

## RACKING AND BENDING TESTS FOR PREFABRICATED WALL PANELS

### TEST DE CIZALLAMIENTO Y DE FLEXION EN MUROS DE MADERA PREFABRICADOS

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#### ABSTRACT

The goal of this research was to evaluate and recommend improvement of wall-to-wall connections for prefabricated wall panels. Results of 6 racking tests and 9 bending tests on full-size 2.44 by 2.44 m walls are presented. The specimens were composed of two 1.22 by 2.44 m segments attached with three different types of connection configurations. Static racking tests were performed according to ASTM standards and with two types of wall-to-foundation attachments. Bending wall tests were carried out according to a proposed protocol based on the calculation of the wind pressure corresponding to the five hurricane categories. For bending tests three types of wall-to-foundation attachments were used.

The results of the racking tests indicate that the type of connection between the wall segments did not have a definite influence on the strength of the anchored walls, whereas for walls nailed to the base the underutilization of the full capacity was noted. For bending tests, the connected wall segments resisted high pressure independent on the central connection used. The trend of the test results will be used to validate a preliminary finite element model developed to predict the performance of prefabricated wall panels subjected to bending and racking forces, with special focus on the interface and anchorage behaviour for performance optimization.

**Keywords:** *prefabricated wall panels, racking test, bending test*

#### RESUMEN

El objetivo de este estudio fue evaluar y optimizar conexiones lineales "muro a muro" utilizadas en la instalación de muros de madera prefabricados para casas. Los resultados de 6 tests de cizallamiento y 9 tests de flexión realizados en muros de 2.44 x 2.44 m son presentados a continuación. Los muros fueron prefabricados a partir de dos segmentos de 1.22 x 2.44 m conectados con tres tipos de conexión. Los tests de cizallamiento fueron realizados de acuerdo a las indicaciones presentes en la norma ASTM, y dos conexiones diferentes a los cimientos fueron utilizadas. Los tests de flexión fueron llevados a cabo utilizando un protocolo de carga propuesto, ideado a partir del cálculo de la presión de viento ejercida perpendicularmente a los muros. Para estos tests, tres tipos de conexión a los cimientos fueron utilizadas.

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En los tests de cizallamiento, los resultados demuestran que el tipo de conexión utilizada entre los muros no incide significativamente en la resistencia de los muros anclados a los cimientos, mientras que, para los muros clavados a los cimientos, la capacidad de resistencia no fue alcanzada. Respecto a los tests de flexión, los muros resistieron alta presión de viento, independiente del tipo de conexión muro a muro utilizada. Los resultados serán usados para validar un modelo preliminar de elementos finitos, desarrollado para predecir el comportamiento de muros prefabricados ante solicitaciones de cizallamiento y de flexión, con especial atención en la optimización de la conexión muro a muro y del anclaje a los cimientos.

**Palabras clave:** *muros prefabricados, test de cizalle, test de flexión*

## INTRODUCTION

Panelized housing is a construction system which has grown in the last few years and a constant growth is predicted for at least the next twenty years in North-America (Schuler and Adair, 2003). One of the characteristics of this system is the prefabrication of the structural components at a factory, followed by delivery and fast and easy installation on site. As for any other type of construction, an adequate performance is expected from this system when subjected to extreme environmental loads, such as high winds or earthquakes, which are common natural disasters in North-America. The south of the USA is a zone prone to be affected by hurricanes every year and the central zone frequently experiences tornadoes, while the west coast of North America is a zone of high seismic activity. As part of the lateral load resisting system, shear wall panels transfer external loads from the diaphragms (roof systems or upper floors) to the foundation. The shear wall performance under load is intimately related to different parameters such as framing-to-framing and sheathing-to-framing connections configurations, type and thickness of sheathing, type of wall-to-foundation anchorage, aspect ratio, and number of openings (Dolan and Heine 1997, Shenton *et al.* 1998, Salenikovich and Dolan 2000). The performance of walls with hold-down anchors, representing typical engineered constructions, was studied extensively, while the performance of non-anchored walls, representing non-engineered practices, is not well studied or documented yet.

Wall panels are fabricated in lengths ranging from 0.6 m to 4.88 m (Bouchard *et al.*, 2002) and are connected linearly or forming a corner. Prefabricated wall panels could be furnished with insulation and finished with exterior and interior sheathing. Joining the segments and attachment to the upper and lower structures may require extra efforts and cost which should be justified. Practices to connect wall segments differ from one manufacturer to another, and the type of anchorage at the base depends on the geographic location of the construction site. In order to successfully market the prefabricated system in different regions, it should be easily adaptable to the local code requirements.

The objective of this work was to quantify and compare the racking and transverse bending resistances of prefabricated wall segments joined with various types of connection configurations when connected to the foundation in different ways. The results will be used in future work for the development of a finite element model and optimisation of new and improved connection systems.

## MATERIALS & METHODS

### Specimens

Based on common practices observed during industrial visits to five manufacturers of prefabricated wall components of the Province of Québec, three types of connection configurations, that means number of studs, type and schedule of fasteners at the joint zone, were selected to join two wall segments

of 1.22 by 2.44 m to produce a wall specimen of 2.44 by 2.44 m (see Figure 1). A total of 15 walls, 9 for bending tests and 6 for racking tests, were fabricated with two types of wall segments, which were built depending on the central interface connection used.

The frame of the wall segments was built with 38 by 89 mm (2x4 in. nominal) studs of spruce-pine-fir (SPF) grade No. 2 lumber. Before fabrication, specific gravity and modulus of elasticity of framing members were measured using a Metriguard Model 340 transverse vibration E-computer. The framing members were selected on the basis of equal MOE distributions between different wall specimens. A multiple range test LSD (Least Significant Difference, 95%) for MOE average by wall specimen confirmed the homogeneity of MOE average between the wall framing members. The average MOE values of the framing for each wall are shown in Tables 2 and 3.

Studs were spaced 406 mm on centres (o.c.) and the chords were made of double studs fastened with two 16d common nails ( $\text{Ø} 4.1 \times 89 \text{ mm}$ ) every 610 mm o.c. The studs were fastened to the top and bottom plates with two 16d common nails at each end. The frame was sheathed on one side with 11 mm thick oriented strand board (OSB) panels attached to the frame with 8d power-driven SENCO® nails ( $\text{Ø} 3.3 \times 63.5 \text{ mm}$ ) every 152 mm o.c. on the perimeter, and 305 mm o.c. in the field. The sheathing panels were oriented with the long side parallel to the studs. The edge distance for sheathing nails was 19 mm at the chords and at the bottom plate and 56 mm at the top plate taking into account the accommodation of the second top plate joining the two segments at the time of the assembly without nailing to the sheathing panel. The edge distances at the central zone, where the segments were joined, were different depending on the type of the connection configuration used between the two segments.

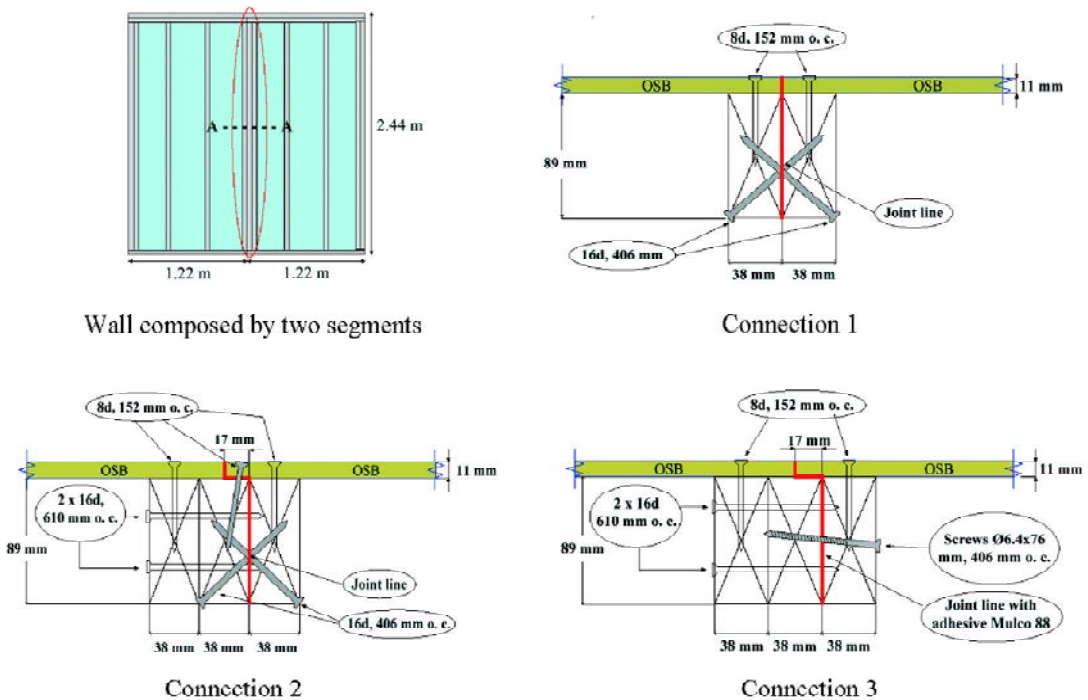


FIGURE 1: Transversal view A-A of the three central joints for prefabricated wall panels.

*Connection 1* was made by joining the segments, with single studs in the joint zone, with 16d common nails driven in the opposite corners of the studs at 45° angle (two nails every 406 mm) as showed in Figure 1. *Connection 2* had the same nailing pattern in the joint zone with additional nailing of the OSB panel overlap (offset) to the double stud of the adjacent segment with 8d power-driven SENCO® nails. *Connection 3* was made with SDS Ø6.4 x 76.2 mm Simpson Strong Tie® screws (one screw every 152 mm) combined with the application of non-structural type of adhesive (Mulco 88) between the studs (see Figure 1). This type of adhesive is used sometimes in the panelized construction to improve the air-tightness of the wall.

For walls joined with *Connection 1*, the edge distance was 17 mm, allowing for a gap of 3 mm between the sheathing panels; whereas for walls joined with *Connections 2* and *3*, the edge distance was 36 mm. For *Connection 2*, the second row of nails joining the two segments was driven with the minimum edge distance of 10 mm (see Figure 1).

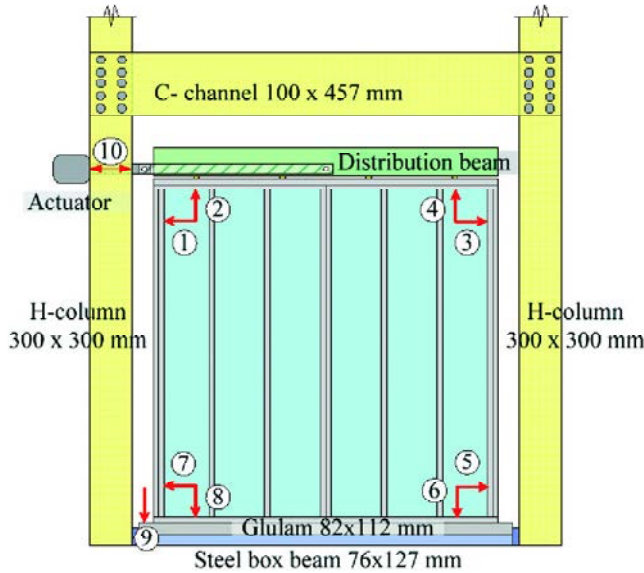
Each wall assembly was coded as shown in Table 1. The number identified the type of central connection between the two wall segments and the letter denoted the type of attachment to the base (described in the next two sections). All the materials and fabricated walls were stored and tested in a laboratory with controlled ambient conditions.

**Table 1:** Identification of the specimens.

Connection	Monotonic Racking test		Bending test		
	Wall-to-foundation attachment		Wall-to-foundation attachment		
	N	HD	A	B	C
1	1NM	1HDM	1A	1B	1C
2	2NM	2HDM	2A	2B	2C
3	3NM	3HDM	3A	3B	3C

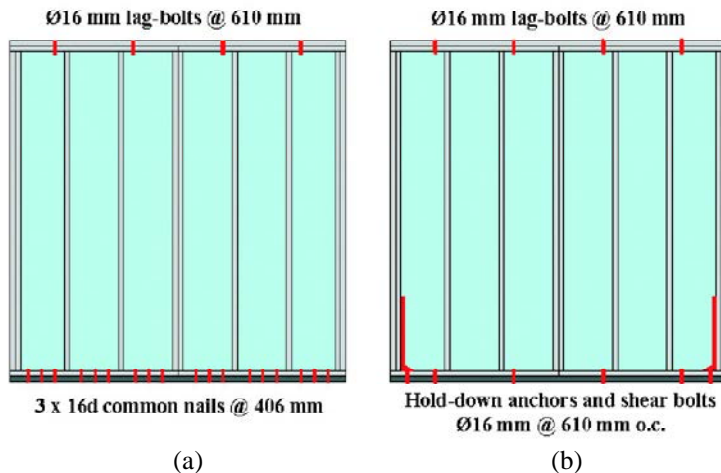
### Racking Test-Setup

The reaction frame consisted of two 300 x 300 mm H-columns anchored to the foundation and joined with two 100 x 457 mm C-channels at the top as shown in Figure 2. The loading system was composed of an MTS actuator, with a load capacity of 45 kN and maximum stroke of 254 mm. The load was transmitted to the wall through a load-distribution beam made of steel box profile. To ensure a symmetric load application, two 50 x 100 mm steel C-channels forming a “fork” were used to transfer the load from the actuator to the mid-span of the distribution beam using steel pins. The wall was attached to the distribution beam with Ø16 mm lag-bolts at 610 mm o.c. Steel spacers (13 x 38 x 89 mm) were installed between the distribution beam and the top-plate at every lag-bolt location to simulate the presence of trusses. Lateral supports were provided using two 100 x 100 mm steel angles with ball-bearings on both sides of the distribution beam to prevent out-of-plane movement of the wall. At the bottom, the wall was attached to a glulam beam bolted with Ø16 -mm bolts at 610 mm o.c. to a steel box beam anchored to a concrete floor and to the bases of the columns. The wall was installed such that the supports did not interfere with the rotation of the sheathing panels.



**FIGURE 2:** Setup for racking tests.

The load applied by the actuator and the drift of the top-plate were measured, respectively, by a load cell and a linear variable differential transducer (LVDT, #10 in Figure 2) built in to the actuator. The relative movement between the sheathing panel and the wall frame was measured at the corners with eight LVDTs. LVDT #9 was attached to the stud at the foot of the wall to measure the uplift displacement. The walls were loaded at a constant rate of displacement of 0.25 mm/min according to ASTM E 564 standard (ASTM 2005). Measurements were recorded at a frequency of 10 Hz. Two wall-to-foundation attachments were tested as illustrated in Figure 3. The attachment with nails (3 – 16d at 406 mm) (N in Table 1, Figure 3a) represented the minimum International Building Code (ICC, 2003) requirements for conventional construction. The nails were staggered and the glulam beam was replaced after two consecutive tests. The second attachment (HD in Table 1, Figure 3b), representing an engineered wall in a high-risk seismic zone, was made of two hold-down anchors HTT 22 Simpson Strong Tie nailed to the end studs. Four Ø16 mm bolts at 610 mm o.c., with 6.4 x 64 x 64-mm plate washers were used to connect the bottom plate to the support. The holes for the bolts were oversized by 0.8 mm.



**FIGURE 3:** Top and bottom attachments used in racking tests.

### Bending Test-Setup

The support frame was built with two 254 x 305 x 2743 mm H-beams anchored to a concrete floor and equipped with roller bearings to support the wall ends as shown in Figure 4. A custom-made airbag with dimensions of 127 x 2235 x 2438 mm, rated for a maximum pressure of 34.5 kPa, was installed under the test wall with the sheathing facing the airbag to simulate the inward wind-pressure. The top and bottom plates of the wall were supported by built-up beams composed of one steel box beam (76 x 127 mm), one 38 x 140 -mm and one 38 x 89-mm lumber boards joined by eight SDS Ø6.4 x 76-mm Simpson Strong Tie® screws.

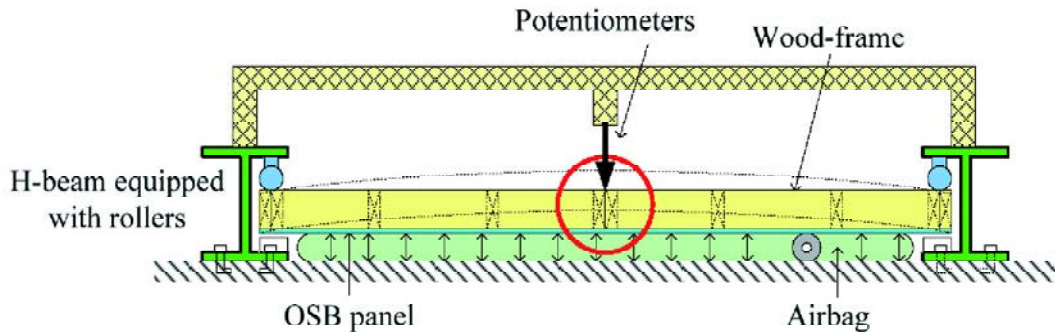


FIGURE 4: Setup for bending tests.

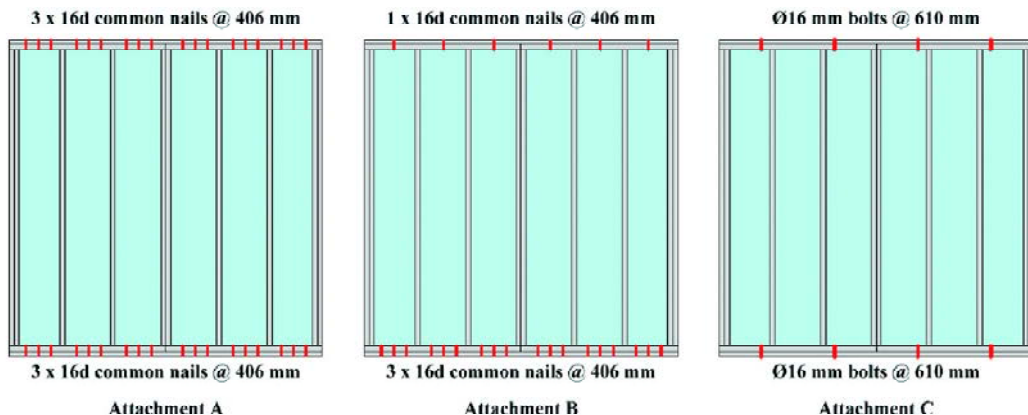


FIGURE 5: Wall-to-foundation attachments used in bending tests.

Figure 5 shows the details of the attachment of the top and bottom plates to the supports. The *Attachment A* was done by connecting the bottom and top plates to the “first” and “second” floor with three 16d common nails every 406 mm (ICC, 2003). *Attachment B* was similar to *Attachment A* at the base, but by using just one 16d common nail every 406 mm at the top-plate. Finally, the *Attachment C* was made by attaching the top and bottom plates to the “first” and “second” floors with Ø16 mm shear bolts at 610 mm.

Three linear potentiometers were mounted on a rigid frame attached to the support to measure the deflections of the studs along the length of the central connection between the wall segments. The deflections were measured near the top and bottom plates and at the middle of the central stud. The pressure was applied to the wall through the airbag at a rate of 0,06 kPa/s and was controlled by a system of four electronic valves (two to deflate and two to inflate). The internal pressure of the airbag

was measured with a differential sensor ranging from 0 to 34.5 kPa and with a precision of  $\pm 0.1$  kPa. The specimens were loaded from zero pressure until rupture and all data was collected at 10 Hz frequency. The wind pressure,  $P$  (kPa), was estimated using the following equation from the National Building Code of Canada (NBCC 2003):

$$P = q \cdot C_e \cdot C_g \cdot C_p \quad (1)$$

where,  $q = 0.00064645 (V^2)$ , reference velocity pressure, with  $V$  as the wind speed in metres per second;  $C_e = 1.0$ , exposure factor;  $C_g = 2.0$ , gust effect factor; and  $C_p = 0.8$ , external pressure factor. Table 2 shows the values of the wind pressure corresponding to the wind speed on the Saffir-Simpson Hurricane Intensity Scale.

**TABLE 2:** Saffir-Simpson Hurricane Intensity Scale.

Category	Wind-speed * (km/h)	Wind-pressure (kPa)	Damage potential
1 (Weak)	119 – 153	1.13 – 1.87	Minimal damage to vegetation
2 (Moderate)	155 – 177	1.92 – 2.50	Moderate damage to houses
3 (Strong)	179 – 209	2.56 – 3.49	Extensive damage to small buildings
4 (Very strong)	211 – 249	3.55 – 4.95	Extreme structural damage
5 (Devastating)	Greater than 249	> 4.95	Catastrophic building failures possible

\* From [www.nhc.noaa.gov/aboutshs.shtml](http://www.nhc.noaa.gov/aboutshs.shtml)

## RESULTS

As only one specimen was tested for each connection configuration, the results are to be interpreted as a trend. More tests should be conducted in order to consolidate these exploratory observations.

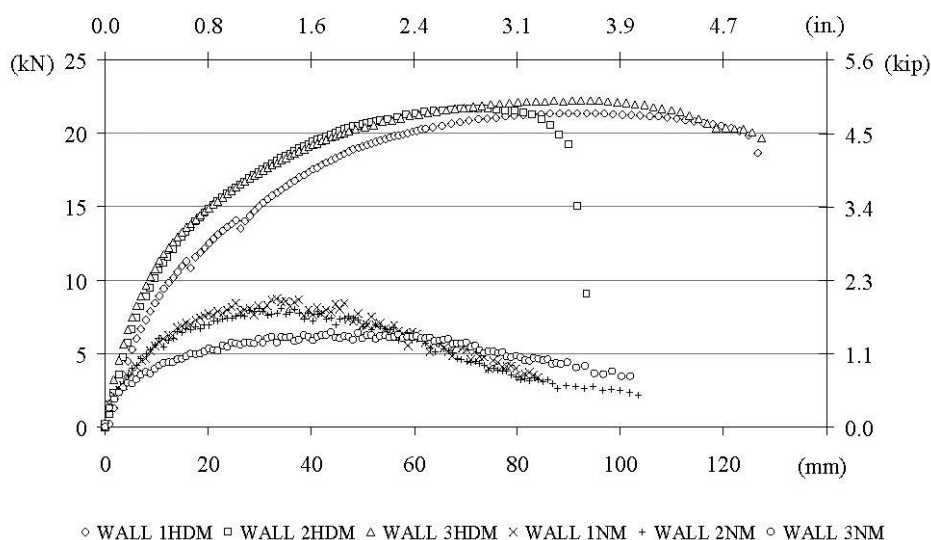
### • Static Racking Tests

Results of the static racking tests are summarized in Table 3 and the corresponding load-displacement curves are displayed in Figure 6. The performance parameters were determined according the ASTM E 564 and E 2126 standards. The elastic stiffness (at  $0.4P_{max}$ ) of Walls 2NM and 3NM, fabricated with Connections 2 and 3 and nailed to the base, was 23% higher than that of Wall 1NM, possibly due to the stiffer connections between the segments. Wall 3NM showed 50% higher displacement at failure and dissipated approximately 20% more energy than the two other walls. Apparently, Connection 3 offered the highest ductility and toughness to the assembly followed by Connection 2. However, the load capacities of the walls did not follow this pattern. Wall 1NM was the strongest (8.9 kN) followed by Wall 2NM (8.2 kN) and Wall 3NM (6.5 kN).

**Table 3:** Results of static racking tests.

Parameter		Specimens					
		1NM	2NM	3NM	1HDM	2HDM	3HDM
Maximum load	$P_{max}$ (kN)	8.9	8.2	6.5	21.4	21.7	22.2
Displacement at maximum load	$\Delta_{pmax}$ (mm)	38.4	34.5	44.0	90.8	71.9	91.6
Yield point	$P_{yield}$ (kN)	8.5	7.5	5.7	19.4	19.6	20.3
Displacement at yield point	$\Delta_{yield}$ (mm)	37.5	21.4	29.2	52.2	41.2	49.1
Load failure	$P_{failure}$ (kN)	7.2	6.6	5.2	17.1	17.3	17.8
Displacement at failure	$\Delta_{failure}$ (mm)	48.0	48.4	72.0	127.0	91.2	127.8
Elastic stiffness (at $0.4P_{max}$ )	$K$ (kN/mm)	0.53	0.70	0.66	0.94	1.18	1.27
Work to failure	$w$ (N·mm)	330	316	384	2236	1605	2412
Ductility ratio	$\Delta_{failure}/\Delta_{yield}$	1.3	2.3	2.5	2.4	2.2	2.6
Modulus of elasticity of framing	MOE (GPa)	9.78	11.0	11.0	10.6	11.0	11.1

The typical mode of failure observed for nailed walls was withdrawal of the nails attaching the bottom plate to the base (glulam beam). At the beginning of the test, the sheathing panels rotated and showed some displacements at the bottom plate, particularly in Positions 6 and 8 (see Figure 7, Nailed walls). In spite of this movement between sheathing panel and framing, the uplift measured at the stud near actuator was proportional to the deflection of the wall. Then, the failure began with the bending of the bottom plate, in the first segment, which provoked the gradual withdrawal of the nails from the end to the centre of the wall. The analysis of failure mode and sheathing displacements relative to the framing explains why the strength of these walls was likely independent of the connection between the segments. It is evident from this scenario that the interface connection between the two wall segments was not as critical for this type of relatively weak bottom plate to foundation nailed connection.

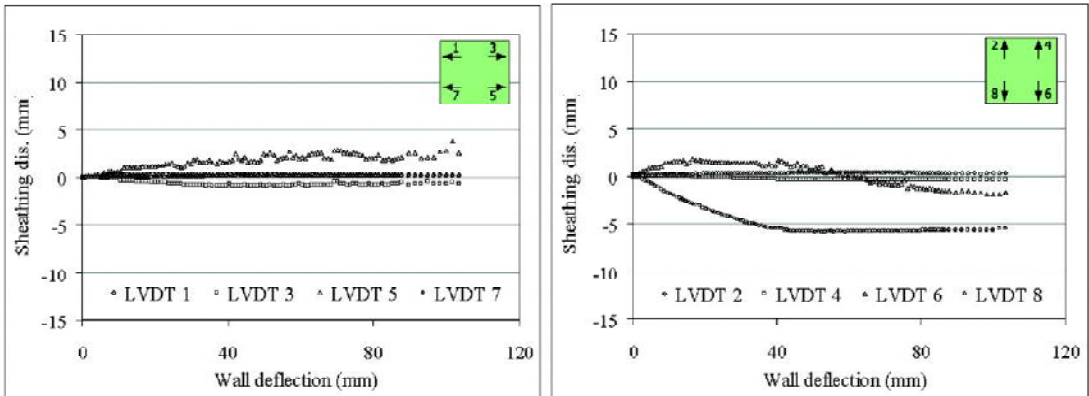


**FIGURE 6:** Load vs. displacement graphic for monotonic racking tests.

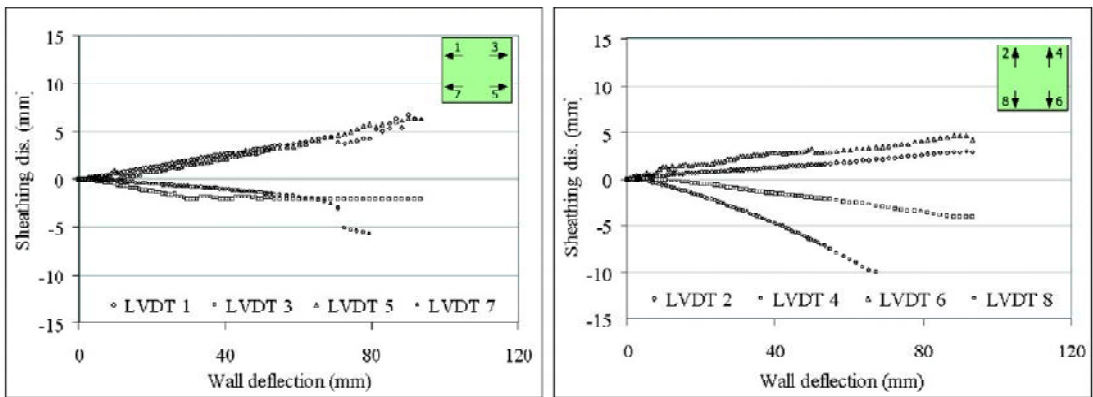
Figure 7a shows typical displacements of sheathing relative to framing near the corners of the walls nailed to the base. No significant work of sheathing-to-framing connections was registered, except for the bottom corner at the loaded end (LVDT #8) which indicated the separation of the sheathing panel from the bottom plate while the wall was pivoted as a rigid body. Therefore, the capacity of the wall assembly was greatly underutilized and the system resistance was restrained by the withdrawal of the nails attaching the bottom-plate to the support.

The average load capacity of the anchored walls was 21.8 kN with a variation of 2% between connection configurations. It means that the type of central connection between the segments did not affect the strength of these walls either. The elastic stiffness of Wall 1HDM was 24% less than that of Walls 3HDM and 2HDM. Wall 3HDM dissipated the largest amount of energy. Again, these results give a favourable comparison to the performance of Connection 3. The failure displacement of Wall 2HDM was 72% of the average for Walls 1HDM and 3HDM. Therefore, the work to failure of this wall was only 70% of the average work of the other two walls. This drop in the damping energy can be explained by the difference in the connections of the wall segments. In Connection 2, the sheathing is doubly nailed in the central zone (see Figure 1), giving an additional restraint to the rotation of the OSB panel. This double nailing, plus the large edge distance at the top plate, generated a non-symmetric system resisting the displacement which induced the rupture at the edges with a single nailing and short edge distances (bottom plate and chord) immediately after the maximum load was reached.





a) Nailed walls



b) Anchored walls

**FIGURE 7:** Sheathing-to-framing displacements at the corners.

Figure 7b shows typical displacements of the sheathing relative to the framing near the corners of the anchored walls with respect to the wall deflection. At every corner significant movements were produced by the transfer of the racking forces between framing and sheathing. The sheathing nails worked more uniformly in anchored walls compared to non-anchored ones (*NM*) which was translated in a more homogeneous work of the assembly.

For anchored walls, the typical mode of failure was nail head pull-through from the sheathing at the stud near the actuator and at the bottom-plate. Some nails tearing through the edge of the sheathing were also observed near the corner close to the actuator. This behaviour can be explained by the presence of hold-down devices that restricted the stud uplift and transferred the load to the foundation.

As a result, the anchored walls (*HD*) were on average almost three times stronger, 80% stiffer and dissipated six times more energy than the walls nailed to the base (*N*). The improved parameters found when testing walls anchored (*HD*) stand out the utilization of these devices in prefabricated housing installation.

• Bending Tests

Table 4 summarizes results of the bending tests and Figure 8 shows the deflections of the centre of the wall relative to the supports under increased wind pressure for wall specimens representing each attachment tested. Similar strength was observed between walls attached with the three types of central connections and the same attachment to foundation. The average capacity of walls with Attachments A, B and C were 6.55 kPa (CV=6%), 5.79 kPa (CV=6%) and 7.44 kPa (CV=13%) respectively. The walls with Attachment C resisted 14% higher pressure than walls with attachment A and 28% higher pressure than walls with Attachment B.

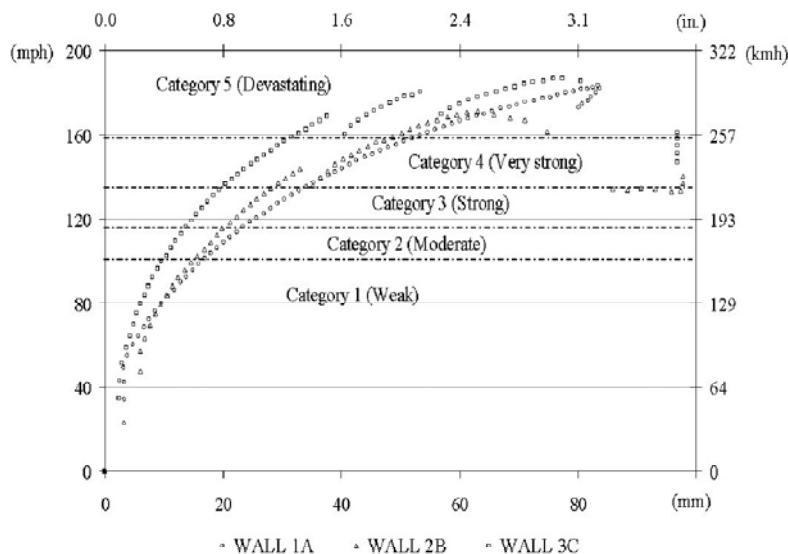
Initial stiffness was calculated between 0.69 and 2.76 kPa which corresponded approximately to 10% and 40% of maximum pressure ( $P_{max}$ ), respectively. Comparison of initial stiffness of tested walls shows that Attachment C (bolts) provided stiffer support than Attachments A and B (nails).

Values from Table 2 indicate that all walls resisted pressures equivalent to hurricanes of the 5<sup>th</sup> category. It is important to note that the wall length effect was not studied and that vertical gravity loads in plane would reduce the wall strength (P-Δ effect).

**Table 4:** Summary of the wall bending tests.

Parameters	Specimens								
	1A	2A	3A	1B	2B	3B	1C	2C	3C
Maximum pressure (kPa)	6.90	6.57	6.17	5.97	5.99	5.42	6.60	8.51	7.20
Maximum speed (km/h)	294	287	278	274	274	261	288	327	300
$\Delta_{Pmax}$ (mm)*	84	67	72	61	63	55	53	70	77
Initial stiffness (kPa/mm)	0.11	0.13	0.16	0.15	0.14	0.16	0.18	0.20	0.20
First rupture (kPa)	6.50	6.57	4.54	5.80	4.29	5.36	6.60	8.51	5.89
MOE of framing (GPa)	9.91	10.1	11.2	10.9	10.7	10.4	10.5	10.6	10.1

\*Displacement measured at the middle of the wall at the maximum pressure



**FIGURE 8:** Wind-speed vs. displacement for bending tests (representative walls).

Under out-of-plane load, the strength of studs governed the overall capacity of the tested walls. For all walls, the failure mechanism initiated when a field stud nearest the centre of the wall broke in bending (usually beyond 4.29 kPa of wind-pressure) which corresponded to 232 km/h sustained wind speed, followed by further disintegration of the wall assembly. Separation of the frame from the sheathing panel was typical for walls with the *Attachment B*. Such separation was controlled by the quantity of nailing of the top plate to the support. In this case, one nail every 400 mm and with a sheathing offset were not sufficiently strong to withstand the pressure concentrated on the central zone after the failure of the first stud as described above. These walls demonstrated the lowest load capacities. Walls with *Attachments A* and *C* were more resistant and the final failure was characterized by the separation of the central studs at the bottom or top plates, and rupture of other studs.

## CONCLUSIONS

The racking and bending tests were carried out on 15 full-size walls joined with three types of connections between the wall segments and attached with different types of fasteners to the foundation. The analysis of the load-deflection relationships and the comparison of failure modes of walls provided a better understanding of the performance of the different assemblies and suggested the following exploratory results.

For racking loads, the type of central connection between the wall segments did not have a definite influence on strength.

- For all types of inter-segment connections tested, the walls with hold-down anchors were nearly three times stronger than those nailed to the base. They showed 80% higher stiffness and dissipated five to seven times more energy before failure.
- For out-of-plane loads, the tested wall assemblies resisted wind-pressure beyond 4.3 kPa corresponding to 232 km/h sustained wind speed equivalent to Category 4 hurricane. Their strength was controlled by the strength of the studs rather than the type of the connections used.

The gathered information provides data for comparative quantitative analysis of conventional and engineered wall assemblies and is expected to be useful for the development of design methodology for lateral load resisting systems of prefabricated houses.

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