Experimental Behavior of a Full-Scale Housing Section Built with Cold-Formed Steel Shear Wall Panels under Horizontal Monotonic and Cyclic Loading

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Abstract: This paper presents the results of an experimental study on the behavior of the cold-formed steel shear wall panel (CFSSWP) with fibro-cement panels as sheathing, when it is subjected to in-plane shear deformations and flexural deformation under perpendicular monotonically increasing horizontal loads on the longest plane. A full-scale housing section was built with three walls and a ceiling using commonly used construction details in El Salvador. The strength and stiffness of the experimental specimen tested overcame significantly critical demand imposed by the technical design standards in this country. Additionally, a simplified finite element model was defined with the objective to analyze stresses in the components. The results of the numerical model were similar to the experimental model tested.

Keywords: seismic action; wind load; cold formed; finite element simulation

1. Introduction

Cold-formed steel constructions (CFS) are structures designed with small thickness cold-formed steel shapes, manufactured at ambient temperature. During the cold-rolling forming process, sheets are fed longitudinally through a series of rolls until they reach the desired shapes. Cold formed steel constructions (CFS) started to be a great success in the USA and Scandinavia in the mid-20th century. This structural typology possesses a lot of advantages in comparison with traditional materials such as concrete, where they distinguished the high strength/weight relationship, the speed of construction and the environmentally friendly material, for example.

The structural properties or shear walls constructed with cold-formed profiles were determined with tests carried out by Serette of the Santa Clara University [1,2]. The sheathing stabilizing properties in sections of cold-formed steel profiles had already been analyzed by Winter [3] of Cornell University. Simaan [4] continued his work. The design of the American US AISI Specifications [5] is based on the results of such investigations. Nevertheless, the model to include the sheathing stabilizing properties, by means of the shear wall stiffness, could not be verified with the latest tests [6].

In Europe, Eurocode 3 [7] specifies the design provisions for CFS members. Due to the need for understanding the behavior of these structures, especially under the effect of seismic loads, lateral resistance of CFS shear wall panels has recently been studied both experimentally [8,9] and analytically [9,10]. Pan and Shan [11] also performed monotonic tests on CFS wall panels and investigated the effect of sheathing material, sheathing...
thickness, and wall aspect ratio. Fiorino et al. [12] studied the typical screw connections used to attach sheathing boards on CFS framing members. The use of X-bracing flat strap on CFS wall panels has also been the subject of several studies [8,13–15]. Serrette et al. and Tian et al. [8,13] performed monotonic load tests on CFS wall panels utilizing flat-strap X-bracing. Kim et al. [14] performed shake table tests on a CFS structure with flat-strap X-bracing, and reported that the structure exhibited a ductile behavior. Tests done by Al-Kharat and Rogers [15] on CFS walls with flat-strap X-bracing revealed that the behavior of wall panels was mostly determined by hold-down details. Use of steel sheet as sheathing on CFS wall panels was investigated by Yu [16], and the failure of wall panels was determined to be primarily due to the buckling of steel sheet sheathing and pull out of screws.

In a typical CFS construction, in addition to being responsible for resisting lateral loads, CFS wall panels also form the gravity load resisting system of the structure. For this reason, the axial compression behavior of CFS studs and wall panels has also been investigated by several researchers [17–20]. These studies indicated that the existence of sheathing improved the axial compression behavior of CFS studs with the level of increase in load capacity being affected by sheathing material and screw spacing. The main objective of the study conducted by E. Baran and C. Alica [21] was to evaluate the performance of CFS wall panels using oriented strand board (OSB) as sheathing material and subjecting them to monotonic lateral loading.

Other similar design methods are unsuccessful in Germany. The intention is to develop a universal design method for shear walls subjected horizontal loads, based on completed research projects [22]. Among these investigations, the tests carried out by J. Lange and B. Naujorks [10], studied the behavior of the CFS wall panels, under the combination of vertical and horizontal loads, testing the model with different types of materials for the sheathing (agglomerate boards with polyurethane, fiber–plaster, wood–cement and trapezoidal sheets).

In the studies of M. Nithyadharan, V. Kalyanaraman [23] used calcium silicate panels, subjecting five of them to a monotonic deformation and another five to a reversible cylindrical in the cutting plane.

Important references are the studies about simplified analysis based on finite element methods and using as support software or computer software comparing experimental and analytical results. This is the case of Joel Martínez-Martínez and Lei Xub [24], who carry out a simplified approach to the construction of a finite element analysis in buildings that can take a long time, due to the large number of elements involved in modeling the structural part and the structural sheathing.

Currently, the research work done by Luis Simões da Silva [25] proposes a single-family housing solution promoting the use of a structure of cold-formed steel profiles. Additional works in the subject of seismically loaded cold steel panels to highlight are the performed by Fiorino et al. [26] and by Landolfo et al. [27,28], where they present a design procedure to design cold-formed steel housing panels based on seismic analysis.

After the 2001 earthquake in El Salvador, a collaboration between the El Salvador and the Japanese governments though the TAISHIN project [29] was established. The objective of this project was to promote the implementation of safer technologies in the house construction in El Salvador, using traditional materials. The work published in this article, additional to the TAISHIN project, was developed with the intention to define an alternative solution in steel, because of the advantages of this material as mentioned previously. The study performed consist in both experimental and numeral analysis of the proposed solution.

2. Full-Scale Specimen Section Test

2.1. Wall Panel Set Configuration

The definition of the experimental specimen was conditioned by the three factors described below:
As low-cost alternative buildings, the walls were made on a unique external fibrocement panel, which presented 14 mm of thickness, as recommended by the manufacturer. A purpose was made: a model formed only with three walls; one that would receive the load on it and two other perpendicular walls on the extremes, all side by side with a CFS system ceiling;

The critical requesting on the walls were applied perpendicularly to the bigger plan (6.10 m long) that was defined as the worst architectonic alternative or situation presented to the designed housing;

On the process of determination of the specimen dimensions some aspects were taken into account: building system and housing changes, the infrastructure and equipment limits of the “Laboratorio de Estructuras Grandes” (Big Structures Laboratory) and economic limits.

The components of the specimen at full-scale were finally identified and defined in this manner and represented on Figures 1–6:

Wall 1: 6.10 m long and 2.44 m high wall, built with five panels, 1.22 m × 2.44 m and 14 mm thick, with 11 vertical support profiles PE (section: 98.4 mm × 50 mm and 0.80 mm thick) every 0.61 m, two profiles PA (cross section dimensions 100 mm × 50 mm and 0.80 mm thick) an upper profile and a lower one, oriented to the north–south direction. (Shown on Figure 1);

Wall 2: 1.22 m long and 2.44 m high wall, built on a panel which incorporates three vertical support profiles PE of 0.61 m each, two profiles PA, an upper profile and a lower one, fixed to Wall 1 on the north extreme. (Shown on Figure 6);

Wall 3: 1.22 m long and 2.44 m high wall, built on a panel which has three vertical support profiles PE of 0.61 m each, two profiles PA an upper one and a lower one, fixed to Wall 1 on the south extreme. (Shown on Figure 6);

Ceiling: 6.10 m long and 1.22 m large Ceiling, built with two-and-a-half sheets, with six horizontal support profiles PE of 1.22 m each, fixed to the formed by wall 1, 2 and 3 (shown on Figures 4 and 5). On the free side of the ceiling, which would be 6.10 m long, it was necessary to put two intermediate supports for avoiding an excessive flexion on it.

The wind load was applied as a uniform load of 38.4 kg/m² to the shell items of the frontal wall. Knowing that the total surface of these items is 2.44 m × 6.10 m, the total load applied should be 570 kg.

Figure 1. 3D Views in full-scale of the experimental specimen: (a) Geometry properties; (b) Inside view.
Figure 2. Wall 1 of the experimental specimen at full-scale (east face).

Figure 3. Wall 1 of the experimental specimen at full-scale (west face).

Figure 4. Ceiling of the experimental specimen.
Figure 5. Gradient base floor of the experimental specimen.

Figure 6. Covered base floor of the ceiling of the experimental specimen in full-scale: (a) North face of wall 3; (b) South face of wall 3; (c) North face of wall 2; (d) South face of wall 2.
The experiment did not consider the application of vertical loads on the ceiling structure. The three walls of the model were fixed to a floor slab previously built and fixed to the reaction slab. This connection is completed through the lower profile PA, fixed to the floor slab with long expansion screws of 0.61 m, as recommended by the manufacturer.

The used screws had to have those characteristics:

- Screws for the attachment of the sheet on the steel framework. #8 × 1/2” extra flat anti-slip head, S-type tread, self-drilling drill point, LH 8-050;
- Screws for the attachment of the sheet on the steel framework. #8 × 1 1/4” bugle head, with self-countersinking, S12-type thread, self-drilling drill point PH 8-125;
- Foundation anchor screws, of 3/8” Diameter.

2.2. Seismic and Wind Critical Loads

Seismic and wind loads were analyzed considering national codes (Norma Técnica de Diseño por Sismo de El Salvador, NTDSES) [30] and Norma Técnica de Diseño por Viento. Reglamento de Seguridad Estructural de las Construcciones [31]). The intention is to evaluate the worst scenery and the design loads to analyze the feasibility of the proposed solution. In this section the calculations performed based on these standards are exposed in detail.

2.2.1. Seismic Critical Load

According to the Technique of Design for earthquakes Rule of El Salvador (NTDSES [30]), the seismic action could be analyzed by different methods. The objective of this calculation is to obtain an approximated value of the maximum expected load, so in this case and due to the characteristics of the structural system, the lateral force analysis procedure was considered. According to this method the base shear is calculated by empirical relationships to specify dynamical inertial forces as static forces, and by this standard it is computed by Equation (1):

\[ V = C_s W \]  

where \( C_s \) is the seismic coefficient and \( W \) is the weight considered in calculations (dead load and a percentage of the live load). The seismic coefficient, \( C_s \) is calculated with the following Equation (2):

\[ C_s = \frac{A I C_0}{R} \left( \frac{T_0}{T} \right)^{3/2} \]  

where \( A \) is the zone factor, \( I \) is the importance factor, \( C_0 \) and \( T_0 \) are the site coefficients depending on the soil properties, \( R \) is the reduction factor, and \( T \) is the fundamental vibration period of the structure subjected to study. Replacing the parameters, we obtained a seismic coefficient of 0.3. With this information the seismic weight \( W \) had an estimation of 500 kg, and the seismic base share, \( V \) would be 150 kg.

2.2.2. Wind Critical Loads

According to the Technique of Design for Earthquakes Rule of El Salvador (NTDSES) [31], the structures of short natural period, such as the housing ones, have to be revised to resist some effects, named as static pressure and static absorption. Dynamic effects do not affect structure of little slenderness, reason why the wind load can be simulated such as a static load. The static pressure design \( P \) is determined by the Equation (3):

\[ P = C_p C_z K P_0, \]  

where \( P \) is the static pressure design, \( C_p \) is the external pressure coefficient, \( C_z \) is the discharge coefficient, \( K \) is the topography factor and \( P_0 \) is the basic pressure which adopts a value of 30 kg/m².

Due to the considerations adopted, the static pressure design resulting is 38.40 kg/m², so the total strength of the wind over the long wall of the model would be \( V_W = 570 \) kg.
2.2.3. Critical Load

According to the values obtained and taking into account that “wind loads demand” results slightly higher to “the seismic loads demand”, the designed force for the experimental specimen was defined following the first one: 570 kg. For that reason, the experimental test was conducted on a cyclical way, and not on a reversible, paying attention precisely to the nature of the wind load.

2.3. Materials Tests

In this section the most important information related with the experimental program is presented. It involves the building material of the full-scale model test.

2.3.1. Cold-Formed Steel Profiles

To characterize the mechanical properties of the material of the cold-sheeted steel profiles, it was necessary to manufacture specimens designed according to the ASTM A-370 [32] standard. Table 1 presents the results obtained on the experiment of the two specimens obtained from the profiles in which the model was built. On this table, the values of yield strength, ultimate strength and the “elongation” percentage are registered from all the specimens.

Table 1. Results of the tension tests on the steel.

| Specimen | Yield Strength, $f_y$ (kg/cm$^2$) | Ultimate Strength, $f_u$ (kg/cm$^2$) | Elongation % |
|----------|----------------------------------|------------------------------------|-------------|
| 1        | 3164                             | 3895                               | 36          |
| 2        | 3276                             | 3768                               | 32          |

The standards used on our middle to define the effort properties of yield strength and last effort are: 2320 kg/cm$^2$ and 3150 kg/cm$^2$ respectively, and the average values exceed in 1.4 and 1.2 times those standards, respectively.

2.3.2. Fibrocement Panels

The characterization of the mechanical properties of the material’s panel were performed according to ASTM E72-02 [33] and ASTM C551/C551M-07 [34] standards. The flexion experiments were performed in two specimens one removed from the sheets of the model building, that were on their extremes and suffered a concentrated load on the middle of its length. The force–displacement relationship was obtained by a load cell and a ring load (see Figure 7).

![Figure 7. Gradient base floor of the experimental specimen.](image)
The breakup unit, $\sigma_r$, is calculated evaluating the bigger $\varepsilon$ on the section of maximum moment for the failure load, through the mathematical expression of Equation (4):

$$
\sigma_r = \left( \frac{M}{I y} \right)_{\text{max}} = \left( \frac{\frac{P_{\text{max}} L}{4}}{b \frac{t^3}{12 \varepsilon}} \right) \frac{t}{2} = \frac{3 P_{\text{max}} L}{2 b t^2}
$$

(4)

where $P_{\text{max}}$ is the failure load, and $b$, $L$, and $t$ are, the large, long and thick measures of the specimen tested, respectively. The results obtained for the breakup unit of the specimen tested are presented on Table 2.

Table 2. Results of the experiment tension on the fibrocement panel.

| Cylinder | L (mm) | b (mm) | t (mm) | $P_{\text{max}}$ (kg) | $M_{\text{max}}$ (kg-cm) | $I$ (mm$^4$) | $\sigma_r$ (kg/cm$^2$) | $\sigma_{r\text{ average}}$ (kg/cm$^2$) |
|----------|-------|--------|--------|----------------------|-------------------------|-------------|------------------------|-------------------------------|
| 1        | 312   | 27.9   | 13.6   | 10.2                 | 79.7                    | 5848.4      | 92.7                   | 93.4                          |
| 2        | 315   | 28.6   | 13.5   | 10.4                 | 82.1                    | 5903.1      | 94.1                   | 93.4                          |

Likewise, to evaluate the Young modulus, an electric strain gauge was glued on the lower central zone of the cylinder, to relate the flexion effort with the unit deformation. To evaluate the elasticity unit, the flexion effort was calculated on the higher momentum zone, precisely where the stain gauges were situated, making loads, $P$ and the $\varepsilon$ on the elastic zone of the material. The expression used is presented below, see Equation (5), and the results obtained are registered on Table 3.

$$
E = \frac{\sigma}{\varepsilon} = \left( \frac{\frac{P_{\text{max}} L}{4}}{b \frac{t^3}{12 \varepsilon}} \right) \frac{t}{2} = \frac{3 P L}{2 b t^2} \frac{1}{\varepsilon}
$$

(5)

Table 3. Results of the experiment tension on the fibrocement panel.

| Cyli. | L (mm) | b (mm) | t (mm) | $P_{\text{max}}$ (kg) | $M_{\text{max}}$ (kg-cm) | $I$ (mm$^4$) | $\sigma$ (kg/cm$^2$) | $\varepsilon$ (e-6) | $E_{\text{average}}$ (kg/cm$^2$) | $E_{\text{average}}$ (kg/cm$^2$) |
|-------|-------|--------|--------|----------------------|-------------------------|-------------|------------------------|----------------|-----------------------------|-----------------------------|
| 1     | 312   | 27.9   | 13.6   | 10.2                 | 79.7                    | 5848.4      | 71.9                   | 1831.1 | 39,298                      | 38,822                      |
| 2     | 315   | 28.6   | 13.5   | 10.4                 | 82.1                    | 5903.1      | 73.3                   | 1965.1 | 38,345                      | 38,245                      |

Taking into account the information provided by the manufacturer, the lower breakup strength value specified would be about 70 kg/cm$^2$, and the values of the elastic modulus would be between 25,000 kg/cm$^2$ and 40,000 kg/cm$^2$. As we can observe, the values obtained for the breakup strength are higher than 70 kg/cm$^2$ (shown as well on the Table 2) and the values obtained for the elastic modulus are into the specified rank (shown also on the Table 3).

3. Experiment Details

3.1. Instrumentation Test Setup

3.1.1. Location of Displacement and Control Transducers (CDP)

Taking into account the structural behavior observed on the numeric model developed, a displacement of 0.9 cm was selected for the experiment control corresponding to the upper extreme part of wall 1. Additionally, the selected displacement was the higher expected on any load cycle and it was also used to define the stiffness model. We decide to put three displacement transducers as close as possible from the control displacement. These transducers are able to register values from 10 mm to 25 mm or 200 mm, and furthermore to register displacements on the different behavior steps of the model: elastic and inelastic. In Figure 8, a diagram is presented showing the order of the different displacement transducers, ordered on the west face of wall 1. In addition to the three transducers mentioned
before, six more were also placed, equipped with measuring capacities of 100 mm and another five with measuring capacities of 50 mm.

![Figure 8. Location and mediation capacities of the displacement transducers.](image1)

Finally, with the purpose of corroborate the non-existence of the relative displacements between the anchors of the model and the foundation beam, other two displacement transducers with measuring capacities of 10 mm were placed on the model making contact with the two central anchors of the east face of wall 1. The aforementioned are presented in Figure 9.

![Figure 9. Displacement transducers in anchors to the foundation.](image2)

3.1.2. Strain Gauges Location

The strain gauges were placed trying to be uniformly distributed on the steel profiles and the fibrocement sheets to capture the expected behavior of the numerical model. They captured the behavior of the PE’s base and the ceiling structure; the behavior of PA, which is located on the upper extreme of wall 1 between wall 2 and 3, and the behavior of the fibrocement sheets between the PE.

Due to the limits of the way the loads were applied, modelled in 8 concentrated and distributed loads, it was planned that the strain gauges situated on the center of the sheets
would have difficulties to collect the sheet behavior; anyway, the decision was to maintain the strain gauges location thinking of the possibility of collecting the unexpected behavior changes in case of local failures, for example, detachments of the screws of excessive deformations (see Figures 10–13).

Figure 10. Strain Gauges identification on Wall 1 (east face).

Figure 11. Strain Gauges identification on Wall 1 (west face).

Figure 12. Strain Gauges identification on Wall 2: (a) North face; (b) South face.
3.2. Wall Panel Set Loading Protocols

On the LEG, the load system is used on the experiment to apply the load perpendicularly to the longest plane of one of the walls. The pressure is applied through a hydraulic jack system, fixed to frames that would distribute the loads through neoprene pads which would be located on one of the jack extremes (Figure 14). The hydraulic jacks are able to apply a force of nearly 5000 kg, distributed on neoprene pads with a contact surface of 30.0 cm × 30.0 cm. The hydraulic jacks were connected to the hydraulic hose, and it is through them that the pressure would be applied, because a manual activation pump, and the pressure applied would be measured through the pressure cells (as shown on Figures 14 and 15). The load system was designed to adjust eight hydraulic jacks, on a way that the distribution of the applied loads on wall 1 would obey to the pattern schematized on Figure 8.

![Figure 13. Strain Gauges identification on Wall 3: (a) North face; (b) South face.](image)

![Figure 14. Load system acting on the specimen: (a) General view of the load frame; (b) Detail view of the load system.](image)
Figure 15. Hydraulic pump, hoses and pressure cells.

It was decided to take the experiment to the most gradual manner, taking into account that the critical demand, the wind load, was 570 kg, and the corresponding displacement to that demand obtained with the analytical model was 8.97 mm.

Every load cycle was identified as the step of the experiment composed by two phases: the first one in which the load would be applied as much as possible to get as close as possible to the nominal displacement expected, identified as load phase. Additionally, a second one, in which the model would be totally unloaded, was identified as unload phase. Before the elaboration of the experiment, it was planned to prepare 12 load cycles, two for every nominal displacement of 10 mm, 20 mm, 40 mm, 60 mm, 80 mm and 100 mm, and to decide about the following cycles taking into account the behavior shown on them.

3.3. Full-Scale Specimen Section Set Results
3.3.1. Record of Displacements and Loads

The results obtained from the full-scale test are presented in this section. Table 4 summarizes the most relevant information related to the history of the displacements and loads corresponding to the ones deduced on the development of the test, where the displacements correspond to the control displacement, that means to the center of the highest extreme of Wall 1 and the loads on the total external force practiced by the eight hydraulic jack system.

| Load Cycle | δni (mm) | ∆ni (mm) | ∆oi (mm) | Poi (kg) | ∆i (mm) | Pi (kg) | Notes |
|------------|----------|----------|----------|---------|---------|--------|-------|
| 1          | 1.0      | 1.0      | -        | -       | 1.2     | 75.7   |       |
| 2          | 1.0      | 1.0      | -0.1     | 0.6     | 1.0     | 74.9   |       |
| 3          | 2.0      | 2.0      | -0.1     | 0.9     | 1.9     | 99.5   |       |
| 4          | 2.0      | 2.0      | 0.2      | 0.2     | 3.1     | 129.3  |       |
| 5          | 4.0      | 4.0      | 0.9      | 1.9     | 3.9     | 152.3  |       |
| 6          | 4.0      | 5.5      | 1.1      | -5.9    | 5.4     | 188.2  |       |
| 7          | 6.0      | 7.2      | 1.2      | -2.5    | 7.4     | 240.8  |       |
| 8          | 6.0      | 7.9      | 1.9      | 7.4     | 7.7     | 247.2  |       |
| 9          | 8.0      | 9.9      | 1.9      | 5.9     | 8.3     | 295.0  |       |
| 10         | 8.0      | 11.0     | 2.9      | 25.5    | 11.2    | 329.0  |       |
| 11         | 10.0     | 13.3     | 3.2      | 23.2    | 13.3    | 367.9  |       |
| 12         | 10.0     | 14.0     | 3.9      | 6.2     | 14.1    | 390.3  |       |
| 13         | 20.0     | 20.0     | 4.4      | 5.2     | 20.4    | 533.0  |       |
| 14         | 20.0     | 20.0     | 6.8      | 8.8     | 19.9    | 505.9  |       |
| 15         | 25.0     | 25.0     | 7.0      | 0.1     | 23.4    | 693.5  |       |
| 16         | 25.0     | 25.0     | 8.8      | 16.3    | 24.8    | 908.0  |       |
| 17         | 30.0     | 30.0     | 9.1      | 30.1    | 30.1    | 1054.7 |       |
| 18         | 30.0     | 30.0     | 11.3     | 31.6    | 29.9    | 1037.9 |       |
| 19         | 40.0     | 40.0     | 9.4      | 41.8    | 39.3    | 1361.2 |       |
| 20         | 40.0     | 40.0     | 15.4     | 22.4    | 40.1    | 1292.4 |       |
| 21         | 50.0     | 50.0     | 16.5     | 27.5    | 50.1    | 1592.8 |       |
| 22         | 50.0     | 50.0     | 19.9     | 0.0     | 50.0    | 1502.5 |       |
| 23         | 60.0     | 60.0     | 21.7     | 29.0    | 55.0    | 1660.1 | Sheathing failure |

where: δni: rise of the nominal displacement proposed for the current load cycle. ∆ni: nominal displacement proposed for the current load cycle. ∆oi: permanent deformation on the precedent load cycle anterior, and built on the initial deformation of the current load cycle. Poi: load corresponding to ∆oi. ∆i: deformation corresponding to the end of the load face of the current cycle. Pi: load corresponding to ∆i.
From the 6th to the 12th cycle, the nominal displacements proposed were oscillating around the target of 9.0 mm calculated. At the end, 22 complete cycles were realized, arriving to impose displacements of nearly 5.0 cm with loads higher than 1500 kg, and the failure of the model appeared on the load phase corresponding to the 23th cycle, with a displacement of nearly 5.5 cm and a load of 1660 kg approximatively.

3.3.2. Failure Modes in Monotonic and Cyclic Loading Tests

The failure sequence observed during the execution of the test is summarized below:

1. From start to the load cycle 6, no damage was observed at all on the model;
2. At the end of the load cycle 7, a rotation on the vertical profiles and a sprain of the anchor profile was observed on the wall 1 base (shown on Figure 16a). A short separation between the intermediate vertical steel sheet profiles and the fibrocement sheets were also detected (shown on Figure 16b);
3. At the end of the load cycle 12, torsional buckling of the higher anchor profile PA of wall 1 could be appreciated, more evident on the central section (Figure 16c).

![First identified failures modes: (a) Rotation of the anchor profile; (b) Separation between of the sheet-profile; (c) Torsion of the profile PA.](image)

4. At the end of the load cycle 18, in front of slightly high to the 1000 kg load, an elevation of 6.0 mm or less was evident uniformly on the length of the wall 1. The walls 2 and 3 presented variable elevations, near to the 6.0 mm on the extremes that joined to the wall 1, and simultaneously they reduced on the other extremes. This deformation pattern was incremental. At the end of the load cycle 22, because of the use of a nearly 1500 kg load, an elevation of 8.0 mm on the wall 1 was registered (see Figure 17).
5. At the end of the load cycle 19, a fissure was generated on one of the fibrocement sheets on the east face of wall 1, identified as Crack G1, that will spread and widen to later stages (shown on Figure 18). This fissure appeared close to one of the neoprene pads of the hydraulic jacks and to the vertical sheet profile.

6. The ceiling structure flexed itself to the top, on a way that makes it lose contact with the other two vertical posterior supports given (as shown on Figure 19). Separations start to appear between the steel and the fibrocement sheets on the intersections of wall 1 with the transversal walls, that were rising on later load cycles (shown on Figure 20);
7. At the end of the load cycle 20, a failure is presented in another fibrocement sheet of the east face of wall 1, identified as crack G2, that will expand and spread on the following cycles (Figure 20). This crack appears also next to one of the neoprene pads of the hydraulic jacks, but there was not a vertical steel profile next to it as happened with crack G1;

8. At the end of the load cycle 21, due to a load of nearly 1600 kg, crack G3 and G4 appeared on the west face of wall 1. Both cracks correspond to crack G1 and G2, respectively;

9. At the end on the load cycle 22, the rotation of wall 1 compared to the base is clear and produces horizontal displacements of the ceiling structure of about 60 mm. Walls 2 and 3 deform themselves to adapt themselves to the spin. The vertical displacement of the ceiling on the pillars is nearly 20 mm;

10. At the end on the load cycle 23, the collapse of the fibrocement sheet happens, on the place that G2 and G4 had appear (shown on Figure 21);

11. As a consequence of the crash of the sheet, the adjacent contact profiles PE suffered a permanent deformation, and the anchor profile PA of the base suffered a considerable separation to its base.

3.3.3. Relationship between the Load–Displacement Obtained

Figure 22a presents the load–displacement relationships that were obtained on the different load cycles of the experiment, where the load corresponds to the total applied load and the displacement corresponds to the center is the upper extreme of wall 1. Otherwise, Figure 22b presents the involving data corresponding to it.
Figure 22. Cyclic response: (a) Load-displacement relationships on the different load cycles; (b) Involving curves of load displacement.

Table 5 presents the average values obtained in all load phases of the experiment and represent the stiffness of the full-scale model in every phase. All average values of every group of load cycle measured with different CDP are registered. So, we can observe that there is a notable difference between the second group and the other two.

Table 5. Record of the shifting and loads on the experiment development.

| Load Cycle | Δoi (mm) | Poi (kg) | Δi (mm) | Pi (kg) | Ki (kg/cm) | CDP | K Average (kg/cm) |
|------------|---------|---------|--------|--------|---------|-----|------------------|
| 1          | -       | -       | 1.2    | 75.7   | 631     | 10  |                  |
| 2          | -0.1    | 0.6     | 1.0    | 74.9   | 658     | 10  |                  |
| 3          | -0.1    | 0.9     | 1.9    | 99.5   | 494     | 10  |                  |
| 4          | 0.2     | 0.2     | 3.1    | 129.3  | 448     | 10  |                  |
| 5          | 0.9     | 1.9     | 3.9    | 152.3  | 507     | 10  |                  |
| 6          | 1.1     | -5.9    | 5.4    | 188.2  | 454     | 10  |                  |
| 7          | 1.2     | 2.5     | 7.4    | 240.8  | 384     | 10  |                  |
| 8          | 1.9     | 7.4     | 7.7    | 247.2  | 408     | 10  |                  |
| 9          | 1.9     | 5.9     | 8.3    | 295.0  | 452     | 10  |                  |
| 10         | 2.9     | 25.5    | 11.2   | 529.0  | 369     | 25  |                  |
| 11         | 3.2     | 23.2    | 13.3   | 567.9  | 344     | 25  |                  |
| 12         | 3.9     | 6.2     | 14.1   | 390.3  | 379     | 25  |                  |
| 13         | 4.4     | 5.2     | 20.4   | 533.0  | 330     | 25  |                  |
| 14         | 6.8     | 8.8     | 19.9   | 509.9  | 376     | 25  |                  |
| 15         | 7.0     | 0.1     | 23.4   | 609.3  | 373     | 25  |                  |
| 16         | 8.8     | 16.3    | 24.8   | 908.0  | 557     | 200 |                  |
| 17         | 9.1     | 30.1    | 30.1   | 1054.7 | 486     | 200 |                  |
| 18         | 11.3    | 31.6    | 29.9   | 1037.9 | 538     | 200 |                  |
| 19         | 9.4     | 41.8    | 39.3   | 1361.2 | 441     | 200 |                  |
| 20         | 15.4    | 22.4    | 40.3   | 1292.4 | 513     | 200 |                  |
| 21         | 16.5    | 27.5    | 50.1   | 1592.8 | 467     | 200 |                  |
| 22         | 19.9    | 0.0     | 50.0   | 1502.5 | 498     | 200 |                  |
| 23         | 21.7    | 29.0    | 55.0   | 1660.1 | 490     | 200 |                  |

4. Finite Elements Model Details

As commented previously, before defining the experimental program, it was necessary to elaborate a numerical model to predict the structural behavior and resistance expected. The results obtained gave criteria for the instrumental phase, helping to define the location and needing some of the measures instruments. The numerical model (see Figure 23a) was developed with the help of the SAP 2000 software [35], taking into account the following considerations:

- Steel profiles were modelled as frame elements. Nominal values were used on the cold-formed steel to define its properties: elasticity unit ($E_s = 2.074 \times 10^6$ kg/cm$^2$), yield strength ($F_y = 2310$ kg/cm$^2$), Poisson’s ratio ($\mu = 0.30$), volumetric weight ($\gamma_s = 7.849$ kg/cm$^3$) being the physical properties of each profile; PA (d = 10.00 cm, b = 3.20 cm, t = 0.80 cm), PE (d = 9.84 cm, b = 5.00 cm, t = 0.80 cm);
• Fibrocement sheets were modelled as shell elements, combining the plate and membrane behavior. Nominal values given by the manufacturer were used: elasticity unit \(E_{fc} = 33,130\ \text{kg/cm}^2\), Poisson’s ratio \(\mu = 0.20\), volumetric weight \(\gamma_{fc} = 1.100\ \text{kg/cm}^3\), being the properties of this section: the thickness for the plate behavior \(t_{pl} = 1.40\ \text{cm}\), and the thickness for the membrane behavior \(t_{m} = 1.40\ \text{cm}\);
• Connections between the steel sheets were considered as pinned connection and they were only retrained to displacements and not to rotations;
• Screwed connections between profiles and panels were modelled as rigid links, to the profile’s centroidal axis to the sheet’s centroidal axis. When there was not relative displacements or deformations in this connection, the links had the goal of applying similar actions as the interface of the two structural component screws did;
• Wind load was considered and applied on a uniform load of 38.4 kg/m\(^2\) to the shell items of the frontal wall.

Figure 23. FEM model: (a) SAP 2000 FEM model of the experimental specimen; (b) Deformation on the central profile PE of the FEM model.

The most important results obtained with the FEM model (Figure 24a) are shown following.
The higher experimented deformation shown by the system was on the center of the frontal wall, on its upper extreme, resulting to be 0.897 cm;  
The stiffness of the system, defined in this particular case as a total applied load (571.55 kg) and the higher experimented deformation by the system being 637 kg/cm;  
The profiles PE rotate from the base and deform on the same way as the steel beam. Figure 24b shows the deformation scaled from the central profile PE.  
The upper profile PA suffers nearly the same displacements on the length of it, and its deformations, even if they are very little, are similar to the experimented by the standing beam. Figure 24a shows this deformation;  
Figure 24b shows the variation of bending moments on the direction that they apply the load to the profiles PE, that is, the horizontal direction.

5. Discussion, Evaluation and Comparison of the Results of Analytical and Experimental Models

5.1. Resistance Parameters and Criteria

The critical demand imposed to the model corresponds to the wind condition, that was estimated on 570 kg. The experimental model was able to resist a maximum load of 1660 kg, while developing the load phase of the 23th cycle.  
The crack on the model was due to a breakup on the fibrocement sheet that was supporting one of the hydraulic jacks, one of those that applied the load. The crack presented on the thin load level was attributed to the concentration of the applied load from hydraulic jacks to the fibrocement sheets and not on how wind acts on the structure.  
According to these results, a security factor of 2.90 was calculated for both experimental results and numerical analysis.
5.2. Stiffness Parameters and Criteria

The stiffness of the model is expressed as the quotient between the full load applied and the control deformation observed. This deformation has a high modification depending on the load level applied, from the previous load–deformation record and the measuring instrument used on the load step considered.

Table 6 presents a comparison of the experimental specimen stiffness, calculated as the stiffness average of the 23 load cycles, with the numerical model calculated stiffness considered on the different values of the elasticity unit of the fibrocement sheet: the bigger value and the lower specified by the manufacturer, the average of both of them and the value obtained on the research project of the fibrocement sheet of the experiment. All the obtained stiffness values with the analytic model overestimate the stiffness values of the experimental specimen, even if they are situated on the same order scale.

| Specimen            | Detail                                                      | Stiffness (kg/cm) |
|---------------------|-------------------------------------------------------------|-------------------|
| Experimental specimen | Average of the stiffness values obtained on the different load phases | 461               |
|                     | Higher elasticity unit specified by the manufacturer E = 40,816 kg/cm² | 692               |
| Numerical specimen  | Lower elasticity unit specified by the manufacturer E = 25,510 kg/cm² | 578               |
|                     | Average elasticity unit specified by the manufacturer E = 33,163 kg/cm² | 637               |
|                     | Elasticity unit obtained on the experimental program E = 38,822 kg/cm² | 678               |

5.3. Structural Behavior Criteria and Parameters

The experimental specimen has a similar behavior as expected by the numerical model. Vertical supports of the Wall 1 rotated from their base and bended, simultaneously with the standing beams transversally loaded. The deformation pattern expected for the upper anchor profile, and the behavior pattern of wall 2 and 3, which was the transversal support of wall 1, was also observed. The state of the simple support on the base reproduces perfectly the building process, as the only restriction to the spin of the vertical steel profiles is this torsional stiffness of the anchor profile of the base (shown on Figure 25) and this stiffness is nearly minimal on opened sections. Nevertheless, the perpendicular walls collaborate in the increasing of the stiffness of the module.

Figure 25. Diagram of the deformation experimented by the specimen.
On the other hand, the rising permanent deformations appearance due to the increasing loads and displacement were remarkable. The origin of these permanent deformations was on the local buckling of some of the components of the transversal section of the thin sheet profiles and the separations that would appear between the steel profiles and the fibrocement sheets as the applied loads would be increased.

Finally, compared to the registered information of the strain gauges, that gave no evidence of damage of strange behavior, we could confirm that the experimental model behaved effectively in a similar way as predicted by the numeric model and was observed on the experimental specimen.

On Figure 26, two charts are presented, corresponding to the strain gauges that provided information about the experiment and registered values of strain comparing with the experiment step. Figure 26a presents the charts of strain of the strain gauges identified as 1WV2, 1WV9 and 1WV16. Figure 26b presents the ones which are identified as 1EV9 and 1EV16 (and the one identified as 1EV2, presented evidence of damage).

![Figure 26. Strain of strain gauges: (a) strain of the strain gauges identified as1WV2, 1WV9 and 1WV16; (b) The charts of strain of the strain gauges identified as 1EV9 and 1EV16.](image)

According to the location of the strain gauges shown on Figures 11–13 and the behavior expected, the prevision was that the first group registering compressions, and the ones from the second group located on the east face registered tensions. Expansions and reductions are consistent on the bending moment that would have been generated on the vertical direction on these zones.

6. Conclusions

The article presents the details and results of the experimental and numerical study of the section of a housing built in full-scale on a cold-formed steel structure and fibrocement panels as coating, submitted to monotonic and cyclic loads perpendicular to the longest plane. The strength, stiffness and structural behavior conditions would be verified from a full-scale model, comparing with a numerical model. From the results obtained the following conclusions can be obtained:

- The panel system with one single lining formed by fibrocement sheets and thin steel sheets is satisfactory for the use as a one floor housing principal structure. The experimental model strength reaches the critical demand imposed by the technical design rules for earthquakes and wind of El Salvador. Unless under special conditions, this typology of structure is safety for the wind and seismic demands;
- The numerical model behavior is very similar to the experimental specimen tested. Therefore, the structural analysis developed by simulation can be used on a reliable way to model different alternatives;
• The resistance levels offered from the manufacturers are enough reliable to use them as structural revision criteria on the alternative design process, since the manufacturing process of the considered material is industrialized;
• The stiffness obtained from the numerical model was higher than the ones obtained on the experimental specimen. For that reason, on the case of structural simulation the deformations obtained can be under-estimated;
• The permanent deformations registered on the experiment are caused by local buckling of the components of the transversal section of the steel profiles and the separations between these profiles and the fibrocement sheets that appear when the loads are raised. The profiles have a post-buckling resistance reservation, where some parts of the profile conserve their full capacity of resistance and the possibility of using this property on structures even if it exists a partial failure on them;
• The deformations appeared on the ceiling are considered excessive due to some of the load values. The revision and the improvement of the structural system should be considered as future steps.

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