Research on Cold Formed Steel Stud Walls

A. Mirza Goltabar Roshan & F. Zahedi Tajrishi
University of Technology, Babol, Mazandaran, Iran

H.R. Vosoughifar
Islamic Azad University, Tehran, Iran

SUMMARY:
Cold-formed steel (CFS) has been an attractive alternative to traditional materials and light steel framed (LSF) building has been a suitable replacement for traditional building systems. Light steel framed (LSF) systems with cold formed steel members have a widespread use for low and medium rise residential, industrial and commercial building construction in the US, Australia, Canada, Japan and some European countries. In recent years important research activities have been undertaken in order to evaluate the performance of light-gauge steel stud walls. This paper reviews the researches evaluated the lateral behaviour of sheathed Cold-Formed Steel shear walls and Cold-Formed Steel strap-braced walls. Many of the researches concern about the lateral performance of the light wall systems and have focused on the failure modes of different systems and on the main factors contributing to the ductile response of the CFS walls.

Keywords: cold formed steel, strap-braced walls, shear wall, stud

1. INTRODUCTION

The steady-growth in the housing requirements has generated the searches for new constructive materials and construction methods to improve life quality, cost-efficiency and ease construction. Cold-formed steel (CFS) has been an attractive alternative to traditional materials such as concrete or timber while light steel framed (LSF) building has been a suitable replacement for traditional building systems. LSF systems with cold formed steel members have a widespread use for low and medium rise residential, industrial and commercial building construction in the US, Australia, Canada, Japan and some European countries. The advantages of cold-formed steel, such as being dimensionally stable, non-combustible, termite and borer proof, durable, lightweight and 100% recyclable, are the main factors for this increase in use.

Cold formed steel (CFS) walls can be built up from C-shaped light gauge steel studs fastened to C-shaped light gauge steel tracks at the top and bottom having different solutions for interior and exterior cladding. In order to maintain the integrity of these structures when subjected to horizontal forces due to wind or an earthquake, the use of diagonal flat steel strap cross bracing or structural sheathing are practical solutions. The straps act as a vertical concentric bracing system which transfers the lateral forces from the roof and floor levels to the foundation. The overall lateral strength, ductility and stiffness of this bracing system may not be related solely to the steel straps; many other elements in the lateral load carrying path can play a role, such as the strap connections, the gusset plates (if needed), the anchorage including hold down and anchor rod, etc. CFS walls with structural sheathing named shear walls are the main structural elements which act against horizontal loads. However, the behavior of shear walls subjected to earthquake is not yet fully understood, in recent years an important effort has been made to clarify certain aspects related to their shear strength, stiffness and ductility. According to experimental evidences, the performance of the wall panels, as a whole, is governed by the performance of the connectors e.g.: sheeting-to-sheeting connectors, and sheeting-to-framing connectors.

This paper reviews the researches which have evaluated the lateral behaviour of sheathed Cold-Formed Steel shear walls and Cold-Formed Steel strap-braced walls. Almost all studies approach the problem of seismic response of these walls by characterizing, experimentally and numerically, the performance of wall panels. Many of them concern about the lateral performance of the light wall systems and have focused on the failure modes of different systems and on the main factors contributing to the ductile response of the CFS walls.

2. RESEARCHES ON THE CFS STRAP-BRACED WALLS

Rogers et al. [1] have a research to evaluate the inelastic lateral load carrying performance of typical weld and screw-connected single-storey strap braced wall configurations. These walls were designed based on the capacity approach, required for the design of limited ductility walls, as described in AISI-S213 [2]. The results were used to verify the strap braced wall seismic design provisions in the 2007 version of AISI-S213. The scope of the research also included the determination of ‘‘test-based’’ seismic force modification factors based on the measured ductility and over strength of the test walls for comparison with the R-values recommended in AISI-S213. The strap braces were expected to experience gross cross-section yielding along their length, while the other elements in the seismic force resisting system (SFRS) were selected to be able to carry the probable brace capacity. A total of 44 tension- only X- braced walls ranging in size from 610-2440 mm^2 to 2440-2440 mm^2 (aspect ratios from 4:1 to 1:1), designed in three brace sizes and three wall sizes consisting of light, medium and heavy CFS construction. All wall specimens were tested using displacement controlled monotonic and reversed cyclic protocols.

Test results consisted of material properties for the straps, chords, tracks and gusset plates, measured wall performance and failure mode and seismic force modification factors. The results of this testing illustrated the ability of walls with an aspect ratio less than or equal to 2:1 to reach and maintain their yield resistance in the inelastic range of deformation if capacity design principles were implemented and material requirements were met, as described in AISI-S213. It was, however, recommended that the use of high aspect ratio strap braced walls, i.e. having aspect ratios greater than 2:1, be avoided unless the moments associated with flexure of the chord studs are included in the capacity design procedure for these studs. Inelastic deformations resulted from brace yielding in the 1:1(2440 mm) and 2:1(1220 mm) aspect ratio walls, where as the 4:1 (610 mm) walls experienced combined axial compression and flexure of the chord studs and only minimal brace yielding. The seismic force modification factors R_d=2.0 and R_o=1.3 listed in AISI-S213 were appropriate for the 1220 and 2440 mm long walls, however not for the 610 mm long walls, which did not show adequate ductility or over strength.

Moghimi and Ronagh [3] studied on 20 full-scale specimens of CFS strap-braced walls to evaluate the performance of five different bracing arrangements by means of cyclic loading. In the first strap-bracing scheme, similar to the conventional bracing normal in Australian trade practice, straps were screwed to top and bottom tracks and to left and right studs. These walls were tested with and without vertical load. In the second scheme four brackets were placed at the four corners of the wall. The strength, stiffness and ductility of this system would depend mostly on the brackets' shape and size and to a lesser extent on the chords. The effect of gypsum board in conjunction with bracket members was studied. The third scheme investigated direct screw connection of straps to the four outer corners of the wall panel. The effects of chords, vertical load and double side-strap bracing on the lateral performance of this wall system were investigated. A similar study was conducted for the connection of straps to the interior frame joints. Finally, the lateral performance of a wall panel strap-braced with gusset plates at four corners was investigated. All the specimens were designed based on FEMA 450 [4] regulations apart from Type I. The following effects were studied in this research: the effect of vertical load on the lateral response, the effect of non-structural gypsum board on lateral performance of a strap-braced wall system with and without vertical load, the effect of double-sided bracing, and the effect of doubling the chords. The following results were made from the study.

1-In general, it would be more conservative not to include gypsum boards in lateral resistance calculations. 2-The conventional bracing type in practice, rendered unacceptable results because of the premature distortional buckling of the left and right studs. 3-If the chord members were selected so that each chord could carry the strap's full resistance capacity (due to yielding and strain hardening of the strap material), the existence of a concurrent vertical load up to 80% of actual vertical load capacity of the remaining studs would have no adverse effect on the lateral load resistance of the wall panel. 4-Adding brackets at four corners of the wall panel would improve the lateral performance (strength, stiffness and ductility) of the wall panel considerably. 5-Double-side bracing did not offer a great deal of advantage over single-side bracing when a wall panel was designed to allow straps to develop their full plastic capacity. 6-Although gusset plates provided enough room for connecting straps to the panel (eliminating the possibility of strap-to-panel connection failure), and presented a good performance with sufficient ductility and stiffness, they would be manually too labor intensive, their added thickness may cause aesthetic problems due to unevenness of the covering plasterboard, and weakness at the position of the tension unit hole would remain a problem. 7- Using double section chords would offer a lot of advantages, such as strengthening of the track-to-stud joints under bearing failure and providing more room for the insertion of screws that connect the strap to the wall panel. 8-Strap-braced walls without gypsum board or bracket members presented severe pinching in their hysteretic loops due to plastic slack of strap braces and lack of redundancies. This may trigger brittle connection failure or damage to non-structural elements. The use of brackets would be therefore recommended at the corner of the wall panels.

In another research, Moghimi and Ronagh [5] introduced new strap- bracing systems that comply with codes provisions and satisfy ductility criteria. The program consisted of nine full-scale specimens to evaluate the performance of four different strap-braced walls. The first strapping scheme had four brackets at the four corners of the wall. Strength, stiffness and ductility of this system depended mostly on the bracket's shape and size and to a lower extent on the chords. The second scheme investigated direct screw connection of straps to the four outermost corners of the wall panel. A similar study was conducted for the connection of straps to the interior frame joints. Finally, the lateral performance of a strap-braced wall panel with solid strap connected to gusset plates at four corners was investigated.

Based on the results, following conclusions could be drawn: (a) New systems for strap-braced system: 1- Adding brackets at four corners of a wall panel would improve the lateral performance of the panel considerably, even in the case of using only a single stud as a chord member. 2- By choosing appropriate perforated straps, the tearing of the strap at the tension unit location or at the strap-to-frame connection would not occur. In addition, yielding of the strap would occur along side the distributed holes. For a strap with close tensile and yield strength, perforating may be the only option to eliminate the brittle tension failure. (b) Appropriate details to improve seismic performance of strap-braced system: 1- When straps are connected to the exterior chord–track joints, the overall buckling load capacities of these members would be low, especially when hold-downs were not provided at the top track. In order not to allow the undesirable buckling failure modes to govern, double back-to-back studs could be used as chord members. This restored the strap yielding failure mode as long as the chords were properly connected to tracks. 2. The performance of the X-strap system could be improved by placing four C-section cut-offs in the track at the four corners of the frame where the straps were connected. 3- When straps were connected to interior joints, a very good performance was observed, even with single chords and a top track without hold-downs. 4- The initial slackness in the strap should be as small as possible; otherwise, premature failure in the strap at the tension device location would occur. 5- When hold-downs were located inside the frame, better performance and strength would be achieved, in contrast to hold-downs that were connected to the outer face of the chord members, due to higher punching shear capacity. (c) Lateral performance of strap-braced system: although the system represented highly pinched hysteretic behavior, it would be still very much ductile and could reach a shear resistance equal to that of the first cycle in every subsequent stabilized cycle.

3. RESEARCHES ON THE SHEATHED CFS SHEAR WALLS

Pan and Shan [6] are focused on the experimental study of the structural strength of cold-formed steel wall frames with sheathing under monotonic shear loading. Two aspect ratios, 1.0 and 2.0 were considered 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 and 12 mm) were adopted in the test specimens. Totally, 13 wall specimens including 5 walls with gypsum board sheathed, 5 walls with calcium silicate board sheathed, 2 walls with oriented-strand board sheathed, and 1 wall frame without sheathing were tested. The ultimate strength, stiffness, energy absorption, and ductility ratio were studied for each test specimen.

Based on the test results, the following findings were concluded: (1) there were five types of failure for wall specimens with sheathing 1-Bearing failure of sheathing material, 2-Shear failure of self-drilling screw, 3-Sheathing relative movement, 4-Fracture of sheathing material and 5-Deformation of stud and track. The bearing failure of sheathing around the self-drilling screw area and the separation between sheathing and screw were the main reasons to induce failure. (2) Based on the test result analysis, the full-size specimens with two-side sheathings had the highest ultimate strength, and wall frame without sheathing had the lowest value of an ultimate strength, as expected. And the ultimate strength of wall frame with sheathing increased with increasing the thickness of board. (3) As comparing the ultimate strength for the same type of specimens, the wall frame with OSB sheathing had the greatest value, the wall frame with calcium silicate board had the secondary value, and the wall frame with gypsum boards had the smallest value. (4) The ultimate strengths of specimens with one-side sheathing were 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 an aspect ratio of 2.0 were about 35% less than those of the specimens having an aspect ratio of 1.0 for the wall frames with the same sheathing configuration. (5) The energy absorption of full-size or half-size specimen with two-side sheathing was greater than the full-size or half-size specimen with one-side sheathing. For the same type of specimens, the specimen with calcium silicate board had the highest value of energy absorption, the specimen with oriented-strand board had the secondary value, and the specimen with gypsum had the lowest value. (6) For the specimens with the same sheathing arrangement, the ductility ratio of the half-size specimen was greater than that of the full-size specimen. And, for the specimens with the same aspect ratio and sheathing material, the ductility ratio of the specimen with one-side sheathing was greater than that of the specimen with two-side sheathing. (7) For the cold-formed steel framing wall employed to resist shear force, it was suggested that the design ductility ratio of wall sheathed with gypsum board, calcium silicate board, or an OSB may be assumed to be 6.6, 3.8, or 3.9, respectively. However, changes of screw arrangement and anchor condition would affect the ductility ratio. From the test results, it showed that the mechanic properties of sheathing material would influence not only the specimen's loading capacity, but also the structural behavior.

Yu and Chen [7] studied on the cold-formed steel stud framed shear wall using steel sheet sheathing (referred as CFS sheet steel shear). Although the 1.83 m wide 2.44 m high CFS shear walls were practically used in the field, it had not been completely studied and the codified nominal strengths of CFS sheet steel shear in AISI-S213 were only based on experimental results on 0.61 and 1.22 m wide and 2.44 m high walls. Therefore the objective of the research was to experimentally investigate the behavior and shear strength of 1.83 m wide 2.44 m high CFS shear walls and to identify the appropriate framing and sheathing details to ensure satisfactory seismic performance. The test program included both monotonic and cyclic tests on a total of 19 CFS shear walls using 4 different framing and sheathing configurations. The various test parameters were framing member web depth and thickness, sheathing sheet thickness, joint stud details, and shear wall bracing details. The test results indicated that besides the sheet buckling and screw pull out, the interior studs may buckle when the 1.83 m wide shear wall was subjected to cyclic lateral forces if the minimum framing required by AISI-S213 was used without additional detailing. To prevent the failure in the studs, special detailing was developed in the research. It was discovered that the special detailing could increase both the shear strength and the ductility of the shear walls. The research also found that the codified nominal shear strengths could be conservatively used for walls with an aspect ratio of 3:2. Based on the test results, the nominal seismic shear strength for 1.83 m wide CFS shear walls was established for design purposes.

Yu [8] also presented a research project aimed to evaluate 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 project consisted of two series of tests in a displacement control mode. The first series was monotonic tests for determining the nominal shear strength for wind loads. The second series was the cyclic tests using CUREE protocol to obtain the shear strength for seismic loads. The studied shear walls used 0.838 mm or 1.092 mm thick CFS framing members. The sheathing was only attached to one side of the frame. No. 8 modified truss head self-drilling screws were used for all the test specimens. Two wall aspect ratios were investigated in this test program: 2.44 m wide-1.22 m high (aspect ratio 2:1) and 2.44 m wide-0.61 m (aspect ratio 4:1). The test parameters also included three steel sheathing thicknesses: 0.686 mm, 0.762 mm, and 0.838 mm and three fastener spacing configurations on the panel edges: 152 mm, 102 mm, and 51 mm. The fastener spacing in the field of the sheathing was 305mm for all shear walls. A total of 30 monotonic tests were conducted in this test program. The nominal shear strengths were calculated as the average of the peak loads of two identical tests. A total of 30 cyclic tests were conducted and the nominal shear strength was determined as the average peak load of all the identical tests. The nominal shear strength for wind loads was based on monotonic test results and the nominal shear strength for seismic loads was obtained from the cyclic tests. Therefore the nominal shear strengths for wind loads and seismic loads were established from the test results. Test results indicated that a linear relationship could be assumed between the nominal shear strength and the fastener spacing at panel edges. In this test program, fastener spacing of 152 mm, 102 mm, and 51 mm were investigated, therefore the nominal strength for walls with other fastener spacing could be estimated accordingly. The buckling of the steel sheathing and pullout of sheathing screws were the primary failure modes for sheet steel CFS shear walls. Flange distortion of the boundary studs subjected to tension was also observed on the walls with 251 mm/305 mm screw spacing. This project also showed that CFS framed shear walls with large aspect ratios had relatively low stiffness but yielded significantly large drift capacity. The AISI Lateral Design Standard permits some CFS shear walls resisting wind or seismic loads to exceed the 2:1 aspect ratio limit, but requires that the nominal shear strength be reduced by a factor of 2w/h for those assemblies with a height to width aspect ratio greater than 2:1. It also requires that the allowable strength (ASD) be determined by dividing the nominal shear strength by a safety factor of 2.5 for shear walls resisting seismic loads and 2.0 for shear walls resisting wind loads. The test results indicated that the code reduction factor is a simple reduction factor that represented fairly well the strength reduction based on the drift limit for walls that have an aspect ratio of 4:1.

Fiorino et al. [9] evaluated the behaviour of sheathed Cold-Formed Steel Structures (SCFS) in the case of "frequent", "rare" and "catastrophic" seismic events. Basically, if CFS structures were designed according to the "sheathing-design" methodology, then the seismic behaviour of shear walls would be strongly influenced by the sheathing-to-frame connections response, characterized by a remarkable nonlinear response and a strong pinching of hysteresis loops. Therefore, in the research, nonlinear dynamic analysis using an ad-hoc model of the hysteresis response of SCFS shear walls was carried out. Several wall configurations were considered investigating various parameters such as sheathing panel typology, wall geometry, external screw spacing, seismic weight and soil type. Based on results of incremental dynamic analysis (IDA), three behaviour factors were defined, which would take into account overstrength, ductility and both overstrength and ductility, respectively. A "multi-performance" approach was proposed to evaluate the behavior factors considering the response of SCFS walls also in the case of catastrophic earthquakes. According to the obtained results, a behaviour factor equal to 1 should be considered for *IO* level, while q_1 =2 and q_3 =3 could be used for *LS* and *CP* levels, respectively. Moreover, a case study on the "multi-performance" design methodology was presented, in which the obtained results confirm an adequate design which would allow SCFS constructions to show an acceptable behaviour also in case of catastrophic seismic events.

Dubina [10] summarized the research activities carried out in the last few years at the Politehnica University of Timisoara with the aim to evaluate the performance and to characterize the specific features of CFS structures for design purpose. The review consisted of experimental program made by monotonic and cyclic tests on full-scale shear panels, tests on connection details, and in situ ambient vibration tests on a house under construction.

The experimental program was based on six series of full-scale wall tests with different cladding arrangements. Each series consisted of 3600 mm × 2440 mm identical wall panels tested statically, both monotonic and cyclic. The main frame of the wall panels were made of cold-formed steel elements. In specimens using corrugated sheet as cladding the sheets were placed in a horizontal position (Series I and II). Additionally on the ‘interior’ side of specimens in Series II, gypsum panels were placed vertically. Bracing was used in Series III specimens, by means of straps on both sides of the frame. Series IV consisted of three specimens prepared with door opening. OSB panels were placed in similar way as the gypsum panels in earlier specimens (series OSB I and OSB II), only on the ‘external’ side of the panel and fixed to the frame.

Based on the results, difference between series I and series II could be attributed to the effect of the gypsum board. There was an increase in the ultimate load and slightly improvement in ductility. By comparing series I and series IV, it could be understood that there was a significant decrease of initial rigidity, but ductility values were essentially unaffected. Because of the different sheeting system in series I and series III, comparison was more qualitative. Strap-braced wall panels had the advantage of stable hysteretic loops, but also the disadvantage of higher pinching than the sheeted ones. Comparison between series I and series OSBI, because of the different wall panel arrangements, was more qualitative. Failure of OSB specimens under cyclic loading was more sudden than in the case of corrugated sheet specimens where degradation occurred gradually. This was also reflected by the reduced ductility for OSB specimens. The effect of opening in comparison of series OSB I–series OSB II produced similar results as in the cases of series I–series IV. Initial rigidity, ultimate load and ductility decreased. Results showed that the shear-resistance of wall panels were significant both in terms of rigidity and load bearing capacity, and could effectively resist lateral loads. Failure started at the bottom track in the anchor bolt region; therefore, strengthening of the corner detail would be crucial.

4. CONCLUSION

In recent years important research activities have been undertaken in order to evaluate the earthquake performance of light steel framed structures. Almost all researches consist of characterizing the performance of wall panels experimentally and numerically in order to study the problem of seismic response of these structures. The present paper summarizes the most important research activities carried out in the last few years with the aim to explain the performance and to characterize the specific features of these structures. Usually, the overall behavior of wall panels is mainly addressed. However, according to reviewed papers, the performance of the wall panels, as a whole, is governed by the performance of the connectors e.g.: sheeting-to-sheeting connectors, and sheeting-to-framing connectors.

REFERENCES

[1] Velchev, K., Comeau, G., Balh, N. and Rogers, C.A. (2010). Evaluation of the AISI S213 seismic design procedures through testing of strap braced cold-formed steel walls. *Thin-Walled Structures* **48**,846–856.

[2] AISI. (2007). North American standard for cold- formed steel framing- lateral design. S213. *Washington DC: American Iron and Steel Institute (AISI).*

[3] Moghimi, H. and Ronagh, H.R. (2009). Performance of light-gauge cold-formed steel strap-braced stud walls subjected to cyclic loading. *Engineering Structures* **31**,69-83.

[4] FEMA 450.(2004). NEHRP recommendation provisions for seismic regulations for new buildings and other

structures, 2003 edition. Part 1: provisions. *Washington, DC, USA: Building Seismic Safety Council, National Institute of Building Science.*.
[5] Moghimi, H. and Ronagh, H.R. (2009). Better connection details for strap-braced CFS stud walls in seismic regions. *Thin-Walled Structures* **47**,122–135.
[6] Pan, Ch. L., Shan, M.Y. (2011). Monotonic shear tests of cold- formed steel wall frames with sheathing. *Thin-Walled Structures* **49**,363–370.
[7] Yu, Ch. and Chen, Y. (2011).Detailing recommendations for 1.83 m wide cold-formed steel shear walls with steel sheathing. *Journal of Constructional Steel Research* **67**,93-101.
[8] Yu, Ch. (2010). 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* **32**,1522-1529.
[9] Fiorino, L., Iuorio, O., Macillo, V. and Landolfo, R. (2010). Seismic response of sheathed cold-formed steel structures under catastrophic events. *Urban Habitat Constructions under Catastrophic Events (Proceedings) – Mazzolani (Ed).Taylor & Francis Group, London,* **ISBN 978-0-415-60685-1**.
[10] Dubina, D. (2008). Behavior and performance of cold-formed steel-framed houses under seismic action. *Journal of Constructional Steel Research* **64**, 896–913.