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This paper deals with the experimental and theoretical behavior of plybamboo (kind of plywood made out of bamboo) wall-panels subjected to lateral load. The wall-panels are part of a house design method proposed in the author's PhD thesis for prefabricated social housing in developing countries. Sixteen full-scaled wall-panels with or without window and door openings were tested and their theoretical capacities estimated. Design wind and seismic loads were determined according to the International Building Code 2000. The results showed that all the specimens present ductile behavior adequate for expected wind and seismic loads. The theoretical models for calculating lateral capacities of timber framed walls gave lower values than the experimental ones. The wall-panels sheathed with plybamboo and traditional plywood showed similar behavior and hence, plybamboo could be used as an alternative sheathing material in timber frame construction.

Keywords: shear-walls, plybamboo, timber framed walls, structural panels, bamboo structures.

1 Introduction

This paper deals with the structural behavior of plybamboo wall-panels submitted to lateral load. The wall-panels are part of a prefabricated house design method, which is intended to be applied for social housing in developing countries . The prefabricated wall-panel units consist of two timber studs on which a plybamboo sheet is connected. These wall-panels are after mounted on a fixed bottom plate. A top plate is joined to the wall-panels and finally they are all connected to each other (Figure 1a represents two prefabricated wall-panel units already connected to each other, to the top and to the bottom plate).

The purpose of the research presented is to determine the structural response of plybamboo wall-panels when submitted to lateral load; to obtain their capacity and to compare the experimental results with plywood wall-panels, theoretical values and design loads.

This paper is divided in seven sections. The following section describes the methods and materials employed during the experimental tests. Section 3 presents the most relevant results. Section 4 gives an overview of various analytical procedures available to estimate the lateral capacities of timber framed walls and numerical results for the corresponding test specimens. Section 5 continues with design loads estimations according to IBC 2000 [10] for wind and seismic effects. Section 6 discusses the results including experimental, theoretical and design loads. Section 7 ends the paper with some concluding remarks and recommendations.

2 Methods and materials

2.1 Materials

The materials used to build the test specimens were:

1. Plybamboo: bamboo mat boards from India with mats woven at 45° angles, thickness: 12 mm, mean density: 868 kg/m^3, moisture content: 4%, bending strength: 60 N/mm^2. For more details, see Zoolagud [15] and Ganapathy [8].
2. Okoume Plywood: thickness: 12 mm, three plies, mean density: 467 kg/m^3, moisture content: 7%, class B/BB. Other properties were unavailable.
3. Timber: softwood, class K24 for 75x50 mm sections and K17 for 50x50 mm sections according to NEN 6760 [13], mean density: 458 and 446 kg/m^3 respectively, moisture content: 12%, characteristic bending strength: 24 and 17 N/mm^2 respectively.
4. Steel angles: 60x60x2 mm and 40x40x2 mm.
5. Screws: 5x50 mm, 4x35 mm and 5x80 mm.
6. Nails: 2.8x55 and 4x90 mm smooth round nails.

2.2 Test specimens

The test specimens were composed of wall-panels with dimensions $h \times b$ (Figure 1). The height h and the width b varied from 2400 to 2500 mm and from 2400 to 2450 mm respectively.
Three types of specimens are distinguished in figure 1: full-specimen (F), window-specimen (W) and door-specimen (D). The dimensions of the elements composing the test specimens are given in Table 1.

Full-specimen (Figure 1a)

The full specimen consists of a timber frame composed of four studs (2), one bottom and top plate (3) all on which two plybamboo or plywood sheets (1) are fixed. The studs and top (or bottom) plate are connected with a steel angle and four screws (figure 2). The sheets are fastened to the studs with 5x50 mm screws spaced at 200 mm on center and to the top (or bottom) plate with 2.8x55 mm nails spaced at 190 or 200 mm on center. Finally, the two central studs are nailed to each other with 4x90 mm nails spaced at 200 mm on center.

Window-specimen (Figure 1b)

The window-specimen is the same as the full-specimen but in one half of the timber frame, an additional frame is added for the window opening. The latter frame is composed by two window sills (7) which are connected to the studs via steel angles (in this case, the steel angles are smaller-40x40x2 mm - as well as the timber sections - 44x44 mm) and two window studs (6) which are fixed to the window sills using 5x80 mm screws. One plybamboo or plywood sheet (1) is joined to one half of the frame and four sheets (4, 5) are fastened to the other half and the additional window frame. The window sheathing 1 (4) is connected to the studs with 5x50 mm screws spaced at 200 mm on center and to the top (or bottom) plate and sill with 2.8x55 mm nails spaced at 190 or 200 mm on center. The window sheathing 2 (5) is fixed to the studs and window studs with 5x50 mm screws spaced at 200 mm on center and to the top (or bottom) window sill with two 2.8x55 mm nails spaced at 75 mm on center.

Door-specimen (Figure 1c)

The door-specimen follows the same pattern of the window-specimen. In this case, the additional timber frame is composed by a door sill (11) connected to the studs with steel angles as shown in figure 2 and a door stud (10) joined to the bottom plate with a steel angle and to the door sill with a 5x80 mm screw. The door sheathing (8) is fixed to the studs with 5x50 mm screws spaced at 200 mm on center and to the top plate and door sill with 2.8x55 mm nails spaced at 190 or 200 mm on center. The door sheathing 2 (9) is connected to the stud and door stud with 5x50 mm screws spaced at 200 mm on center and to the bottom plate with three 2.8x55 mm nails spaced at 75 mm on center.

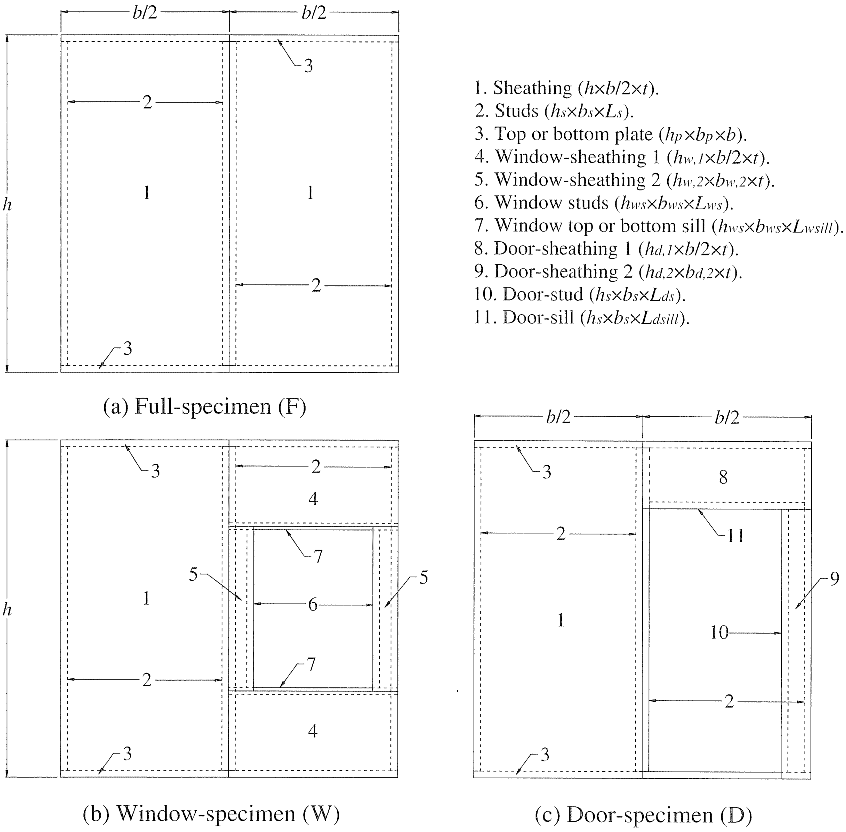


Figure 1. Test specimens used for the experimental tests

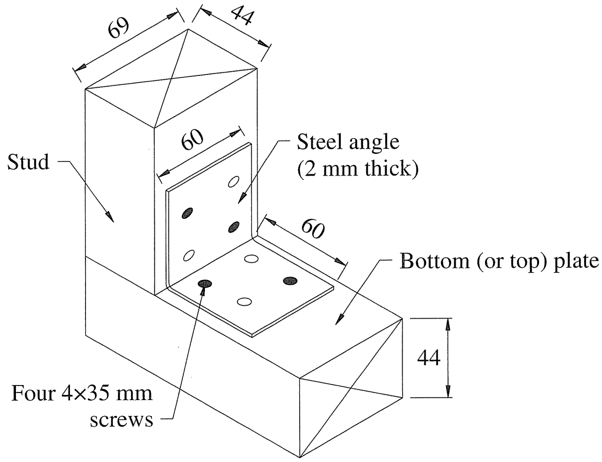


Figure 2. Stud-to-bottom (or top) plate connection

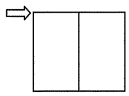
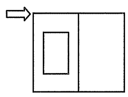
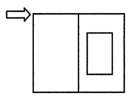
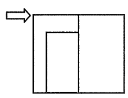
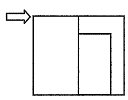
Table 1. Dimensions (in mm) of the elements (Figure 1)

Element	Test series with plybamboo	Test series with plywood
1	2400×1200×12 (2445×1225×12)	2500×1220×12
2	69×44×2312 (69×43×2359)	69×44×2412
3	69×44×2400 (69×43×2450)	69×44×2440
4	600×1200×12	640×1220×12
5	1200×180×12	1220×185×12
6	44×44×1156	44×44×1176
7	44×44×1112	44×44×1132
8	400×1200×12	500×1220×12
9	2000×200×12	2000×220×12
10	69×44×1956	69×44×1956
11	69×44×1112	69×44×1132

2.3 *Test configurations*

In total sixteen tests were performed. Four tests were carried out on the full-specimen (F) whereas three tests were carried out on each test configuration as shown in table 2. Five specimens sheathed with plywood were tested on each of the five variations.

Table 2. Different test configurations with number of tests and type of sheathing

Test name	Test configuration	Number of tests	Sheathed with plybamboo	Sheathed with plywood
Full (F)		4	3	1
Window on the windward side (WA)		3	2	1
Window on the leeward side (WB)		3	2	1
Door on the windward side (DA)		3	2	1
Door on the leeward side (DB)		3	2	1
	Total	16	11	5

⇨ Horizontal force representing the wind or seismic load.

2.4 Test setup

The lateral resistance of the wall-panels was obtained by subjecting a full-scaled wall-panel to a racking load according to ASTM E 564-95 [2]. Other standards are ASTM E 72-98 [3] and EN 596 [5] but the chosen one is more appropriate for the purpose of this research.

The test setup consisted of a steel frame composed by two HE-columns and two HE-beams (HE: wide flange sections) on which the test specimen was mounted and loaded in its own plane with a hydraulic jack at one of the upper corners (figure 3 and figure 4a). The test specimen is joined to the bottom HE-beam using six bolts (10 mm Ø) spaced at 400 mm on center as shown in figure 3 (in the case of the full-specimens sheathed with plybamboo, only four bolts were utilized). At the top, the specimen is restricted to move sideways (out of its own plane) by steel rollers (see top detail of figure 3 and figure 4c). For the full-specimens sheathed with plybamboo, only two of these supports were employed.

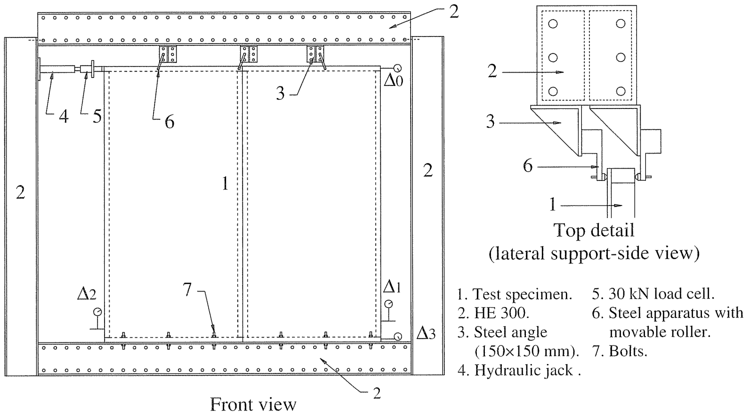
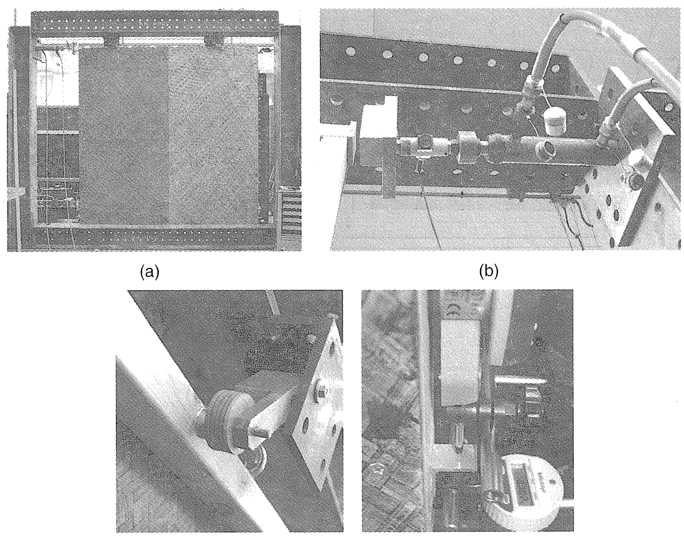


Figure 3. Test setup with a full-specimen (F)

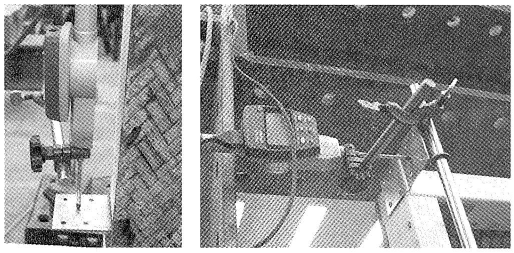
Measurement of load and displacement

A hydraulic jack is fixed to one HE-column and connected to a pneumatic pump. A 30 kN load cell is attached to the hydraulic jack followed by a steel plate, a wood piece and a piece of soft material in order to transmit the load evenly via the top plate (figure 4b). In each test, the load was manually applied at an approximate rate of 1 kN/min. After the first minute, the load was sustained for five minutes and afterwards released for another five minutes. Finally, the load was applied until failure [2]. For the full-specimens sheathed with plybamboo, the hydraulic jack was connected to a compression machine so that the speed could be automatically controlled. In this case, the load was applied at a speed rate of 5 mm/min.



(a) (b) (c) (d)

Figure 4. Test setup details (a) General overview (b) Loading detail (c) Lateral support detail (d)



Continue of figure 4. Measurement of Δ_1 and Δ_3 (e) Measurement of Δ_2 (f) Measurement of Δ_0

The displacement measurement setup for the window and door test configuration is shown in figure 5. For the full-specimen only four displacements were measured as shown in figure 3. The displacements are defined as follows:

Δ_0: Top plate horizontal displacement (Figure 4f).
Δ_1: End-stud (farthest stud from the loading point) vertical displacement (Figure 4d).
Δ_2: Leading stud (closest stud from the loading point) vertical displacement (Figure 4e).
Δ_3: Bottom plate horizontal displacement (Figure 4d).
Δ_4: Top window- or door-sill horizontal displacement (Figure 5).
Δ_5: End-window- or door-stud vertical displacement (Figure 5).
Δ_6: Leading-window- or door-stud vertical displacement (Figure 5).
Δ_7: Bottom window-sill horizontal displacement (Figure 5).

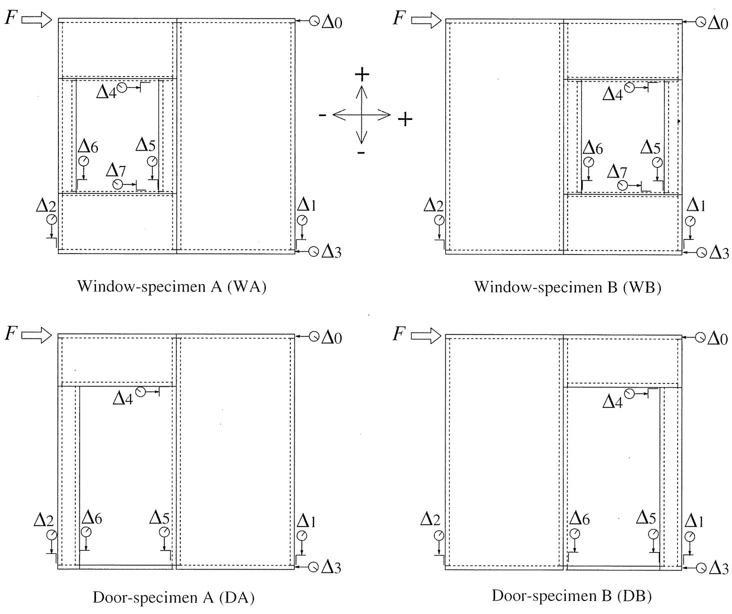
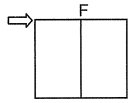
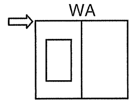
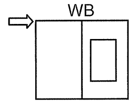
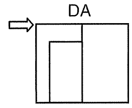
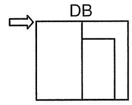


Figure 5. Displacement measurement system for each of the test specimens and configurations

3 Results

The experimental results obtained from all the tests are given in table 3. The Load-displacement curves of all tests are shown in figure 6. For more details, see González 2003 [9]. Typical failure modes of all test configurations observed during the experiments are presented in figure 8.

Table 3. Experimental results. The values in italic are for the tests sheathed with plywood and k_i is the wall-panel racking stiffness for F = 1 kN

Test configuration	Δ_0 = 25 mm					F_u [kN]	k_i [kN/mm]
	Δ_1[mm]	Δ_2 [mm]	Δ_3 [mm]	Δ_2 / Δ_0	F [kN]		
F 	*-2.2*	*13.3*	*0.11*	*0.53*	*11.8*	*12.2*	*1.06*
	-4.3	18.3	0.16	0.73	11.6	13.2	0.64
	-2.3	20.7	0.32	0.83	10.5	12.0	1.75
	-5.0	17.4	0.18	0.69	10.6	13.0	0.71
WA 	*-2.4*	*7.0*	*-0.09*	*0.28*	*12.7*	*14.4*	*1.01*
	-2.9	7.6	0.48	0.31	13.0	13.1	1.18
	-2.7	11.1	0.07	0.45	11.9	12.3.	1.07
WB 	*-0.97*	*12.2*	*-0.02*	*0.48*	*8.6*	*8.7*	*0.71*
	-2.77	13.0	-0.15	0.52	7.6	7.8	0.66
	-2.13	11.9	1.59	0.47	8.4	11.0	0.75
DA 	*-1.68*	*3.61*	*-0.02*	*0.14*	*6.9*	*7.8*	*0.57*
	-1.95	3.31	0.01	0.13	7.2	7.4	1.10
	-1.77	3.25	0.12	0.13	7.1	7.3	0.75
DB 	*-1.24*	*11.0*	*-0.01*	*0.43*	*4.2*	*4.8*	*0.54*
	-2.17	11.5	0.03	0.46	4.0	5.2	0.48
	-2.57	11.4	-0.05	0.44	4.3	5.6	0.52

The racking stiffness was estimated at 1 KN of load in order to compare all the test configurations. Therefore, it was not according to ASTM E-564.

The observed behavior of the full-specimens sheathed with plybamboo according to the Load-displacement curves can be summarized in four phases (see Figure 7):

1. Gap accommodation between individual members: this phase was present in two of the three tests. It was up to a lateral displacement of 5 mm and a lateral load of 3 kN. This effect was caused by panel fabrication in which certain connections between plates and studs were not tight enough.
2. Racking stiffness: after the gaps are closed, the specimen behaves with its "real" racking stiff-

ness, which is given mostly by the screwed and nailed connections between sheet and frame. As the load increases, the stiffness decreases due to non-linear behavior of the sheet-to-frame connections.

3. Low stiffness: this phase begins when most of the stiffness is lost due to yielding of the steel angles connecting the studs and the bottom plate, yielding of the nails and bending of the bottom plate and sheet (figure 8a, b). After this phase, all the connections behave plastically.
4. Failure: the complete failure or collapse was not reached due to excessive deformation. The tests were stopped after reaching approximately 45 mm lateral displacement. All tests with the same test configuration gave similar maximum loads F_u (Table 3).

From the remaining experimental tests it was observed that:

1. The behavior of the full-specimen sheathed with plywood was similar to that one sheathed with plybamboo although the gap accommodation effect was not present.
2. The behavior of the window-specimen with the window on the windward side (WA-series) was similar to the full-specimen in stiffness and strength. However, there is a more accentuated failure at the central studs due to different distribution of forces (Figure 8c).
3. The behavior of the window-specimen with the window on the leeward side (DA-series) was less stiff and strong than the two previous ones. The failure only occurred at the connection between the leading stud and the bottom plate (Figure 8d).
4. The behavior of the door-specimen with the door on the windward side (DA-series) was similar to the WB-series in stiffness. The strength of the WB- and DA-series was similar with the exception of one test (see Figure 6). In this case, the failure occurred at the central studs (Figure 8e).
5. The behavior of the door-specimen with the door on the leeward side (DB-series) was the weakest in both stiffness and strength (see Figure 6). The partial failure only occurred at the connection between the leading stud and the bottom plate (Figure 8f).
6. There was no significant difference between the specimens sheathed with plybamboo and plywood.

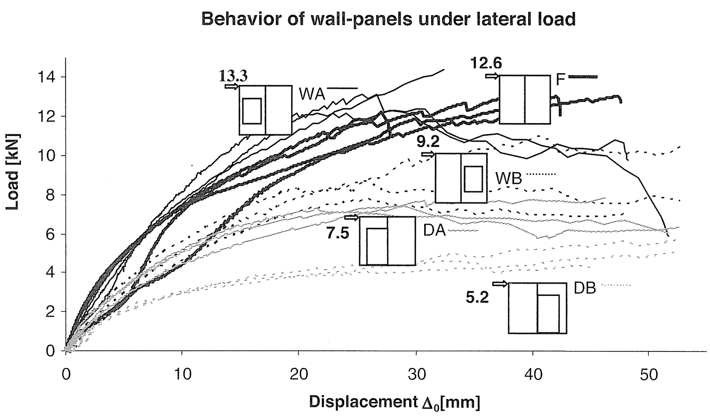


Figure 6. Load-displacement curves obtained from the performed tests. The average of the maximum load of each test configuration are in bold

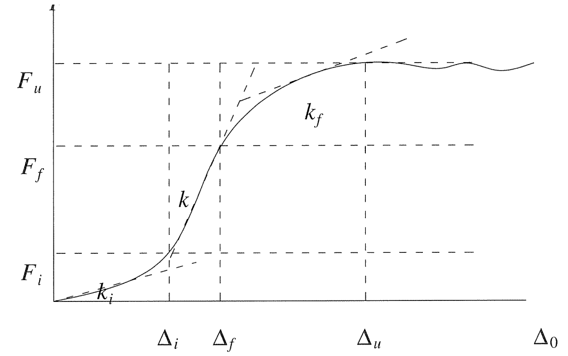


Figure 7. Schematic behavior of full-specimens sheathed with plybamboo.

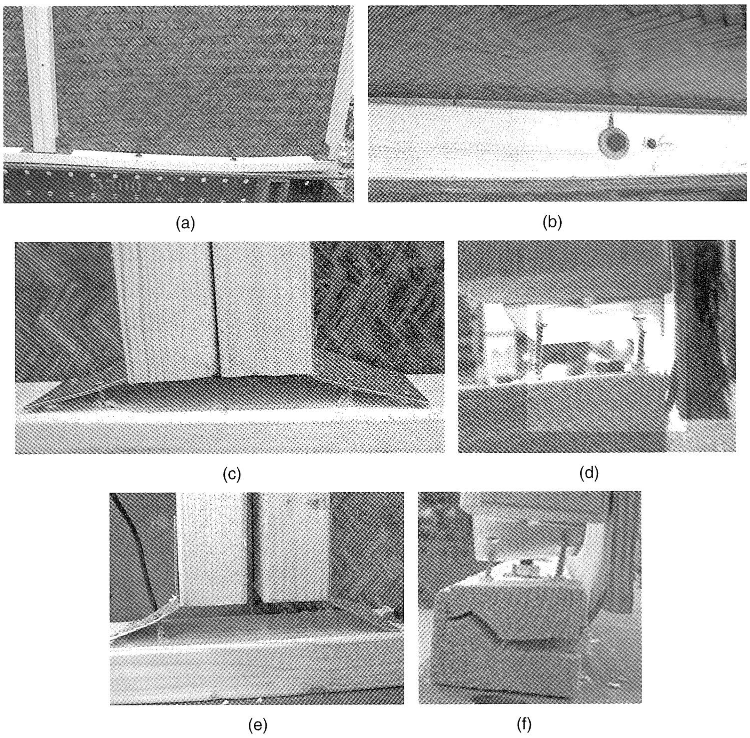


Figure 8. Failure modes presented during the tests (a) Test series F: bending of the bottom plate (b) Test series F: yielding of nails and bending of the sheet (c) Test series WA: failure at the connection between the central studs and the bottom plate (d) Test series WB: failure at the connection between the leading stud and the bottom plate and the sheet-to-bottom plate connection (e) Test series DA: same as (c). Notice the opening between the central studs (f) Test series DB: same as (d). Notice the splitting of the bottom plate

4 Analytical procedures

The following sections present some of the available models for calculating lateral capacities of timber framed walls. Each of the models are briefly explained and further applied to the test results of each specimen.

4.1 Linear-elastic analysis

This model assumes linear-elastic behavior of the fasteners, hinged connections between individual elements and that uplifting is prevented. Besides, the beam elements and sheathing are considered to be completely stiff against bending and elongation in the loading plane. With these assumptions, the lateral capacity of a timber framed wall F_H is attained when the maximum loaded nail reaches its maximum capacity $f_{i,max}$, see Alsmarker, T. Lund University, step lecture B13 [4]. Equalizing the external and internal moments according to figure 9, the following expressions can be obtained:

$$f_i = \sqrt{f_{x,i}^{\ 2} + f_{y,i}^{\ 2}} \tag{1}$$

$$f_{x,i} = \frac{F_H h y_i}{\sum y_i^{\ 2}}, \quad f_{y,i} = \frac{F_H h x_i}{\sum x_i^{\ 2}} \tag{2}$$

$$F_H = \frac{f_{i,\max}}{h} \frac{1}{\sqrt{\left(\frac{y_{i,\max}}{\sum y_i^{\ 2}}\right)^2 + \left(\frac{x_{i,\max}}{\sum x_i^{\ 2}}\right)^2}} \tag{3}$$

Where $f_{x,i}$ and $f_{y,i}$ are the force components in x and y directions respectively for a fastener in position(x_i,y_i), f_i is the total force for a fastener i and h is the height of the wall-panel (see figure 9). The most loaded fasteners will be those ones located at the corners ($x_{i,max}$, $y_{i,max}$).

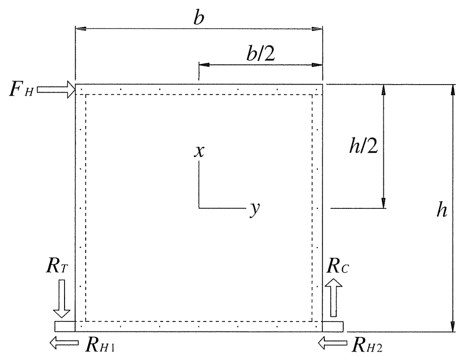


Figure 9 Structural model of timber framed wall assuming linear-elastic behaviour of fasteners

4.2 Eurocode procedure

The model adopted by Eurocode 5 gives the following formula (based on equation 5.4.3a in Eurocode 5) to calculate the lateral capacity of timber framed walls:

$$F_H = f_{i,\max} b / s = f_p b \tag{4}$$

Where b is the sheet width, s is the spacing of the fasteners and f_p is the maximum capacity of fastener per length. In this model the applied force is uniformly distributed over the fasteners connecting the sheathing to the top plate and does not account for concentrated forces at the corners of the panel, see Alsmarker, T. Lund University, step lecture B13 [4]. When using equation (4) it must be provided that (based on equation 5.4.3d in Eurocode 5):

$$R_T \geq F_H h / b \tag{5}$$

Where R_T is the tensile reaction in the leading stud (Figure 9).

4.3 *Plastic analysis*

The plastic model shown in figure 10 is based on a method developed by Kallsner et al [12]. In this method, a force distribution that complies with force and moment equilibrium for each timber member and sheet is chosen (plastic lower bound theory). The basic assumptions are that the load-displacement relationships in the sheet-to-frame connections are completely plastic, the displacements are small compared to the width and height of the sheet, the connections between individual elements are hinged and the sheet and the beam elements are considered to be completely stiff against bending and elongation in the loading plane. Furthermore, the sheet to timber joints in the top plate are assumed to transfer forces only parallel to the top plate and the sheet to studs joints are assumed to transfer forces only parallel to the studs. For the bottom plate, only perpendicular or parallel forces may be transmitted. After these considerations, the model of figure 10 is assumed based on visual inspection during the tests.

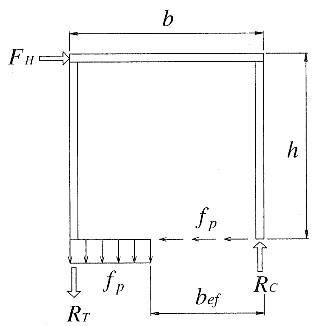


Figure 10. Plastic model when leading stud is connected to the bottom plate

From figure 10 it can be obtained that:

$$F_H = f_p b_{ef} \tag{6}$$

And from sum of moments at the bottom right corner:

$$b_{ef} = -h + \sqrt{h^2 + b^2 + \frac{2bR_T}{f_p}} \tag{7}$$

4.4 Empirical procedure for walls with openings

An empirical method for calculating the lateral capacity of perforated timber framed walls under cyclic loading was developed by Sugiyama, 1993 [14]. It consists of determining the following factor:

$$\alpha = \frac{r}{3-2r} \tag{8}$$

Where,

$$r = \frac{A_T - A_o}{A_o} \tag{9}$$

A_T is the total wall-area and A_o is the perforated sheathing area. The perforated shear-wall capacity is estimated multiplying the factor α by the capacity of a shear-wall without openings.

4.5 Results of analytical methods

F-series

1. Linear elastic model: if the two sheets are considered to rotate independently, equation (3) is applied for each of them and the final result is multiplied by two (the center of rotation is assumed to be at the centroid of each sheet). Thus,

$$\sum x_i^2 = 12 \times 2 \times 590.5^2 + 4\left(200^2 + 400^2 + 600^2\right) = 10.6 \times 10^6 \text{ mm}^2$$

$$\sum y_i^2 = 7 \times 2 \times 1200.5^2 + 4\left(100^2 + 300^2 + \ldots + 900^2 + 1100^2\right) = 31.6 \times 10^6 \text{ mm}^2$$

$$\frac{F_H}{2} = \frac{f_{i,\max}}{2445} \frac{1}{\sqrt{\left(\frac{1200.5}{31.6 \times 10^6}\right)^2 + \left(\frac{600}{10.6 \times 10^6}\right)^2}} = 6.0 f_{i,\max}$$

$\Rightarrow F_H = 12 f_{i,\max}$. With $f_{i,\max} = 0.86$ kN [9], F_H = **10.3 kN**

2. Eurocode: from equation (4), $F_H = f_{i,max}$ x 2450/200 = 12.25$f_{i,max}$ $\Rightarrow F_H$ = 10.5 kN.
   This result is similar to that one obtained with the linear elastic model. However, the spacing of 200 mm is not constant at the connection between the two sheets.

3. Plastic analysis: for the full-specimen, the plastic model of figure 11 can be assumed. In the case of the full-specimens sheathed with plybamboo, only four bolts were used to fix the bottom plate and the yielding of the nails in a vertical direction occurred between the two bolts localized in the tension zone (see Figure 8a, b). Thus, $b_{ef} = b\text{-}l$ = 2450 - 400 (distance between bolts) = 2050 mm and F_H = 0.86 x 2050/188.5 = **9.4 kN**. In the case of the full-specimen sheathed with

plywood, the yielding of the nails in a vertical direction occurred near the leading stud as shown in figure 11. The effective width b_{ef} cannot be determined because the values R_{T1} and R_{T2} are unknown but assuming that $l \approx 400$ mm, a similar value of 9.4 kN is obtained.

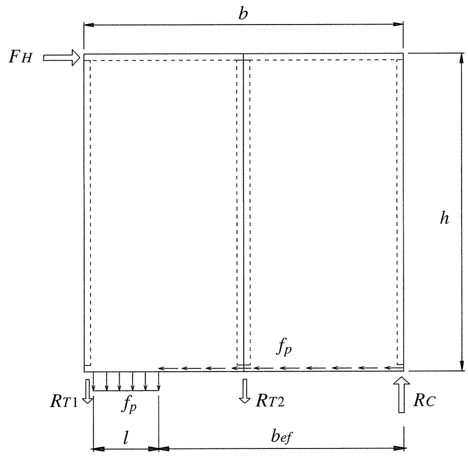


Figure 11. Plastic model for full-specimen.

WA-series

The WA-series (window-specimen with window on the windward side) behaved as stiff and strong as the full-series, which is a surprising fact. It was observed during the experimental tests that both the leading and central studs (figure 8c) were uplifted from the bottom plate. The distribution of forces at failure can be modeled as shown in figure 12. Since the force R_{T2} is unknown, b_{ef} cannot be estimated. However, it seems possible that F_H is similar to that one of the full-series.

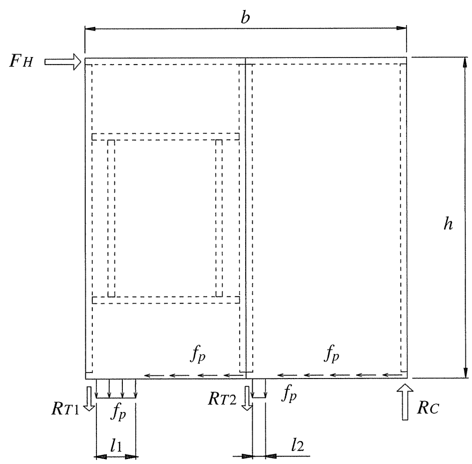


Figure 12. Plastic model for the WA-series based on visual observation at failure. R_{T1} develops all of its capacity and yielding of the nails in the vertical direction occurred in two zones. R_{T2} does not develop all of its capacity.

WB-series

The WB-series (window-specimen with window on the leeward side) behaved less stiff than the WA-series. In this case, only the leading stud was uplifted from the bottom plate (figure 8d).

1. Plastic analysis: the distribution of forces at failure can be modeled as shown in figure 16. The effective width can be estimated using equation (7). Taking h = 2.4 m, b = 2.4 m, R_T = 4.2 kN (9) and f_p = 4.53 kN/m (= $f_{i,max}$ / s = 0.86/0.19), a value of b_{ef} = 1.6 m is obtained. Hence, F_H = 1.6x4.53 = **7.2 kN**
2. Empirical procedure: using equation (8) and (9), A_T = 2.4x2.4 = 5.76 m^2, A_o = 1.2x0.84 = 1.01 m^2, r = 0.825 and α = 0.611. Taking the mean capacity of the F-series as the capacity of the shear-wall without openings, F_H = *0.611x12.6* = **7.7 kN**

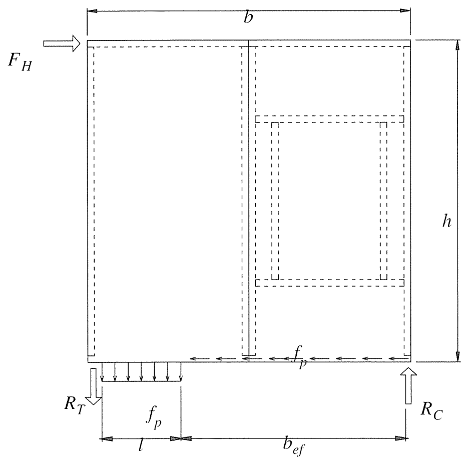


Figure 13. Plastic model for the WB-series based on visual observation at failure. The tension at the central studs is not developed.

DA-series

The DA-series (door-specimen with door on the windward side) behaved as stiff and strong as the WB-series which is considered as a coincidental fact. The distribution of forces at failure is modeled as shown in figure 14 where the panel with the complete sheet is taking all the forces. Equation (7) can be employed in this case as well and with h = 2.4 m, b = 1.2 m, R_T = 4.2x2 = 8.4 kN and f_p = 4.53 kN/m, a value of b_{ef} = 1.01 m is obtained. Hence, F_H = 1.01x4.53 = **4.6 kN**.

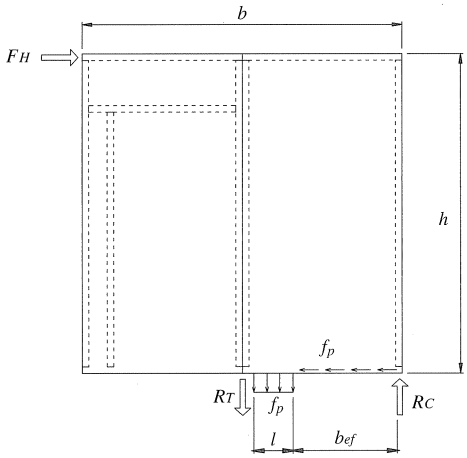


Figure 14. Plastic model for the DA-series based on visual observation at failure. The tension at the central studs is completely developed.

DB-series

The DB-series (door-specimen with door on the leeward side) were the least stiff of all the test series.

1. Plastic analysis: the distribution of forces at failure is modeled as shown in figure 15. From this force configuration, the effective width can be determined as:

$$b_{ef} = h + b - \sqrt{h^2 + \frac{b^2}{4} + 2b\left(h - \frac{R_T}{f_p}\right)} \tag{10}$$

   With h = 2.4 m, p = 2.4 m, R_T = 4.2 kN and f_p = 4.53 kN/m, a value of b_{ef} = 1.02 m is obtained. Hence, F_H = 1.02x4.53 = **4.6 kN**

2. Empirical procedure: using equations (8) and (9), A_T = 5.76 m², A_o = 2x1 = 2 m², r = 0.653 and α = 0.385. Thus, F_H = 0.385x12.6 = **4.8 kN**

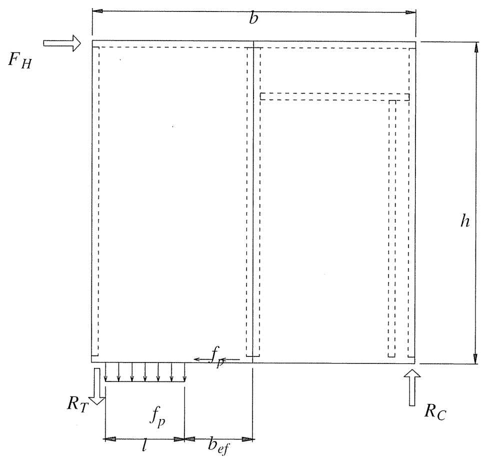


Figure 15. Plastic model for the DB-series based on visual observation at failure. R_T develops all of its capacity.

5 Design loads

In order to have an idea of the magnitude of the load to which the wall-panels will be subjected during their lifetime, a simple house example is taken for practical purposes (figure 16). In the following example, the design wind and seismic load is estimated for this house based on the IBC 2000 .

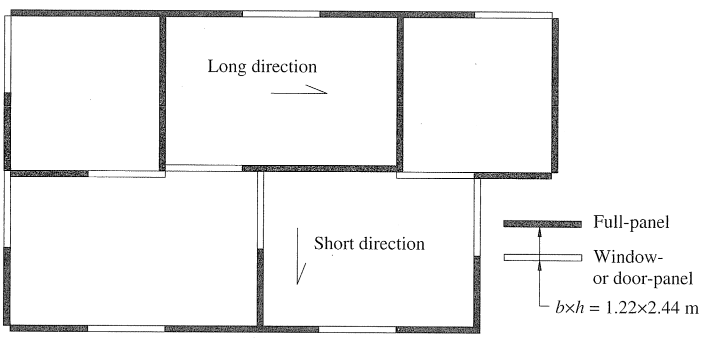


Figure 16. Floor plan of house showing panel configuration

5.1 Wind loads

In order to estimate the design wind load to which the shear-walls will be subjected, the floor plan of the house shown in figure 16 (see González, 2003 for more details) is adopted and the following procedure is applied:

1. Calculation of shear-walls critical length

   figure 16 shows that the house has 10 full-panels in the short direction of the house and 13 in the long direction. Hence, the critical length L_{cr} would be **12.2 m** considering that the wall-panel unit width is 1.22 m. In this model, each full-panel is assumed to be equally stiff and to take the same reactions. In order to achieve this, the roof trusses or ceiling must be very stiff and consequently be able to transmit the load to the top plates, which in turn transmit it to the shear-walls. If necessary, bracing members connecting non-continuous shear-walls via the top plate may be used.

2. Estimation of the total wind force acting on the critical direction
   a. Calculation of the critical area facing the wind
      The critical area A_{cr} would be **10.4 m²** considering that the height of the wall-panels is 2.44 m and half is taken directly by the foundation (7x1.22x2.44 / 2, where 7 is the number of wall-panels perpendicular to the critical direction). The roof area is not considered because the horizontal component for a slope of 20° is canceled by the horizontal component of the adjacent roof. If a two-story house were considered, the critical area would be 31.2 m² (10.4+10.4x2).
   b. Determining the design wind pressure w_d
      Considering a basic wind pressure q_w = 1.07 kN/m² (for a wind velocity of 120 km/h and exposure category C according to ASCE 7-98), a value w_d = 1.07 (0.8 + 0.64) = **1.54 kN/m²** is obtained. The internal pressure is canceled in the horizontal direction whereas 0.8 and 0.64 are the external pressure coefficients for windward and leeward walls respectively.
   c. Estimation of total wind load F_w
      The total wind load is calculated multiplying the design wind pressure w_d by the critical area A_{cr}. Therefore, F_w = 1.54x10.4 = **16.0 kN**. For a two-story house, F_w = 1.54x31.2 = 48.0 kN.
3. Determining the design wind load per meter-wall
   The design wind load per meter-wall $F_{w,d}$ is estimated dividing the total wind load F_w by the critical length L_{cr}. Thus, $F_{w,d}$ = 16/12.2 = **1.31 kN/m**. For a two-story house, $F_{w,d}$ = 48/12.2 = 3.93 kN/m.

5.2 Seismic loads

Considering the example of the previous section, the following procedure is applied in order to estimate the design seismic loads for the house in consideration.

1. Calculation of shear-walls critical length
   Same as for wind loads, L_{cr} = **12.2 m**
2. Calculation of total seismic force F_s [10]
   a. Calculation of seismic coefficient C_s according to the regional seismic code
      In this example F_s is calculated using equation (16-49) of IBC 2000 [10]. First C_s is calculated. Assuming that S_s =1.0 (mapped spectral acceleration at short periods, the maximum value in USA is 2.0) and F_a = 1.1(site coefficient, maximum value for S_s = 1.0) a value of S_{DS} = 0.73 (design spectral response acceleration at short periods) is obtained. For bearing wall systems composed by light framed walls with shear panels a value R = 2.0 (response modification factor) can be used. From equation (16-49), $C_s = 1.2S_{DS} / R$ = 1.2x0.73 / 2 = **0.44.**

b. Estimation of total weight of the structure W_1 or W_2

For one-story houses, the following equation can be used:

$$W_1 = W_w + w_r A_r \tag{11}$$

Where W_w is the total weight of the walls per story, w_r is the weight of the roof per area and A_r is the total roof area. There are 34 panels weighing approximately 44 kg or 0.44 kN each (openings are assumed to weight as much as the wall-panels). Hence, W_w = 0.44x34 = 15 kN. Considering a light-weight roof of 0.15 kN/m² and a roof area of 63 m², W_1 = 15+0.15x63 ≈ **25 kN**. For two-story houses,

$$W_2 = W_1 + W_w + w_f A_f + \alpha w_L A_f \tag{12}$$

Where W_f is the floor weight per area, A_f is the total floor area, w_L is the live load and α is a factor that reduces the live load. For this example, a wooden floor weighing 0.4 kN/m² is considered. The live load is taken as 1.92 kN/m² [10] with a reduction factor of 0.15 [7]. The total floor area is 39 m². Thus, W_2 = 25+15+0.4x39+0.15x1.92x39 = 67 kN

3. Estimation of design seismic load $F_{s,d}$

$F_{s,d1}$ = 25x0.44/12.2 = **0.90 kN/m** for one-story house and,

$F_{s,d2}$ = 67x0.44/12.2 = 2.42 kN/m for a two-story house.

6 Discussion

6.1 Experimental setup and results

The experimental setup used to test the wall-panels under lateral load allows the rotation of the specimens because that would be the real case. Therefore, the wall-panels are considered to be partially anchored to the foundation.

The full-specimens sheathed with plybamboo were fixed to the bottom plate with four bolts but still the bottom plate bent. The behavior of these specimens might have been affected by this but also by panel fabrication. Especially the initial stiffness (gap accommodation) for two of the specimens as can be seen in figure 6. The previous was solved for the rest of the specimens by using six bolts and thereafter, none of the specimens showed the gap accommodation effect. The test-specimens were restricted to move sideways, which would be close to the real case considering the bracing provided by the roof structure and / or ceiling.

In general, all the test specimens showed ductile behavior adequate for wind and seismic effects and collapse could not be reached due to large deformations (failure due to deformation at approximately 45 mm of lateral displacement).

6.2 Theory versus experiments

All the experimental capacities for the full-specimens sheathed with plybamboo (13.2, 12.1 and 13.0 kN) were higher then the maximum theoretical capacity (11.2 kN), which means that the theory is on the safe side for the specific test geometry (table 4). The expected theoretical capacity - according to the plastic model - (9.4 kN) is 16% less than the maximum theoretical capacity- Eurocode 5 - due to the fact that the specimen is partially anchored to the foundation. For the full-specimen sheathed with plywood, $f_{i,max}$ is slightly reduced due to the lower density of the plywood and the maximum capacity according to Eurocode 5 is 9.5 kN which is lower than the experimental one (12.2 kN). For the rest of the test configurations, all the experimental capacities were higher than the theoretical ones (table 4) and once more, the theory seems to be on the safe side. It is remarkable how the empirical equation (8) predicts the capacity which in both cases is close to the experimental values. The plastic model for the DA-series underestimates its capacity.

Table 4. Comparison between experimental and theoretical capacities in kN

Test series	F	WA	WB	DA	DB
Experimental value	12.6	13.3	9.2	7.5	5.2
Linear-elastic analysis	10.3	-	-	-	-
Eurocode 5	11.2	-	-	-	-
Plastic analysis	9.4	9.4	7.2	4.6	4.6
Empirical procedure	-	-	7.7	-	4.8

6.3 Design loads

The capacity per meter wall of the full-specimens is 5.2 kN/m (12.6 / 2.44) in average which is higher than the design wind and seismic loads estimated in section 5 (1.31 and 0.9 kN/m respectively). For serviceability, a value of 0.2 kN/m for both wind and seismic loads can be estimated considering a wind speed of 50 Km/h and a seismic coefficient of 0.1. If the serviceability limit is set at h / 1200 $\approx$ 2 mm [11], a minimum value of 0.5 kN/m is obtained from figure 6 for the full-specimens with the gap accommodation effect. For two-story houses, the required values (3.9 kN/m for wind and 2.4 kN/m for earthquakes) are also lower than the experimental ones but the safety factor is not reliable for wind effects. For serviceability, a value of 0.66 kN/m for wind and 0.55 kN/m for seismic load can be estimated. These values are higher than the ones obtained for the full-specimens with the gap accommodation effect.

The procedure applied in section 5 can be conservatively employed for all the wall-panel configurations except for the door-specimen with the door on the leeward side because its capacity (2.2 kN/m) is less than half the capacity of a full-panel (5.2 kN/m). Consequently, wall-panels with a door opening and with $b = h$ should be neglected as shear-walls.

7 Conclusions and recommendations

The behavior of the wall-panels against lateral load is of great importance when horizontal loads caused by wind or earthquakes act on the house structure since they will be in charge of transmitting these forces to the foundation.

All theoretical models (section 4) seem to safely predict the lateral capacity of the test specimens. However, the linear-elastic analysis and the Eurocode 5 procedure require that the leading stud is completely anchored to the foundation whereas the plastic analysis takes into account the anchoring capacity of the leading stud.

The plybamboo wall-panels (acting as shear-walls) to be utilized in one-story houses seem to be able to withstand high wind and seismic loads and deformed at adequate levels when usual winds or tremors occur.
The plybamboo wall-panels might be able to withstand high seismic loads with a second floor on top of them (safety factor of 2) but the gap accommodation effect must be avoided in order to comply with serviceability. On the other hand, it is not recommendable to build a two-story house with these wall-panels in areas prone to high wind velocities.
When submitted to monotonic load, the plybamboo wall-panels with $b = h$ and composed of two wall-panel units can be listed according to stiffness and strength in the following descending order (Figure 6):

1. Full-specimen and window-specimen with window on the windward side.
2. Window-specimen with window on the leeward side and door-specimen with door on the windward side.
3. Door-specimen with door on the leeward side.

Three bolts spaced at 400 mm on center were enough to avoid bending of the bottom plate and the accommodation of gap effects were not present. The latter was also due to panel-fabrication improvement.
The test specimens sheathed with plywood exhibited the same behavior as the ones sheathed with plybamboo. Even though the embedding strength of the utilized plybamboo is higher than that one of the plywood used, the lateral capacity of the sheet-to-bottom (top) plate connection is not considerably reduced. Plybamboo could consequently substitute plywood as sheathing material in timber framed-house construction.
The behavior of the plybamboo wall-panels under cyclic load has not been investigated and needs further research. In that case, stiffness degradation and impairment of strength and pinching could be expected after the fasteners have plastically deformed.
More experimental tests are needed to confirm the applicability of the plybamboo wall-panels as shear-walls in order to obtain design values derived from statistical analyses, especially if two-story houses are to be implemented.
The transfer of loads from the walls and roof perpendicular to the shear-walls by means of roof structure or bracing members (i.e. the ceiling) needs further research.

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