Monotonic and Cyclic Loading Tests for Cold-Formed Steel Wall Frames Sheathed with Calcium Silicate Board

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Abstract

This research is concentrated on the structural strength and behavior of cold-formed steel wall frame sheathed with calcium silicate board under shear load. Test specimens were assembled with two different thicknesses of sheathing, 9 mm and 12 mm, and one-side or two-side of attachment. Monotonic shear and cyclic loading tests are conducted on wall specimens utilizing two C sections connected back-to-back to be as chord studs and calcium silicate board sheathing on the exterior. Based on the test results, detailed discussions on the strength, stiffness, energy absorption, ductility ratio, and failure mode of cold-formed steel wall specimens are given. It is noted that the failure mostly occurred at the bottom track of wall specimens due to the large deformation or tearing failure of track. The wall strength is not affected by the change of sheathing's thickness significantly, but wall frames attached with two-side calcium silicate board sheathing provide higher resisting strength and stiffness than those attached with one-side sheathing. In this study, test results are also used to compare with previous study that single chord stud was used in the assembly of wall frame. In addition, the suggested response modification factor of the wall sheathed with calcium silicate board is proposed for design purpose.

Keywords: Cold-formed steel, Wall frame, Shear test, Cyclic load, Response modification factor

1. Introduction

Cold-formed steel (CFS) structure has been widely used in the building construction industry due to its
ease of assembly, high strength, and low cost. There are many researches were conduct to study the behaviors of CFS wall frame, and these walls are mainly attached with Oriented Strand Board (OSB) and gypsum board sheathing. Because of higher shear resistance capacity character, using steel sheet as a sheathed material for CFS wall frame becomes more popular in the building construction. The lateral design of CFS wall framing using plywood, OSB, gypsum board, and steel sheet can be found in the AISI S213 standard [1]. Because there is not much supporting information about the design of CFS wall frame using calcium silicate board (CSB) as a cladding, this research focused on the structural strength and behavior of cold-formed steel wall frame sheathed with calcium silicate board under shear load. Two different thicknesses of sheathing materials, 9 mm and 12 mm, are used in the assembly of wall specimens. Both monotonic and cyclic loading tests were conducted in the test program. The strength, stiffness, energy absorption, ductility ratio, and failure mode of each test specimen are discussed and presented in this paper. To obtain the seismic design parameter for the CFS wall frame sheathed with CSB, the response modification factor is also investigated and proposed in this study. Pan and Shan [2] studied the structural strength of cold-formed steel wall frames with sheathing under monotonic shear loading. Two aspect ratios, 1.0 and 2.0 were utilized in the design of wall specimens. Three different kinds of sheathing material, gypsum board, calcium silicate board, and oriented-strand board, with two different thicknesses (9 mm and 12 mm) were adopted in the test specimens. It was found that the ultimate strengths of specimens with one-side sheathing are about 50% less than those of specimens with two-side sheathing for the wall frames having the same aspect ratio. And the ultimate strengths of specimens having aspect ratio of 2.0 are about 35% less than those of specimens having aspect ratio of 1.0 for the wall frames with the same sheathing configuration. From the test results, it shows that the mechanic properties of sheathing material influence not just the specimen’s loading capacity, but also the structural behavior. The specimens with gypsum sheathed have the lower values of ultimate load and energy absorption as compared with the specimens with either calcium silicate board or oriented-strand board. However, the specimens with gypsum sheathed have the higher values of ductility ratio. Xu and Martínez [3] investigated an analytical method to determine the ultimate lateral strength of the shear wall panel and its associated displacement. The method takes into account the factors that primarily affect the behavior and the strength of the shear wall panel, such as material properties, geometrical dimensions and construction details. The comparisons made on the results obtained from
the proposed method and extensive experimental tests. They found that the proposed method yields a better accuracy in the lateral strength than the displacements.

Fulop and Dubina [4] studied a 12 ft × 8 ft cold-formed stud frame. The experimental shear walls included frame without sheathing, frames two-side sheathed with gypsum board and corrugated sheet, frames one-side sheathed with OSB and corrugated sheet, frames one-side sheathed with opening on OSB and corrugated sheet, and frames braced with steel strap. Each series consisted of identical wall panels tested statically, both monotonic and cyclic. Results showed that the shear-resistance of wall panels is significant both in terms of rigidity and load bearing capacity, and can effectively resist lateral loads. Failure started at the bottom track in the anchor bolt region, therefore strengthening of the corner detail is crucial.

Moghimi and Ronagh [5] investigated wall frame assemblies having diagonal bracing, bracket, gusset plates and base anchors under lateral cyclic loads. It found that the CFS wall frames provided better shear resistance when the diagonal strap bracing or the bracket at four corners was used.

Cheng Yu [6] conducted a research aimed to add shear strength values for 0.686 mm, 0.762 mm, and 0.838 mm steel sheet sheathed CFS shear walls with aspect ratios of 2:1 or 4:1. The test program consisted of the monotonic tests for determining the nominal shear strength for wind loads, as well as the cyclic tests using CUREE protocol to obtain the shear strength for seismic loads. The test results indicate that the reduction factor provided in the AISI standard is a simple reduction factor that represents fairly well the strength reduction based on the drift limit for walls that have an aspect ratio of 4:1.

Previous research [2] conducted a series of tests for CFS wall frames sheathed with gypsum board, CSB and OSB sheathing under monotonic shear loads. However, these test specimens used only single chord stud in the assembly of wall frame for wall test which is not conform to its specification requirements. Nine wall specimens utilizing back-to-back chord studs and CSB sheathing were tested in this study. The objective is to investigate the ultimate strength, stiffness, energy absorption, ductility ratio, and response modification factor of wall frame specimens subjected to monotonic shear and cyclic loads. The test results are also utilized to compare with the aforementioned test study.

2. Experimental Study
2.1 Materials

The cold-formed steel wall specimen assemblies contain three kinds of materials, i.e., cold-formed steel sections, calcium silicate board sheathing, and self-drilling screw. The cold-formed steel sections designated as SSC 400 conform to Chinese National Standard No. 6183 [7] for general light-gauge steel which requires the yield strength and tensile strength shall lie in between 20.4 to 51 kg/mm$^2$ (200 to 500 MPa) and 30.6 to 70.3 kg/mm$^2$ (300 to 690 MPa), respectively. This regulation requires the ratio of tensile strength to yield strength shall not be less than 1.13 and the elongation shall not be less than 10%. The average yield strength, $F_y$, and tensile strength, $F_u$, of CFS sections obtained from three tensile coupon tests are equal to 338 MPa and 412 MPa, respectively, which met the requirements. It is noted that the yield stress was measured by using 0.2% offset method. The average modulus of elasticity is 204,265 MPa. And the average elongations at ultimate tensile and rupture points are about 23% and 34%.

The sheathing material of calcium silicate board having two different thicknesses (9 mm and 12 mm) complies with CNS 13777 [8]. Typical No. 8 self-drilling screws as given in AISI [9] were used to connect the sheathing and steel wall frame. The minimum screw length and spacing of screws were provided to meet the AISI requirements.

2.2 Test Specimens

Cold-formed steel C-section studs, 92 mm x 65 mm x 12 mm x 1.6 mm, were used to assemble the wall frame test specimens as shown in Fig. 1(a). These sections were perforated with web openings, 39 mm x 39 mm @ 50 cm o.c., to provide space for utility conduits. The length of stud is equal to 240 cm. Two studs connected back to back were used at two vertical sides of the steel wall frame. These studs were placed perpendicularly into top and bottom channel tracks as shown in Fig. 1(b), and connected by using self-drilling screws.

The ratio of wall height to width (h/w) for test specimens is equal to 1.87 as can be seen in the wall frame layout in Fig. 2. The spacing of studs is equal to 40 cm o.c.. The calcium silicate boards were installed to the steel framing by using screws spaced 10 cm o.c. at peripheral of wall frame and 20 cm o.c. at middle studs as shown in Fig. 2. The test specimens were labeled to identify the loading condition, the sheathing thickness, the attached sheathing side, and the number of test conducted for
wall frames. In the design of specimen numbering, the M and C are defined as wall tested under monotonic shear load and cyclic loading, respectively, and the number 09 and 12 are classified as the test specimens using 9 mm and 12 mm thick calcium silicate boards, respectively. According to the placement of sheathing material, the O and T represent one-side sheathed frame and two-side sheathed frame.

2.3 Test Setup

Test specimens were placed vertically on the test platform to conduct the structural behavior of wall frames. The bottom track of specimens was bolted at 10 cm o.c. to the support I-beam. A 50-ton capacity MTS testing machine was used to apply the monotonic shear and cyclic loads to wall frame specimens. Fig. 3 is the configuration of test setup used in this study. The LVDTs were applied to obtain lateral and vertical displacements during test. Strain gages were also used to determine the strain variations on the sheathing. The applied monotonic shear and cyclic loads are in accordance with AISI [1] and ASTM E2126-09 [10], respectively. The monotonic shear load is applied in a constant speed of 1-cm/min to the test specimen until the test failure occurred. The applied cyclic load is based on two levels of displacement control as given in the method A of the ASTM specification. Fig. 4 shows the periodic displacement versus with the cyclic numbers as specified in Ref. 10. It is noted that the ratio of measured displacement to yielding displacement are equal to 25%, 50%, and 75% for the first three cycles at the first stage as shown in Fig. 4.

3. Evaluation of Experimental Data

A total of nine test specimens were conducted in this study to investigate the structural behavior of the sheathed CFS wall frames. Three test specimens were tested under monotonic shear load and six test specimens were tested under cyclic loading. These test specimens were arranged so that the effect of thickness of sheathing, number of side of sheathing attached, and the loading type on the ultimate strength can be obtained. The failure type, stress distribution, stiffness, ultimate load, energy absorption, ductility ratio, and response modification factor of wall frame specimens are discussed in the following sections.
3.1 Failure Types of Specimens

Five different types of failure of test specimens were observed in this study, i.e., tearing failure of bottom track, bearing failure of sheathing, fracture of sheathing, shear failure of self-drilling screw, and deformation of bottom track. The failure of wall specimens mostly occurred at the bottom track of steel frame due to the large deformation. Tearing failure of bottom track was only observed in the test specimens under cyclic loads. Other failure phenomena such as fracture of sheathing and shear failure of self-drilling screw were mainly occurred after/with the large deformation of bottom track. Detailed description of failure types is summarized as follows:

(1) Tearing failure of bottom track

When test specimen was applied by lateral loads, it resisted not only the shear force but also additional compression or tension forces induced by moment and overturning actions. The maximum forces are transmitted to the chord studs, which subjected to tension or compression depend on the direction of the applied load. Tearing failure at the end of bottom track was found which was caused by the uplift tension force as shown in Fig. 5.

(2) Bearing failure of sheathing

Because the test specimen was anchored at bottom track and the sheathing was connected to steel wall frame by self-drilling screws, the applied forces were transmitted to the sheathing through the screws. Consequently, the sheathing was subjected to concentrated stress and found bearing deformation or crack around some of connected screws.

(3) Fracture of sheathing

The attached sheathing of wall resists not only the shear force transferred from the wall frame but also the overturning moment produced from the applied lateral loads. It is noted that the sheathing material was locally fractured at the bottom of the wall frame as shown in Fig. 6.

(4) Shear failure of self-drilling screw

The screw shear ruptures occurred mainly on the end of bottom track at the loading side due to its concentrated force on the screws produced from the uplift loads of the chord studs. It was observed that, when the test load increasingly applied to the chord studs, the screw gradually failed by shear rupture as shown in Fig. 7.

(5) Deformation of bottom track

Two ends of bottom track were observed to be deformed after it subjected to tension and/or
compression forces induced from the wall panel. When the lateral loads were gradually applied, bottom channels were found to deform inward or outward for specimens under monotonic shear load or cyclic loads as can be seen in Figs. 6 and 8. Because the applied lateral loads have an instant large movement for specimens under cyclic load, the bottom channels were torn off at connected parts for some specimens.

3.2 Load and Displacement Relationship

Fig. 9 shows the load-displacement curves of wall specimens under monotonic load. The load-displacement hysteretic loops for each individual test specimen as well as the load-displacement envelopes of test specimens under cyclic loads can be seen in Fig. 10 and Fig. 11, respectively. The ultimate load, $P_{ult}$, and its corresponding maximum displacement, $\Delta_{ult}$, for test specimens conducted in this study are listed in Table 1. The ultimate loads obtained from specimens under monotonic shear load are found to be higher than those tests under cyclic loads. It is noted that the average ultimate load and maximum displacement for specimen series FC-C09-HO, having one-side attachment with 9-mm thick sheathing, is equal to 14.68 kN and 67.06 mm, respectively. The average ultimate load and maximum displacement for specimen series FC-C12-HO, having one-side attachment with 12 mm thick sheathing, is equal to 15.24 kN and 58.41 mm, respectively. The average ultimate load and maximum displacement for specimen series FC-C09-HT, having two-side attachment with 9 mm thick sheathing, is equal to 20.66 kN and 69.54 mm, respectively.

The ultimate loads for test specimens having one-side sheathing under monotonic shear and cyclic loads are found slightly different for specimens having either 9 mm or 12 mm sheathing. However, the ultimate load for specimens having two-side of 9 mm sheathing is about 40.7 % higher than those specimens with one-side of 9 mm sheathing when subjected to cyclic loads. As a result, the load capacities for wall frame specimens having two-side calcium silicate board attachment are higher than those having one-side sheathing attachment.

It is noted that, when subjected to monotonic shear load, the ultimate strengths for specimens conducted in this study are 29.56%, 20.49%, and 22.49% higher than those obtained from previous research results [2] for specimens having one-side and 9 mm sheathing, one-side and 12 mm sheathing, and two-side and 12 mm sheathing, respectively. In addition, the average maximum displacement as given in Table 1 is 68.21% less than that obtained from previous research results [2].
The steel wall frame having back-to-back chord studs can provide better resisting strength and stiffness than that having only one end chord stud.

The test strengths of specimens having two-side 9 mm sheathing were 40% and 41% higher than those having one-side 9 mm sheathing when subjected to monotonic shear and cyclic loads, respectively. Previous research [2] indicated that the test strength for specimens having two-side attachment (9 mm calcium silicate board) is 47% higher than those specimens having only one-side attachment. Based on the above-mentioned comparisons, it is noted that from the strength point of view, the influence on the difference between the wall using one-side sheathing and two-side sheathing is higher than the influence on the difference between the wall using one end chord stud and back-to-back chord studs.

The test specimens were found to be subjected to not only shear force at the early stage of loading but also the overturning action when large displacements occurred as shown in Fig. 2. Fig. 12 illustrated that the horizontal and vertical displacements for specimen FM-C09-HO-1 measured at four stages, i.e., early stage (1/4 $P_{ult}$), middle stage (1/2 $P_{ult}$), late stage (3/4 $P_{ult}$), and final stage ($P_{ult}$). It can be observed from Fig. 12 that the wall specimen was mainly subjected to shear force and moment at the early and middle stages, and it produced larger displacements at late and final stages due to the deformation and failure of bottom track.

### 3.3 Stress Distribution on Test Specimen

In this study, strain gages were attached on the sheathing of the test specimen to evaluate the stress variations through the test. Fig. 13 shows the strain gages locations attached on the top, middle, and bottom of sheathing for specimen FM-C09-HO-1. A total of 18 strain gages were used for test specimen, where 9 strain gages were attached separately in horizontal and vertical locations as shown in the figure. The stress distributions determined from the strain gage data on the sheathing are shown in Figs. 14 and 15 in the vertical direction (Y-direction) and horizontal direction (X-direction), respectively. In Fig. 14, it is noted that the top portion of wall panel had a relatively small stress in the Y-direction at all four loading stages, which is identical to the test results that no damages were observed on the test specimen. At the early stage of loading, small tension stresses were observed at bottom of wall specimen as shown on strain gages labeled B1 and B3 in the figure, and the compression stress was shown on strain gage labeled B5 as expected. The tension stresses were then gradually increased for the next stages of loading as can be seen from the variation of stress distribution at the bottom of
specimen. At the final stage of loading, the bottom of wall specimen was totally subjected to tension force as indicated on the strain gages because of large deformation of bottom track. It is also noted that the decrease of tension stress at strain gage B1 is due to the sheathing fracture and bottom track failure.

Fig. 15 illustrated the stress variation of the wall specimen in X-direction. It is also noted that the top portion of wall panel had a relatively small stress in the X-direction at all four loading stages. The bottom of specimen was subjected to compressive stresses which were obtained from strain gage readings on B2, B4 and B6 as shown at four different loading stages. The compressive stress was gradually increased to its maximum value until the specimen failed. However, at the final stage of loading, the stress at B2 was observed to become tension because the sheathing was peeled off from the steel frame.

### 3.4 Stiffness

The stiffness of tested wall specimens can be determined by using the secant stiffness recommended by European Convention for Construction Steelwork (ECCS) [11] and AISI Manual [12]. ECCS method utilizes the slope connected between the origin and the point at 0.4$P_u$ obtained from test results. The AISI method is similar to ECCS method except that the aspect ratio of the wall frame is considered for stiffness calculation. Thus, the stiffness calculated from the AISI method is 1.87 times higher than that determined from the ECCS recommendations. Table 2 gives the calculated stiffness of test specimens based on the ECCS and AISI methods. It is noted that the stiffness for specimens subjected to monotonic shear load are higher than those subjected to cyclic loads. The calculated stiffness is about the same for test specimen having one-side of 9 mm and 12 mm sheathings when subjected to monotonic shear load. The test specimens having two-side sheathing attachment present higher stiffness than those having one-side sheathing, which matches with the previous findings stated in Ref. 2.

### 3.5 Energy Absorption

The energy absorption is the area under the curve in the load displacement graph. Table 3 lists the energy absorption of each test specimen at locations of different displacements, 20 mm, 40 mm, 60
mm, and 80 mm. The changes of energy absorption with two different thicknesses and types of attachment as can be seen in Table 3. It is noted that for the specimen under cyclic loads, the value listed in Table 3 is the average values of energy absorption obtained from both positive and opposite directions of envelope curve. Based on the values list in each column of appointed displacement, the following observation can be obtained: (1) The energy absorption of the specimen under monotonic load is greater than the specimens subjected to cyclic loads for the wall frames with the same arrangement of sheathing, and (2) For specimens with 9-mm thickness of sheathing and tested under either monotonic or cyclic loads, the specimen with two-side sheathing has higher values of energy absorption than those with one-side sheathing. As comparing the wall specimens tested under monotonic load with the wall specimens sheathed with CSB of previous study [2], it can be found that the wall frame using back-to-back chord studs provide better strength, stiffness, and energy absorption than the wall frame using single chord stud.

### 3.6 Ductility Ratio

The definition of ductility ratio ($\mu$) is the ratio of ultimate displacement ($D_u$) to yield displacement ($D_y$), $D_u/D_y$. In this study, the ductility ratio of wall frame is determined on the basis of ECCS recommendation [11] and AISI standard [1]. The former uses the equivalent elastic-plastic model to determine the values of $D_u$ and $D_y$. The AISI method adopts the equivalent energy elastic-plastic analysis model which is based on the notion that the energy dissipated by the wall specimen during a monotonic or reserved cyclic test is equivalent to the energy represented by a bilinear curve. The value of $D_u$ is defined as the displacement at 80% post ultimate load. The determination of ductility ratio for both methods is introduced and discussed in Ref. 2.

Table 4 lists the ductility ratios determined from the ECCS recommendation and AISI standard for test specimens. For the wall specimens tested under monotonic shear loads, the values of ductility ratio calculated by using ECCS and AISI methods vary from 2.21 to 2.53 and 2.44 to 2.74, respectively. For the wall specimens tested under cyclic loading, the values of ductility ratio obtained by using ECCS and AISI methods vary from 1.80 to 2.08 and 2.37 to 2.65, respectively. It was observed that the ductility ratios computed by ECCS recommendation are less than those determined by AISI standard for all test specimens. As noted in Table 4, the ductility ratios are about the same for specimens having one-side attachment with 9 mm and 12 mm sheathings. And, the ductility ratios are also about the same for
specimens having 9 mm sheathing with two-side and one-side attachment when subjected to cyclic loads. For the wall specimens subjected to monotonic shear load, the ductility ratios obtained from this study are lower than those obtained from previous study [2] which only single chord stud was adopted in the assembly of wall frame.

3.7 Response Modification Factor

Based on the arguments that a well-designed structural resistant system has a ductile behavior and can carry large inelastic deformation without collapse as well as the benefit of relying on ductility capacity to reduce the required lateral strength for seismic design, the response modification factor, R, has been adopted and became as an important parameter in the seismic design of buildings. The value of R is expressed in terms of ductility reduction factor ($R_\mu$) and system over-strength factor ($\Omega_o$) [13] and defined as:

$$R = \frac{P_e}{P_s} = \frac{P_s}{P_y} = R_\mu \Omega_o$$  \hspace{1cm} (1)

Where $P_e$, $P_s$, and $P_y$ represent structure's elastic response strength, first significant yield strength, and idealized yield strength, respectively. Based on Newmark and Hall method [14], the ductility reduction factor ($R_\mu$) can be determined in terms structure's ductility ratio ($\mu$) under different period (T) conditions as follows:

$$R_\mu = \mu \hspace{1cm} T>0.5s$$  \hspace{1cm} (2)

$$R_\mu = \sqrt{2\mu - 1} \hspace{1cm} 0.1<s<T<0.5s$$  \hspace{1cm} (3)

$$R_\mu = 1 \hspace{1cm} T<0.03s$$  \hspace{1cm} (4)

According to the ASTM standard [10], the yield strength ($P_y$) can be obtained by using equivalent energy elastic-plastic (EEEP) curve. An ideal elastic-plastic curve circumscribes an area equal to the area enclosed by the observed load-displacement curve or envelop curve between the origin and the ultimate displacement. The yield strength can be determined by the idealized following equation:

$$P_y = \left( D_u - \sqrt{D_u^2 - \frac{2A}{K_s}} \right) K_s$$  \hspace{1cm} (5)

Where

A: the area under the observed load-displacement curve or envelope curve from zero to ultimate
displacement ($D_u$) which is defined as the displacement at 80% post ultimate load.

$K_e$: elastic shear stiffness defined by the slope of the secant passing through the origin and a point on the observed load-displacement curve or envelope curve where the load equal to 0.4 $P_u$.

Table 5 lists the response modification factor for all tested wall specimens. The over-strength factor of each specimen is also presented in Table 5. The response modification factors are about the same for specimens having 9 mm sheathing with two-side and one-side attachment when subjected to cyclic loads. It can be observed from Table 5 that the average value of over-strength factor from this test is 2.01 which is slightly lower than the AISI [1] suggested value of 2.5 for the light-framed walls with shear panel of all other materials. As can be seen in Table 5, the values of response modification factors are around 4.15 to 4.72. According to AISI standard [1], the $R$ values are recommended to be as 6.5 and 2.0 for the lighted-framed walls sheathed with wood structural panels and shear panels of all other materials, respectively. The average value of response modification factor obtained from this test program is 4.46. It is suggested that a value of 4.2 which is the lowest tested value may be used as response modification factor for the steel framing wall sheathed by calcium silicate board for conservatively consideration.

4 Conclusions and Suggestions

A total of nine cold-formed steel wall specimens were conducted in this study, which included three specimens subjected to monotonic shear load and six specimens subjected to cyclic loads. Test specimens were categorized into three groups, i.e., wall frames attached with one-side of 9 mm sheathing, one-side of 12 mm sheathing, and two-side of 9 mm sheathing of calcium silicate board. Based on the test results, the following conclusions were made:

(1) There were five types of failure observed in wall specimens. The tearing failure and/or large deformation of bottom track are the major reasons to induce wall failure. Other failure phenomena were mainly occurred after/with the large deformation of bottom track.

(2) Wall specimens with two-side sheathing provide higher ultimate strength, stiffness, and energy absorption as compared with those having one-side sheathing. But the ductility ratios and response modification factors are about the same for the specimens sheathed with one-side and two-side calcium silicate boards under cyclic loads.

(3) The ultimate strength, stiffness, energy absorption, and ductility ratio of tested specimens under
cyclic loads are less than those subjected to monotonic shear load for most tests.

(4) The wall specimens assembled by using back-to-back chord studs have higher values of ultimate strength, stiffness, and energy absorption as comparing with previous study for which only single chord stud was adopted in the assembly of wall frame of test specimens. However, the ductility ratios obtained from this study are lower than those obtained from previous study for the specimens under monotonic load.

(5) For the steel framing wall sheathed by calcium silicate board, the response modification factor (R) may be assumed to be as a value of 4.2 for design purpose. Because of the effect of moment force and overturning motion of wall, the lateral displacement measured opposite to the loading side was observed to be larger than that at the loading side for the specimen subjected to either monotonic or cyclic loads, and most specimens present failures at the bottom track around loading side and opposite side. In order to reduce the deformation on the wall panel, it shall be beneficial to have anchored mechanism at both ends of wall. Therefore, wall test having hold-down anchors at chord studs is suggested in the future experimental study, and the test results can be used to compare with the findings herein.

Acknowledgments

The authors would like to thank National Science Council for funding this project. Special thanks to Gu, Jheng-Yan, Chan, Po-Yu, and Tsao, Meng-Siou for their contributions throughout the project.

References


[6] Yu C. Shear resistance of cold-formed steel framed shear walls with 0.686 mm, 0.762 mm, and 0.838 mm steel sheet sheathing, Engineering Structures 2010;32:1522-29.


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Table 1. Ultimate tested loads and the corresponding displacements

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>$P_{ult}$ (kN)</th>
<th>$\Delta_{ult}$ (mm)</th>
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<tr>
<td>FM-C09-HO-1</td>
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Table 2. Stiffness calculated for individual test specimen

<table>
<thead>
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<th>Specimen No.</th>
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<th>$K_{AISI}$ (kN/mm)</th>
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<tbody>
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<td>FM-C09-HO-1</td>
<td>0.51</td>
<td>0.95</td>
</tr>
<tr>
<td>FC-C09-HO-1</td>
<td>0.33</td>
<td>0.61</td>
</tr>
<tr>
<td>FC-C09-HO-2</td>
<td>0.37</td>
<td>0.68</td>
</tr>
<tr>
<td>FM-C12-HO-1</td>
<td>0.53</td>
<td>0.99</td>
</tr>
<tr>
<td>FC-C12-HO-1</td>
<td>0.42</td>
<td>0.78</td>
</tr>
<tr>
<td>FC-C12-HO-2</td>
<td>0.41</td>
<td>0.77</td>
</tr>
<tr>
<td>FM-C09-HT-1</td>
<td>0.66</td>
<td>1.24</td>
</tr>
<tr>
<td>FC-C09-HT-1</td>
<td>0.43</td>
<td>0.81</td>
</tr>
<tr>
<td>FC-C09-HT-2</td>
<td>0.46</td>
<td>0.87</td>
</tr>
</tbody>
</table>
Table 3. Energy absorption of individual test specimen

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>20 mm (kN-mm)</th>
<th>40 mm (kN-mm)</th>
<th>60 mm (kN-mm)</th>
<th>80 mm (kN-mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM-C09-HO-1</td>
<td>109.5</td>
<td>363.5</td>
<td>702.0</td>
<td>1018.7</td>
</tr>
<tr>
<td>FC-C09-HO-1</td>
<td>66.0</td>
<td>246.7</td>
<td>501.2</td>
<td>774.6</td>
</tr>
<tr>
<td>FC-C09-HO-2</td>
<td>71.5</td>
<td>268.1</td>
<td>546.9</td>
<td>847.4</td>
</tr>
<tr>
<td>FM-C12-HO-1</td>
<td>94.4</td>
<td>347.4</td>
<td>663.1</td>
<td>967.0</td>
</tr>
<tr>
<td>FC-C12-HO-1</td>
<td>83.5</td>
<td>305.7</td>
<td>596.3</td>
<td>879.0</td>
</tr>
<tr>
<td>FC-C12-HO-2</td>
<td>81.5</td>
<td>302.1</td>
<td>598.4</td>
<td>887.7</td>
</tr>
<tr>
<td>FM-C09-HT-1</td>
<td>129.3</td>
<td>416.4</td>
<td>807.4</td>
<td>1270.5</td>
</tr>
<tr>
<td>FC-C09-HT-1</td>
<td>86.9</td>
<td>328.0</td>
<td>678.6</td>
<td>1074.4</td>
</tr>
<tr>
<td>FC-C09-HT-2</td>
<td>93.5</td>
<td>352.4</td>
<td>727.3</td>
<td>1139.6</td>
</tr>
</tbody>
</table>

Table 4. Ductility ratio of individual test specimen

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>( \mu_{ECCS} )</th>
<th>( \mu_{AISI} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM-C09-HO-1</td>
<td>2.21</td>
<td>2.44</td>
</tr>
<tr>
<td>FC-C09-HO-1</td>
<td>2.07</td>
<td>2.52</td>
</tr>
<tr>
<td>FC-C09-HO-2</td>
<td>2.08</td>
<td>2.43</td>
</tr>
<tr>
<td>FM-C12-HO-1</td>
<td>2.27</td>
<td>2.49</td>
</tr>
<tr>
<td>FC-C12-HO-1</td>
<td>2.06</td>
<td>2.65</td>
</tr>
<tr>
<td>FC-C12-HO-2</td>
<td>2.02</td>
<td>2.41</td>
</tr>
<tr>
<td>FM-C09-HT-1</td>
<td>2.53</td>
<td>2.74</td>
</tr>
<tr>
<td>FC-C09-HT-1</td>
<td>2.00</td>
<td>2.56</td>
</tr>
<tr>
<td>FC-C09-HT-2</td>
<td>1.80</td>
<td>2.37</td>
</tr>
<tr>
<td>Average</td>
<td>2.12</td>
<td>2.51</td>
</tr>
</tbody>
</table>
Table 5. Response modification factor of individual test specimen

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>$R_\mu$</th>
<th>$\Omega_0$</th>
<th>$R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM-C09-HO-1</td>
<td>1.97</td>
<td>2.11</td>
<td>4.15</td>
</tr>
<tr>
<td>FC-C09-HO-1</td>
<td>2.01</td>
<td>2.27</td>
<td>4.58</td>
</tr>
<tr>
<td>FC-C09-HO-2</td>
<td>1.97</td>
<td>2.28</td>
<td>4.49</td>
</tr>
<tr>
<td>FM-C12-HO-1</td>
<td>1.99</td>
<td>2.14</td>
<td>4.27</td>
</tr>
<tr>
<td>FC-C12-HO-1</td>
<td>2.08</td>
<td>2.28</td>
<td>4.72</td>
</tr>
<tr>
<td>FC-C12-HO-2</td>
<td>1.95</td>
<td>2.32</td>
<td>4.52</td>
</tr>
<tr>
<td>FM-C09-HT-1</td>
<td>2.12</td>
<td>2.11</td>
<td>4.46</td>
</tr>
<tr>
<td>FC-C09-HT-1</td>
<td>2.03</td>
<td>2.28</td>
<td>4.62</td>
</tr>
<tr>
<td>FC-C09-HT-2</td>
<td>1.94</td>
<td>2.24</td>
<td>4.34</td>
</tr>
<tr>
<td>Average</td>
<td>2.01</td>
<td>2.23</td>
<td>4.46</td>
</tr>
</tbody>
</table>