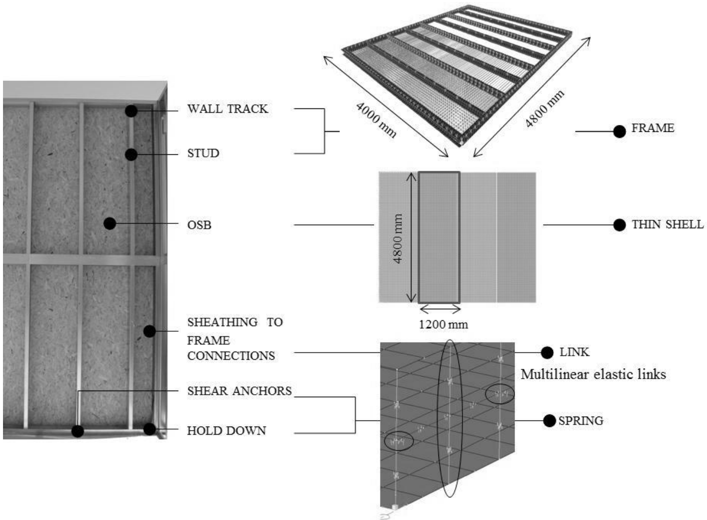


Figure 3: Numerical modeling.

All the modeled joints have been restrained to avoid any out of plane displacement. The connections between studs and tracks have been schematized as hinges. Finally a rigid body constraint to the top track has been applied. As far as the loads are concerned, on the top track distributed vertical loads equal to those applied in the tests have been assigned. The lateral actions have been simulated by a concentrated horizontal force applied to the top track. The intensity of this force gradually rises during the analysis, so that a static pushover analysis under controlled displacement has been carried out. The numerical models have been calibrated on the basis of both the deformation of the whole structure and the slip and the up-lift displacements recorded at the base of the tested walls.

	DIMENSIONS	FEM	MECHANICAL PROPERTIES
COLD FORMED PROFILES			
STUD	C 150x50x20x1.5mm Length 3950mm	Frame	S350GD+Z150 Hot dip galvanized steel E=210000MPa ν=0.3
TRACK	U 150x50x1.5mm (Length 4800mm)	Frame	
SHEATHING PANELS			
OSB TYPE 3	3950x1200x9.0mm	Thin Shell	G=1134MPa ν=0.29
CONNECTIONS and ANCHORS			
HOLD-DOWN and TENSILE ANCHORS	Hold Down devices: Purposely design Anchors: HIT-RE 500 + HAS-E (5.8)-M24	Spring	k_x=450 N/mm k_y= 30000 N/mm
SHEAR ANCHORS	HST-M8	Spring	k_x=450 N/mm k_y=9000N/mm
SHEATHING – TO - FRAME CONNECTIONS	CH 01 42 025 flat head self-drilling screws 4,2x25mm (Diameter x length)	Link	F_V = 1.32kN $k_{f\text{-}s}$= 0.90kN/mm

Table 2: Geometrical and mechanical properties of the wall components.

The numerical results have shown that the wall lateral response is not influenced by the vertical load, therefore the same numerical response curve has been obtained for both wall tests. The comparison between experimental and numerical results is shown in figure 4 in terms force (H) vs. top displacement (d) response curve. The numerical comparison for wall strength (H) and conventional elastic stiffness (K) is shown in table 3.
The model is able to reproduce accurately the tests. As shown in Figure 4, the numerical results in terms of wall strength are 3% and 17% lower than those obtained by the first and second test, respectively.

Instead, in terms of stiffness, the numerical value is 46% lower than that recorded in the first test and it is 5% higher than the one obtained in the second test. The difference strength between the two tests can be explained by the wider number of cycles and corresponding displacement that have been impressed to the specimens in the first phase of the of the second test. Figure 5 shows that the numerical results are in good agreement with the experimental ones.

	H_{ex} [kN]	H_{fem} [kN]	H_{ex}/H_{fem}	K_{ex} [kN/mm]	K_{fem} [kN/mm]	K_{ex}/K_{fem}
Test 1	147.5	152.4	0.97	8.24	5.63	1.46
Test 2	127.6	152.4	0.83	5.33	5.63	0.95

Table 3: Comparison between experimental and numerical results.

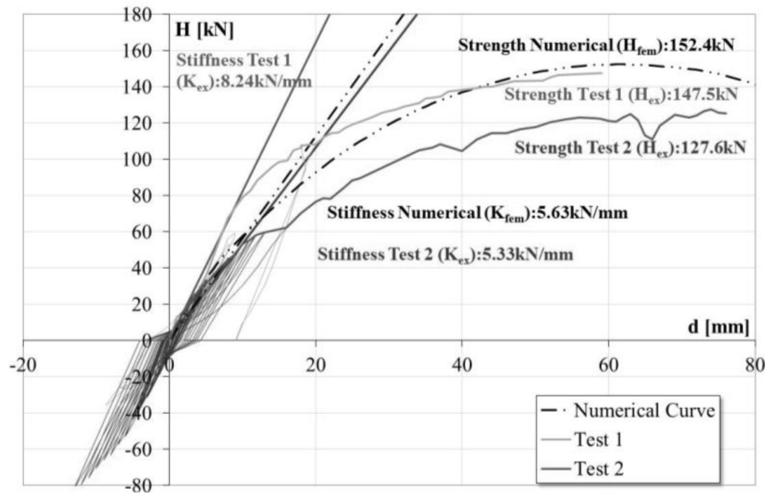


Figure 4. Comparison between experimental and numerical curves.

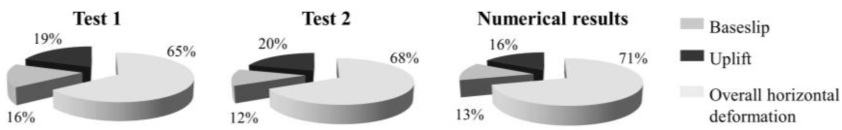


Figure 5. Comparison between experimental and numerical deformations.

Influence of wall aspect ratio on lateral response

In order to investigate the influence of aspect ratio on wall response, lateral strength and stiffness of different wall configurations have been calculated according to three different methodologies: principles of mechanics, AISI

lateral design, FEM. All the walls were obtained by varying wall length from 1.2 to 24.0 meter. The other characteristics of the wall as height, sheathing typology, connections and type of anchors, are the same as assumed in the model previously presented.
In terms of resistance (Fig. 6), the three investigated methodologies provide very similar values. For wall with aspect ratios lower than two in fact the resistances per unit length evaluated according to the principles of mechanics are constant and coincide with those calculated according to the AISI recommendations, while the ones calculated by numerical simulation are slightly higher. On the contrary, for aspect ratios ranging between 2 and 4, AISI Lateral Design reduce the resistance by a factor equal to 2L/h, which produces a resistance decreasing up to 54% with respect the other methodologies results. The different strengths calculated using the above mentioned methodologies are reposted in Table 4.

			Principles of mechanics		AISI Lateral		FEM	
L [mm]	*h* [mm]	*h/L*	*H* [kN]	*H/L* [kN/m]	*H* [kN]	*H/L* [kN/m]	*H* [kN]	*H/L* [kN/m]
1200	4000	3.33	35	0.029	21	0.017	36	0.030
1800	4000	2.22	52	0.029	46	0.026	51	0.028
2400	4000	1.67	70	0.029	69	0.029	72	0.030
3600	4000	1.11	105	0.029	103	0.029	107	0.030
4800	4000	0.83	140	0.029	137	0.029	152	0.032
8400	4000	0.48	244	0.029	240	0.029	251	0.030
9600	4000	0.42	279	0.029	275	0.029	287	0.030
24000	4000	0.17	698	0.029	686	0.029	716	0.030

Table 4. Strength values calculated by the investigated methodologies.

The stiffness per unit length, calculated by the three methodologies, decreases with increasing of the aspect ratio, as shown in Figure 7.
For the numerical results the comparison in terms of stiffness has been made without considering the deformation contribution of the wall due to the base-slip. For aspect ratios greater than 1 the stiffness values obtained by the principles of mechanics and AISI recommendation are very similar, while those obtained with the FEM models are about 30% lower than the previous ones. Instead in case of lower aspect ratios the values obtained with principles of mechanics and FEM are very similar, while those provided by AISI Lateral Design are very higher. The different stiffness calculated using the above mentioned methodologies are reposted in Table 5.

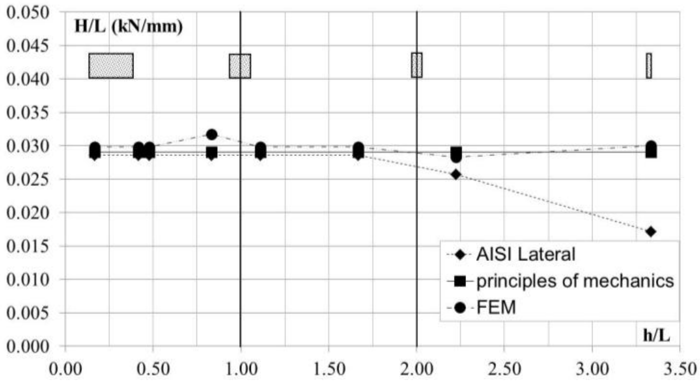


Figure 6. Comparison in terms of resistance.

			Principles of mechanics		AISI Lateral		FEM	
L [mm]	*h* [mm]	*h/L*	*K* [kN/m]	*K/L* [kN/m/mm]	*K* [kN/m]	*K/L* [kN/m/mm]	*K* [kN/m]	*K/L* [kN/m/mm]
1200	4000	3.33	985	0.821	1078	0.898	748	0.624
1800	4000	2.22	1793	0.996	1820	1.011	1403	0.779
2400	4000	1.67	2677	1.115	2781	1.159	2338	0.974
3600	4000	1.11	4562	1.267	5383	1.495	4268	1.186
4800	4000	0.83	6527	1.360	8519	1.775	6459	1.346
8400	4000	0.48	12605	1.501	2034	2.422	12677	1.509
9600	4000	0.42	14659	1.527	2493	2.597	14801	1.542
24000	4000	0.17	39568	1.649	9601	4.000	41487	1.729

Table 5. Stiffness calculated by investigated methodologies

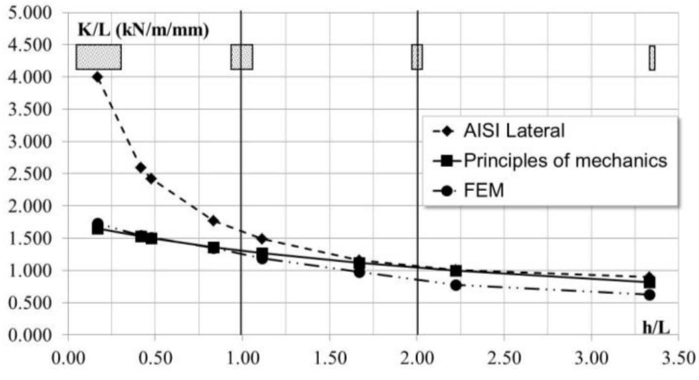
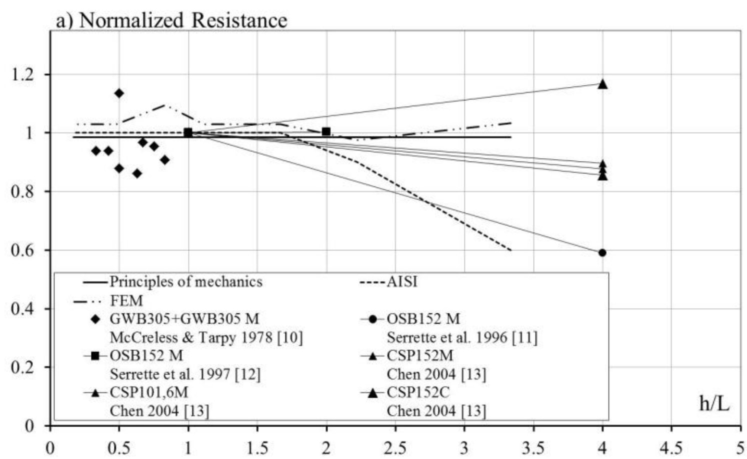


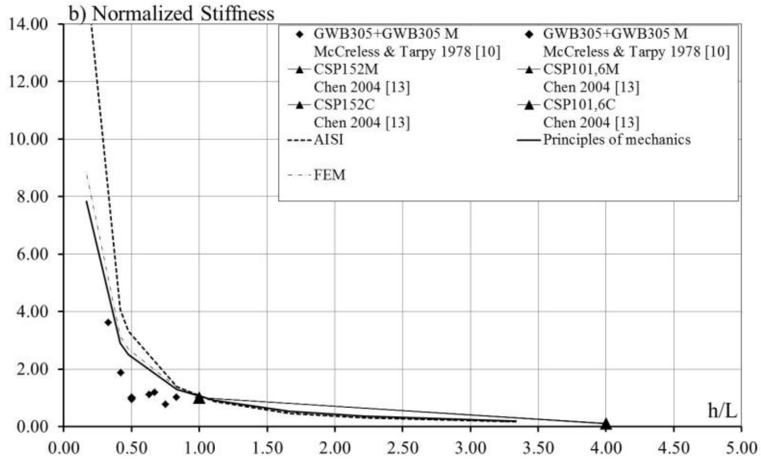
Figure 7. Comparison in terms of stiffness

In order to validate the results obtained by applying the design methodologies and FEM, a comparison with experimental literature data has been carried out. The experimental data have been selected from researches devoted to investigate the effect of aspect ratio for walls sheathed with wood-based or gypsum panels: McCreless & Tarpy (1978), Serrette et al. (1996) Serrette et al. (1997) and Chen (2004).

The comparison in terms of resistance and stiffness is illustrated in Figure 8 a and b, respectively. In order to compare consistent results, the values obtained in this paper and the literature data have been normalized with regards of those corresponding to an aspect ratio equal to 1. In terms of resistance it can be noticed that for aspect ratios lower than 1, the experimental evidence (McCreless & Tarpy, 1978) is in agreement with the results of the all considered methodologies and, therefore, the resistance trend can be considered uniform. Moreover, the reduction proposed by AISI for aspect ratios greater than 2 is fully supported by the experimental data given in Serrette et al. (1996) and Serrette et al. (1997) and only partially in Serrette et al., 1997, but it appears too conservative.

In terms of stiffness, for aspect ratios less than 1, the experimental results given in McCreless & Tarpy (1978) do not confirm the high values obtained by AISI methodology. For aspect ratios greater than 1, the stiffness trend given by the three methodologies is supported by the data given in Chen, 2004.





GWB: Gypsum Wallboard; OSB: Oriented Strand Board; CSP: Canadian Softwood Plywood; M: Monotonic test; C: Cyclic test

Figure 8. Comparison between numerical and available experimental data in terms of : *a*) resistance; b) stiffness.

Conclusions

Three methodologies to calculate the wall lateral response have been presented: AISI recommendations, a method based on principles of mechanics and non-linear finite element models. The results provided by the described methodologies have been compared for different aspect ratios. For walls having conventional dimension (aspect ratio in the range between 1 and 2), the three methodologies provide similar values of strength and stiffness. For walls with large aspect ratios (greater than 2) the AISI Lateral Design provides a resistance reduction which is supported by the comparison with experimental results only in few cases. On the contrary, there is no experimental evidence that confirm the high values of stiffness given by the AISI in the case of long walls (aspect ratio less than 1). Therefore, in order to verify the reliability of the presented design procedures, further experimental studies should be developed for walls with aspect ratios less than 1 and higher than 2.

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Appendix. – Notation

A	is the gross cross-sectional area of an end stud;
b	is the panel width;
β	is a coefficient which depends on the *h/b* ratio;
β	is a coefficient depending on sheathing material (AISI lateral design);
c	is the fastener spacing;
γ	is a coefficient which depends on the *h/b* ratio;

d	is the displacement of wall;
δ	is the vertical deformation of anchors;
d_f	deformation of the wall associated to the connection;
d_{ha}	deformation of the wall associated to the frame-to-foundation anchors;
d_p	deformation of the wall associated to the sheathing panel;
d_s	deformation of the wall associated to the end stud;
E	is the young's modulus of steel;
f_y	is the minimum yield stress;
$f_{p,V}$	is the shear strength of panel material;
F_V	the average strength of single connection between sheathing panel and steel frame;
f_u	is the minimum tensile stress;
G	is the shear modulus of elasticity of the panel material;
h	is the wall height;
H	is the lateral resistance of wall
H_f	is the lateral resistance of wall due to connection;
H_{ha}	is the resistance of wall associated to the failure of frame-to-foundation anchors;
H_p	is the lateral resistance of wall corresponding the failure of sheathing panel;
H_s	is the resistance of wall associated to the buckling failure of the end stud in compression;
K_f	Is The Wall Stiffness Contribution Of Sheathing-To-Frame Connections;
$k_{f\text{-}s}$	is the stiffness of a single connection in shear;
K_{ha}	is the wall stiffness contribution due to hold-down devices
k_{hd}	is the axial stiffness of the hold-down;
k_{mod}	is the modification factor due to duration of load;
K_p,	is the wall stiffness due to the sheathing panels;
K_s	is the wall stiffness due to the steel frame;
L	is the wall length;
n	is the number of panel connected to the frame;
N_{ha}	is the tensional resistance of the anchorage;
N_s	is the buckling resistance of the end stud;
ρ	is a coefficient depending on sheathing material;
R_n	is the nominal resistances;
s	is the maximum spacing at the panel edges;
$t_{sheathing}$	is the thickness of the panel;
t_{stud}	is the stud thickness,
v	is the lateral load per unit length acting on the wall;
ν_p	is the Poisson coefficient;
ω_1	is equal to $s/152.4$ with s in mm;
ω_2	is equal to $0.838/t_{stud}$ with t_{stud} in mm;
ω_3	is a coefficient depending on aspect ratio (h/b);
ω_4	is a coefficient depending on sheathing material;