INNOVATIVE CORRUGATED STEEL SHEAR WALLS FOR MULTI-STORY RESIDENTIAL BUILDINGS

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ABSTRACT:

An alternative lateral bracing system for lightly-framed multi-story residential buildings is needed because the customary wood systems in the US cannot be economically designed for height exceeding four stories. A new system comprising corrugated sheet steel shear walls and cold-formed steel stud framing is proposed herein. The key element of this structural system is the corrugated sheet steel shear wall: the lateral load resistance of this structural element originates with the shear strength of the corrugated sheet steel and the shear resistance of the screws connecting the sheeting to the cold-formed steel framing. To establish a design basis, a total of 44 cyclic racking tests were conducted to establish the relation between corrugated sheet steel shear wall design parameters, such as gauge of the sheet steel, gauge of the cold-formed steel framing, size and spacing of the fasteners, and the shear strength of the wall. The results of these tests are presented. Furthermore, system-level R, Cd and Ωo values consistent with the test results are proposed for adoption into design codes. Finally, a design table listing the nominal shear strength values for corrugated sheet steel shear walls is provided. The primary users of the system would be practicing engineers who design light-framed cold-formed steel buildings.

KEYWORDS: cold-formed steel, shear wall, residential buildings, cyclic test

1. INTRODUCTION

The purpose of the research presented herein is to provide practicing engineers with an alternative lateral bracing system for Type II buildings constructed with light-framed cold-formed steel. Its primary use would be in the housing market. Traditional lateral bracing systems for this type of construction include flat steel sheets, steel tension only flat strap bracing, proprietary hot rolled diagonally braced panels, conventional steel braced frames, and a proprietary board which combines gypsum wall board and sheet metal to form a panel which is fastened to the studs. Each of these systems poses design limitations and construction drawbacks.

The proposed shear wall system utilizes a low profile metal deck as sheathing which is fastened to light-framed cold-formed steel framing with screws. The decking is manufactured and sold by several vendors as a non-composite form deck which supports a concrete slab. In keeping with the terminology used to describe shear walls in the International Building Code, the low profile metal deck tested is referred to as "corrugated sheet steel". The term "corrugated sheet steel shear wall" is abbreviated to "CSSSW" in the body of the paper.

The proposed lateral bracing system will give engineers and contractors the ability to design and construct buildings (multifamily housing in particular) with the same flexibility that wood frame construction has. The corrugated sheet steel would take the place of plywood in providing the lateral bracing. The system also has excellent potential for use with prefabricated (panelized) walls and the modular construction of homes. It is non-combustible (to meet the fire requirements of a Type II construction), light (to easily transport and construct), tough (to withstand shipping and handling), strong (to resist high seismic loads), durable (to allow exposure to the elements during construction), and cost-effective. This test results presented herein provide the basis for developing a shear wall design table listing the nominal shear values for wind and seismic forces for shear walls framed with cold-formed steel studs and sheathed with corrugated sheet steel. Funding from the Charles Pankow Foundation and assistance of Dr. Robert K. Tener is gratefully acknowledged.

2. EXPERIMENTAL INVESTIGATION

Design of the experimental investigation, quality control measures, and test acceptance criteria used to develop the data for the proposed CSSSW are based in part on AC154 (March 2000 edition, editorially revised July 2005), Acceptance Criteria for Cyclic Racking Shear Tests for Metal-Sheathed Shear Walls with Steel Framing and in part on AC130, Acceptance Criteria For Prefabricated Wood Shear Panels. The AC 154 protocol was used to test the panels while the AC 130 protocol was use to establish the nominal shear values for the panels.

All specimen panels were fabricated off site by a local contractor, Anning-Johnson, and delivered to the lab. Each specimen was labeled to include the panel number, gauge of studs and tracks, gauge of decking, and size and spacing of the fasteners. Each specimen was inspected by the lab technicians to verify the gauge of the material and the size and spacing of the fasteners. All specimen panels were stored on site after testing.

2.1 CSSSW Test Apparatus

The test apparatus (Figure 1) consists of a Reaction Frame, a specimen Test Frame, and attachment plates. Because of the large number of specimens to be tested and the large variations in applied forces, it was decided to design the Test Frame with a reusable hold-down system that would accommodate forces up to 100-kips to insure the hold-downs would not fail. This approach deviates from the traditional method of having discrete hold-downs and boundary elements in each specimen to simulate in-situ conditions as closely as possible. Given the high shear capacity of the CSSSW system, double angle hold-downs are used in the Test Frame to more accurately represent in-situ conditions of the boundary elements and the hold-downs.

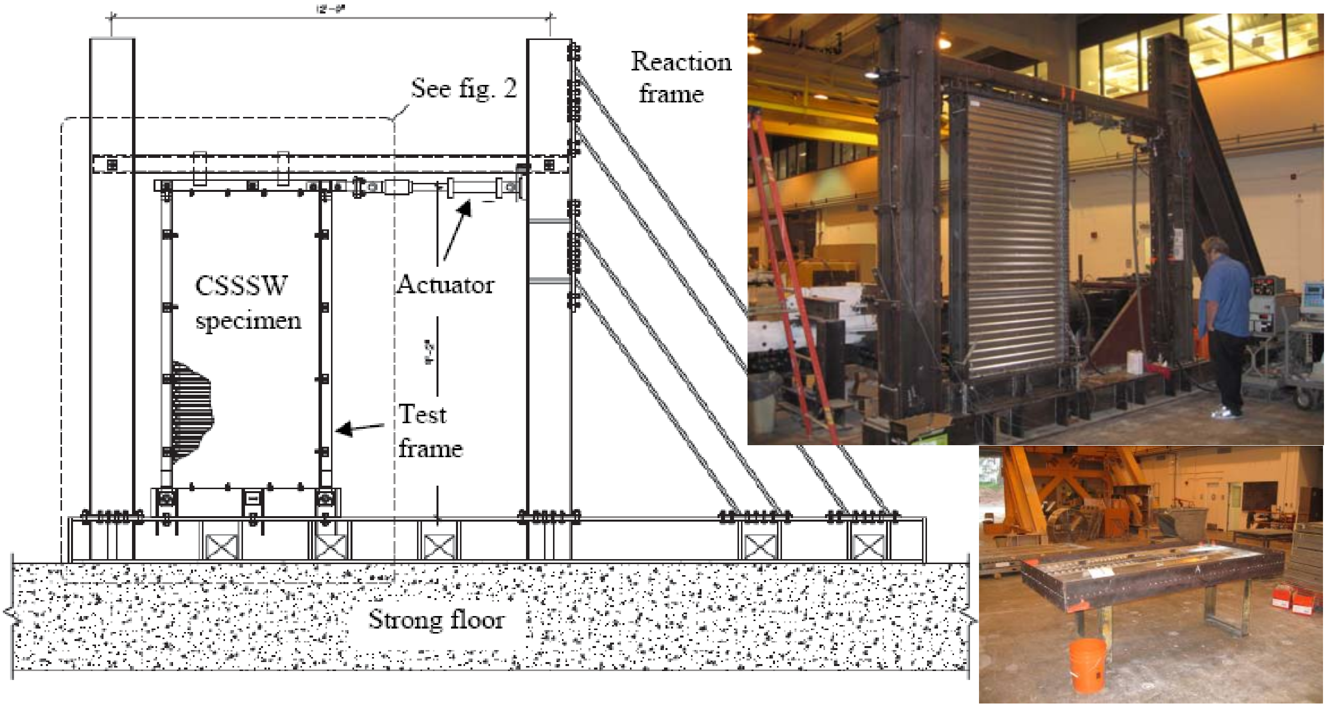

Figure 1. CSSSW Test and Reaction Frames

The Reaction Frame is a semi-permanent frame that belongs to the lab. The vertical members are double C15 x 50's, the diagonal members are W14 x 90's, and the horizontal member which forms the base is a W14 x 145. All members are connected with high strength bolts. The frame is fastened to a strong floor with high strength rods. The specimen Test Frame was designed and fabricated specifically for the subject research. The top member is a double 4 x 4 x 3/8" angle which is attached to the actuator and the test specimen. The two boundary members are double 4 x 4 x 3/8" angles which act as hold-downs to resist uplift forces. The horizontal member which forms the base is a double MC 10 x 28.5. The connections of the Test Frame members were carefully detailed to insure that they did not impede the movement of the specimen during testing and that no added strength or stiffness was transferred through the connection to the specimen. Specifically, the connection of the double angle hold-down connection to the base is a true pinned connection with a slotted hole. The top member of the Test Frame is braced for out of plane buckling by a HSS 6 x 6 x ½" member which spans horizontally just above the frame and has two steel plates extending vertically down through the tube to engage the double angle top member. The Test Frame is connected to the Reaction Frame via a 50-kip actuator with an attached load cell. To facilitate installation and removal of the specimens from the Test Frame, attachment plates were fastened screws spaced at 2" on center (Figure 1). These plates, in turn, were fastened to the Test Frame with high strength bolts. The screws and bolts were sized to insure that these connections would not fail under test loads.

2.2 CSSSW Specimen Design, Instrumentation and Loading

A total of 44 specimens, listed in Table 1, were tested between October and December of 2006 at the Davis Hall Structures Laboratory at the University of California, Berkeley. 40 of the specimens measured 4'-0" wide by 8'-2" high while 4 of the specimens measured 4'-0" wide by 2'-0" high. The typical specimen panel is framed with a top and bottom track, three studs, and corrugated sheet steel sheathing that is fastened to the panel framing (Figure 2). Listed below is a summary of the parameter variables:

1. **Corrugated Sheet Steel**: The corrugated sheet steel (metal decking) was provided by Verco Manufacturing Company. The deck type used was Shallow Vercor (see Figure12) fabricated from G90 galvanized steel conforming to ASTM A653, Grade 50. Three gauges of decking were tested: 22 gauge, 20 gauge, and 18 gauge.

2. **Studs and Tracks**: Generic studs and tracks manufactured per the Steel Stud Manufactures Association (SSMA) were used. Four sizes of studs, with matching tracks, were tested: 362S162-33, 362S162-43, 362S162-54 (50 ksi), and 362S162-68 (50 ksi).

3. **Fasteners**: Three types of fasteners were tested: generic hex head self-drilling screws, a proprietary hex head self-drilling screw by Dynamic Fastener Service, Inc. called Fenderhead, and a pneumatic pin by Aerosmith Fastening Systems. The generic hex head screws tested included #10-16 x ¾", #12-14 x 1 ¼", and #14-20 x 1 ½". The Fenderhead screws tested included #12-14 x 1 ¼" and #14-20 x 1 ½". The pin tested was a .1" diameter x ¾" long x ¼" flat T head.

4. **Fastener Spacing**: Due to the decking profile, the spacing of the fasteners was limited to a 3" module. Fastener spacing at boundaries and seams (horizontal) were tested at either 3" on center or 6" on center while field (vertical) fastener spacing was at 6" on center.

5. **Gypsum Wall Board**: 5/8" gypsum wall board was applied over the corrugated metal decking on two specimens to evaluate its affect on the strength and stiffness of the specimen. The gypsum wallboard was attached to the decking with #6 screws spaced at 6" on center at panel edges and the field.

6. **One Sided and Two Sided Panels**: Two specimens were tested with sheathing on both sides of the panel.

The specimens were organized into groups according to construction type. A total of 24 groups were identified. In accordance with section 4.3 of AC154, a minimum of two identical wall assemblies of a given construction had to be tested. Of the 24 groups, 10 did not have a minimum of two specimens and therefore served only a limited use. Of the remaining 14 groups, the data from 7 were used to develop the final nominal shear values. The number of specimens in each group varied from 2 to 4. Groups 19, 35, and 36, which had only one specimen, were used to evaluate the effects of gypsum board and double-sided panels.

Specimens were instrumented, as shown in Figure 2, to acquire the fundamental force and displacement data, to measure the shear deformation of the panel, and to observe and record any horizontal sliding and/or uplift occurring at the Test Frame connections to the Reaction Frame.

Table 1. CSSSW Specimen Matrix

Group ID	Wall ID	Double-Sided	Panel Length ft.	Metal Decking Thickness ga.	Thickness of Studs & Tracks ga.	Fastener Size (Hex Head) ga.	Fastener Spacing at Boundaries in.	Fastener Spacing at Horiz. Seams in.	Gypboard.
1	3	FALSE	4	22	20	#12-14 x 1 1/4" F	6	6	FALSE
1	18	FALSE	4	22	20	#12-14 x 1 1/4"	6	6	FALSE
2	19	FALSE	4	22	20	#12-14 x 1 1/4"	6	6	TRUE
3	4	FALSE	4	22	20	#12-14 x 1 1/4" F	3	3	FALSE
3	11	FALSE	4	22	20	#12-14 x 1 1/4"	3	3	FALSE
3	20	FALSE	4	22	20	#12-14 x 1 1/4"	3	3	FALSE
4	12	FALSE	4	22	20	#12-14 x 1 1/4"	3	3	TRUE
4	21	FALSE	4	22	20	#12-14 x 1 1/4"	3	3	TRUE
5	35	TRUE	4	22	20	#12-14 x 1 1/4"	3	3	FALSE
6	43	FALSE	4	22	18	#12-14 x 1 1/4"	3	3	FALSE
6	44	FALSE	4	22	18	#12-14 x 1 1/4"	3	3	FALSE
7	24	FALSE	4	22	16	#12-14 x 1 1/4"	6	6	FALSE
7	25	FALSE	4	22	16	#12-14 x 1 1/4"	6	6	FALSE
8	14	FALSE	4	22	16	#12-14 x 1 1/4" F	3	3	FALSE
8	26	FALSE	4	22	16	#12-14 x 1 1/4"	3	3	FALSE
9	31	FALSE	4	22	16	#12-14 x 1 1/4"	3	2	FALSE
9	32	FALSE	4	22	16	#12-14 x 1 1/4"	3	2	FALSE
10	37	FALSE	2	22	16	#12-14 x 1 1/4"	3	3	FALSE
10	38	FALSE	2	22	16	#12-14 x 1 1/4"	3	3	FALSE
11	5	FALSE	4	20	18	#12-14 x 1 1/4" F	3	3	FALSE
11	8	FALSE	4	20	18	#12-14 x 1 1/4" F	3	3	FALSE
11	22	FALSE	4	20	18	#12-14 x 1 1/4"	3	3	FALSE
12	16	FALSE	4	20	16	#12-14 x 1 1/4" F	3	3	FALSE
12	27	FALSE	4	20	16	#12-14 x 1 1/4"	3	3	FALSE
13	17	FALSE	4	18	18	#12-14 x 1 1/4" F	3	3	FALSE
13	28	FALSE	4	18	18	#12-14 x 1 1/4"	3	3	FALSE
14	9	FALSE	4	18	16	#12-14 x 1 1/4" F	3	3	FALSE
14	29	FALSE	4	18	16	#12-14 x 1 1/4"	3	3	FALSE
14	30	FALSE	4	18	16	#12-14 x 1 1/4"	3	3	FALSE
14	42	FALSE	4	18	16	#12-14 x 1 1/4"	3	3	FALSE
15	36	TRUE	4	18	16	#12-14 x 1 1/4"	3	3	FALSE
16	10	FALSE	4	18	16	#14-20 x 1 1/2" F	3	3	FALSE
16	33	FALSE	4	18	16	#14-20 x 1 1/2"	3	2	FALSE
16	34	FALSE	4	18	16	#14-20 x 1 1/2"	3	2	FALSE
16	41	FALSE	4	18	16	#14-20 x 1 1/2"	3	3	FALSE
17	39	FALSE	2	18	16	#14-20 x 1 1/2"	3	3	FALSE
17	40	FALSE	2	18	16	#14-20 x 1 1/2"	3	3	FALSE
18	7	FALSE	4	20	18	#12-14 x 1 1/4" F	6	6	FALSE
19	1	FALSE	4	22	20	#10-16 x 3/4"	6	6	FALSE
20	2	FALSE	4	22	20	#10-16 x 3/4"	3	3	FALSE
21	6	FALSE	4	22	20	0.1 x 3/4" Pin	1.5	1.5	FALSE
22	13	FALSE	4	22	18	#12-14 x 1 1/4" F	3	3	FALSE
23	15	FALSE	4	20	20	#12-14 x 1 1/4 F	3	3	FALSE
24	23	FALSE	4	18	14	#14-20 x 1 1/2"	3	3	FALSE

The loading sequence, shown in Figure 3, consists of both stabilizing cycles and decaying cycles. The loading velocity varied between 0.16 in/sec and 1.92 in/sec during each of the tests. Each test was ended with a final 5”+ and 5”– excursion which represents an inter-story drift of 5%. The amplitudes of the displacement cycles were defined in terms of the Approximate Elastic Displacement (AED), the first significant change to occur in the applied force-displacement response of a monotonic or cyclic test of the shear wall. To estimate the AED for this research, a CSSSW specimen was subjected to the AC154 loading sequence with the AED set at 0.8 inches and using a constant loading velocity of 0.1 inches per second. The new AED, which was used for all subsequent tests, was determined by noting the displacement at the first yield-point (first significant change in the applied force-displacement response).

2.3 CSSSW Specimen Response Data

Data analysis was carried out in accordance with section 3.3 of AC154 with the exception of section 3.3.5, in which case the first hysteretic loop of the last set of stable hysteretic load/displacement loops was used in accordance with AC130 rather than the second hysteretic loop. A computer program was written to process the data and plot the graphs. A force-displacement hysteresis curve was plotted for each specimen: the graph in Figure 3 is representative of a typical specimen.

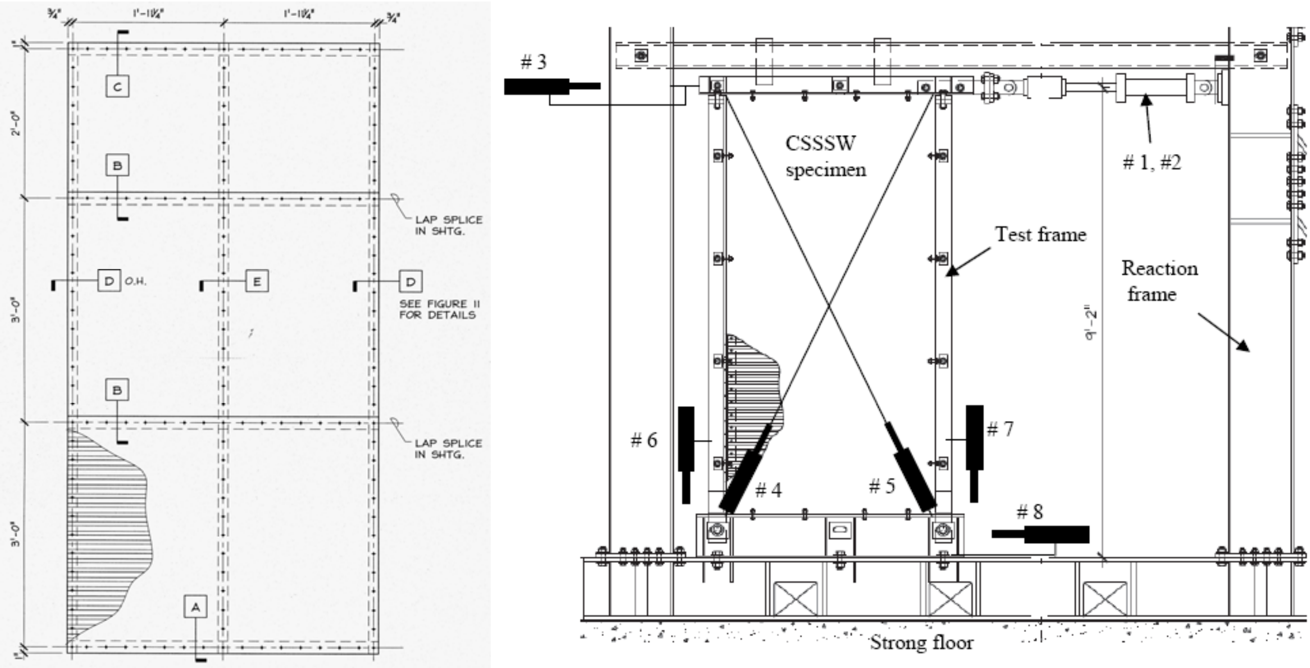

Figure 2. CSSSW Specimen Details and Instrumentation

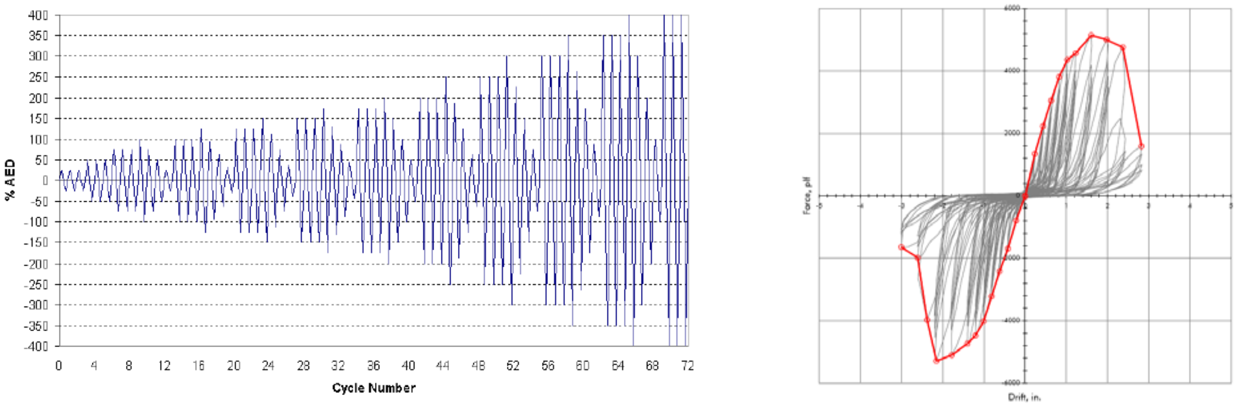

Figure 3. Quasi-Static Displacement-Controlled Loading Sequence and Typical CSSSW Specimen Response

In accordance with sections 3.3 and 4.3 of AC154, the test data for the specimen groups was averaged. A computer program was written to analyze the data and plot the backbone curves. Backbone curves for each group were plotted. The graph in Figure 4 is representative of a typical group.

2.4 CSSSW Specimen Behavior

Of primary interest is the failure mode of the specimen panels. In all cases, the failure mode was the eventual "popping" out of the screws due to warping of the corrugated sheet steel. It was found that as the panels cyclically deformed, the screws would eventually gouge elongated holes in the metal studs and/or sheeting due to racking shear. As the inter-story drift increased, warping of the corrugated sheet steel became more pronounced and simultaneous diagonal tension and compression fields developed across the panel. As the holes in the studs enlarged, the tensile capacity of the screws was reduced and eventually the screws failed in tension due to the warping of the corrugated sheet steel and "popped" out (Figure 4). It is also interesting to note the location of the screws that first "popped" out. In all cases, the first screws to "pop" out were located in the boundary members. The location of the screws that "popped" along the boundary members was random. The locations varied from top to bottom on both the left and right boundary members. The screws fastened into the top track, the bottom track, and the horizontal seams were never the first to fail.

The corrugated sheet steel was installed with the corrugations running horizontally. Two horizontal seams were required to construct a typical specimen. Adjacent sheets were overlapped one corrugation and fastened together with screws of the same size and spacing as the boundary condition. Based on the test results, it was concluded that no special blocking is required at horizontal lap splices. Although no vertical seam splices were tested, the authors believe this is an important detail that should be discussed. The vertical seam splice can be butted at the center line of a vertical framing member, it can be lapped, or in the case of prefabricated wall panels, two panels could be joined by fastening studs together. In any case, this splice is a boundary condition and fasteners should be spaced at the same spacing as all panel edges. In discussing the splice options with a contractor, their preference was to lap the sheets between the studs rather than butt them at the stud because the lap splice would require half the number of screws. The lap splice should be sufficient length to insure development of the shear capacity of the fastener, say 1" minimum. As in the case of the horizontal lap splice, it was concluded that no special blocking is required at vertical lap splices.

Three of the specimens were sheathed with 5/8"gypsum board. The purpose of adding the gypsum board was to evaluate how it affected the strength and stiffness of the test specimen compared to a similar one without gypsum board. A comparison of the backbone curves for the three specimens compared to similarly constructed specimens without the gypsum board (Group 1 vs. Group 2 and Group 3 vs. Group 4) shows little difference between the groups. Based on this comparison, it appears that the addition of gypsum board to a wall sheathed with corrugated metal sheet will not materially change its behavior.

To evaluate the effect of adding electrical outlets, light switches, plumbing lines etc. to an actual wall panel, three of the test specimens had appropriate openings cut in them. Specimen 24 had a 4" diameter hole cut in the upper left hand corner of the panel. Specimen 25 had a 2" by 4" hole cut in the lower left hand corner of the panel. Specimen 42 had a 4" diameter hole cut in the upper left hand corner of the panel an To represent the affect of adding electrical outlets, light switches, plumbing lines, etc. to an actual wall d a 2" by 4" hole cut in the lower left hand corner of the panel. Test observation noted that the panels warped around the holes with no affect on the overall performance of the specimens.

To determine the effect of adding the corrugated sheet steel to both sides of a specimen, two specimens were tested. Specimen 35 (Group 5) was constructed using 20 gauge studs and 22 gauge corrugated sheet steel to represent a more lightly loaded wall while Specimen 36 (Group 15) was constructed with 16 gauge studs and 18 gauge corrugated sheet steel to represent a more heavily loaded wall. When comparing the results of Group 5 to Group 3, of similar one sided construction, and the results of Group 15 to Group 14, of similar one sided construction, it was found that the double sided specimens achieved allowable strengths that are basically double those of the one sided specimens. Based on these results, it is concluded that double sided walls have double the shear strength.

In order to determine how slenderness affects on the CSSSW system, four 24" wide specimens were tested. These include Specimens 37 and 38 (Group 10) and Specimens 39 and 40 (Group 17). Group 10 was constructed using 16 gauge studs and 22 gauge corrugated sheet steel to represent a more lightly loaded wall while Group 17 was constructed with 16 gauge studs and 18 gauge corrugated sheet steel to represent a more heavily loaded wall. When comparing the results of Group 10 to Group 8, 48" wide panels of similar construction, and the results of Group 17 to Group 16, 48" wide panels of similar construction, it was found that the 24" panels are slightly stronger than the 48" panels from a force standpoint; however, from a deflection standpoint the allowable shear values drop substantially due to the flexibility of the panels. This is to be expected. The current US Building Code addresses this issue by requiring the allowable strength of a panel to be reduced when the aspect ratio exceeds 2:1. The authors believe this is an appropriate approach for the CSSSW system.

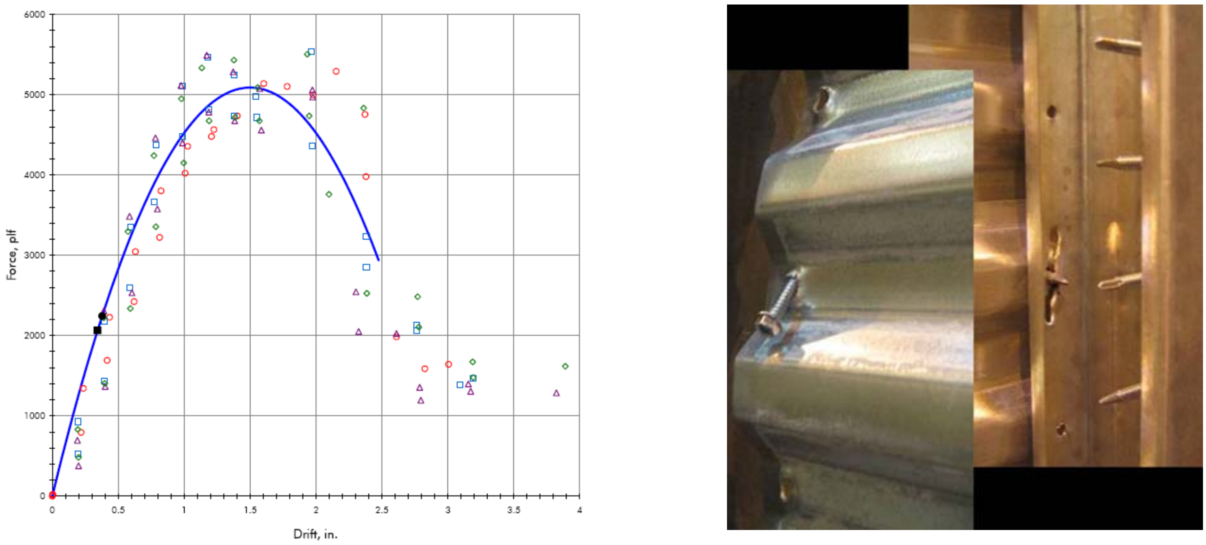

Figure 4. Typical CSSSW Group Response and Failure Mode

3. SEISMIC DESIGN OF THE CSSSW SYSTEM

The relevant factors that determine the design strength of seismic force resisting systems consist of the Response Modification Coefficient (R), the Deflection Amplification Factor (C_d), and the System Over-strength Factor, (Ω_o) and the associated nominal shear strength per unit length of the CSSSW system. Establishing appropriate values for these parameters relies somewhat on engineering judgment to maintain a consistent and rational relationship between both actual test results obtained form CSSSW specimens, assumed CSSSW system response, and the historically accepted codified values for other similar structural systems.

3.1 Seismic Response Parameters R, Ω_o, Cd

ASCE/SEI 7-05, "Minimum Design Loads for Buildings and Other Structures," assigns light-framed bearing wall systems using wood structural panel or steel sheathing a R value of 6.5, a C_d value of 4.0, and an Ω_o value of 3.0. Since the CSSSW system is a variation of the above defined bearing wall system (using corrugated metal sheathing rather than flat metal sheathing or plywood sheathing on wood studs), an evaluation of the seismic response factors was done to see if they were appropriate for the CSSSW system. A R value of 6.5 and a C_d value of 4.0 were assigned to determine the controlling shear forces and associated drifts per the AC 130 protocol. A review of the data found that all 7 groups used to develop the Nominal Shear Strength were controlled by the drift limit. Further review of the data found the C_y/C_s values ranged from 1.84 to 2.24 versus the assumed value of 1.79 (2.5/1.4 = 1.79). To provide a comparison, the R value was lowered to 5.5 , the Ω_o value was lowered to 2.5, and the C_d value was lowered to 3.25 and the controlling shear forces and associated drifts were again determined per the AC 130 protocol. A review of the data found that of the 7 groups actually used to develop the Nominal Shear Strength, 6 of the groups were controlled by the ultimate load limit while only 1 was controlled by the drift limit. For the drift controlled group, the C_y/C_s value was 1.89 versus the assumed value of 1.79 (2.5/1.4 = 1.79). It was observed that lowering the R and C_d values as noted shifts the walls from drift controlled to force controlled and more accurately predicts the over strength factor.

Based on the observations noted above, the authors are proposing a R value of 5.5, a C_d value of 3.25, and a Ω_o value of 2.5 be assigned to the corrugated metal shear walls.

3.2 Nominal Shear Strength Values

Nominal shear strength values for design of CSSSW structural elements were obtained from the above analysis, such that they are consistent with the structural response parameters. The values presented in Table 2 are normalized per unit length of the CSSSW. The authors are proposing these values for use in design.

Table 2. Proposed Nominal Shear Strength Values for the CSSSW Structural System.

Nominal Shear Strength (R_n) for Wind and Seismic Loads for shear walls faced with corrugated sheet steel. (pounds per foot) [1, 3, 4, 7]

Assembly Description[5, 6]		20 gauge studs	18 gauge studs	16 gauge studs	
		#12 screws	#12 screws	#12 screws	#14 screws
Sheathing	Screw Spacing[2]	Shear (plf)	Shear (plf)	Shear (plf)	Shear (plf)
22 gauge	6" o.c.	1173	1505	1836	---
	3" o.c.	---	3050	3290	---
18 gauge	3" o.c.	---	4144	5164	5874

1 *Nominal* shear strength shall be multiplied by the *resistance factor* (ϕ) to determine the *design strength* or divided by the *safety factor* (Ω) to determine *allowable* shear strength as set forth in Section C2.1.

2 Screws in the field of the panel shall be installed 6 inches o.c. unless otherwise shown.

3 A *shear wall* height to width aspect ratio (h/w) greater than 2:1, but not exceeding 4:1, is permitted provided the *nominal* shear strength is multiplied by 2w/h.See Section C2.1.

4 See Section C2.1 for requirements for sheathing applied to both sides of wall.

5 Unless noted as (min.), substitution of a stud or track of a different designation thickness, per the *General Provisions*, is not permitted.

6 Wall studs and track shall be ASTM A1003 Grade 33 Type H steel for members with a designation thickness of 33 or 43 mil, and A1003 Grade 50 Type H steel for members with a designation thickness equal to or greater than 54 mils.

7 For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1lb = 4.45 N

REFERENCES

AC130 (2004), Acceptance Criteria for Prefabricated Wood Shear Panels. ICC Evaluation Service, Inc. Whittier, CA, USA, www.icc-es.org (Effective July 1, 2004).

AC154 (2000), Acceptance Criteria for Cyclic Racking Shear Tests for Metal-Sheathed Shear Walls with Steel Framing. ICC Evaluation Service, Inc. Whittier, CA, USA, www.icc-es.org (Editorially revised July 2005).

AISC (2004) Standard for Cold-Formed Steel Framing – Lateral Design. American Iron and Steel Institute, USA, www.steel.org

ASCE 7-05, (2005) Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Washington, DC, USA, www.asce.org

IBC (2006), International Building Code, International Code Council, Washington DC, USA www.iccsafe.org