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Behavior of Steel-Sheathed Shear Walls Subjected to Seismic and Fire Loads

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Abstract

A series of tests was conducted on six 2.7 m × 3.7 m shear wall specimens consisting of cold-formed steel framing sheathed on one side with sheet steel adhered to gypsum board and on the opposite side with plain gypsum board. The specimens were subjected to various sequences of simulated seismic shear deformation and fire exposure to study the influence of multi-hazard interactions on the lateral load resistance of the walls. The test program was designed to complement a parallel effort at the University of California, San Diego to investigate a six-story building subjected to earthquakes and fires. The test results reported here indicate that the fire exposure caused a shift in the failure mode of the walls from local buckling of the sheet steel in cases without fire exposure, to global buckling of the sheet steel with an accompanying 35 % reduction in lateral load capacity after the wall had been exposed to fire. This behavior appears to be predictable, which is encouraging from the standpoint of residual lateral load capacity under these severe multi-hazard actions.

Keywords

shear walls; cold-formed steel; fire following earthquake

INTRODUCTION

In June of 2016, experimental investigations of the performance of a six-story, cold-formed steel (CFS) framed building (Fig. 1) were conducted on the Large High-Performance Outdoor Shake Table (LHPOST) at the University of California, San Diego (UCSD). The

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building's lateral load resistance system consisted of cold-formed steel framing members sheathed by panels of sheet steel adhered to gypsum board. These and other light-weight construction material lateral load-resisting systems are widely used in seismic regions in the western United States, where they offer significant advantages in construction costs and speed. For information about the design and construction of these wall systems for seismic applications, interested readers are referred to the National Earthquake Hazards Reduction Program (NEHRP) document *Seismic Design of Cold-Formed Steel Lateral Load-Resisting Systems - A Guide for Practicing Engineers* [1]. The aim was to study the earthquake performance of this construction method for midrise structures (five to ten stories), as well as the earthquake-damaged building's response to fire. After the fire tests, additional earthquake shaking was conducted to study the response of the fire-damaged building to earthquake aftershocks. The aftershock test results were intended to help inform decisions about first-responder access to a building in the case of fire following earthquake, as well as repair versus replace assessments. Details about the six-story building tests are provided in [2].

The tests reported in this paper were conducted immediately prior to the six-story building shake table tests. The objective was to experimentally determine the influence of a specific fire load – the one to be used at UCSD – on the lateral load resistance of the investigated shear walls to help inform the selection of the earthquake motion intensities used in the UCSD tests before and after the fires. These tests enhanced the value of the full structure experiments and provide insight into multi-hazard interaction for this construction method.

TEST PROGRAM

The tests were conducted using six wall specimens at the National Fire Research Laboratory (NFRL) at the National Institute of Standards and Technology (NIST) in Gaithersburg, Maryland. Table 1 provides an overview of the test program. The specimens were subjected sequentially to varied combinations of mechanical (shear) deformation and thermal (fire) loading. Specimen 1 was used to establish the monotonic “pushover” load-displacement capacity of the wall system and subsequently to shake down the fire test setup. Specimen 2 was loaded by symmetric-amplitude reverse-cyclic shear deformation to destruction (defined here as 2.8 % drift ratio) to establish the cyclic load-displacement response. Specimen 3 and 4 were cycled to deformations just before and after the peak load was achieved, respectively, burned for 13 minutes and 20 seconds and then cycling was continued until destruction of the wall. For Specimen 5, an undamaged wall was exposed to fire for 13 minutes and 20 seconds and then cycled to destruction. Specimen 6 was tested similarly to Specimen 3, however, the burn duration was doubled. The test program was intended to bound the effects of fire and earthquake shaking on the shear capacity of the walls.

Specimens

Six 2.7 m × 3.7 m shear wall test specimens consisting of cold-formed steel framing sheathed on one side with sheet steel adhered to gypsum board and on the opposite side with plain gypsum board were fabricated. The walls were full-scale, and selected 1) to emulate a common geometry found in multi-residential building construction and 2) to approximate a

shear wall along one side of the corridor on the 2nd floor of the UCSD six-story building (Fig. 1b). They were designed in compliance with current building code provisions within the American Society of Civil Engineers (ASCE) standard ASCE/SEI 7 *Minimum Design Loads for Buildings and Other Structures* [3] and the American Iron and Steel Institute (AISI) documents AISI S100 *North American Specification for the Design of Cold-Formed Steel Structural Members* [4] and AISI S213 *North American Standard for Cold-Formed Steel Framing—Lateral design* [5]. Dimensional details of the test specimens are given in Fig. 2. Additional details can be found in [6].

All vertical framing members were 1.7 mm thick cold-formed steel studs (600S200-68) 152 mm wide with a flange width of 51 mm. The top and bottom tracks were 1.4 mm thick cold-formed steel channels (600T150-54) 152 mm wide with a flange width of 38 mm. The top and bottom tracks were drilled with two rows of 17.5 mm diameter holes at 305 mm on center (OC) to allow for attachment to the loading frame. All of the cold-formed steel sections were Structural Grade 50, Type H (ST50H) conforming to the American Society for Testing and Materials (ASTM) standards ASTM A653 [7] and ASTM A1003 [8] with a minimum specified yield strength of 345 MPa. All fasteners used to connect framing members were 19 mm long self-tapping, self-drilling sheet metal screws with shank diameter of 4.8 mm. The boundary elements (stud packs) were secured back-to-back with two rows of screws at 305 mm on center. All other joints had one screw where flanges met.

The side of the wall to be fire tested (corridor side) was sheathed with one layer of 2.7 m × 1.2 m Sure-Board 200[®] panels which consisted of 0.686 mm thick sheet steel adhered to 16 mm thick type X gypsum board. The panels were attached with 45 mm long self-tapping, self-drilling sheet metal screws with shank diameter of 4.2 mm. The screw spacing was 76 mm on center on the board perimeter and 305 mm on center in the field. The opposite side of the wall (cold side) was sheathed with one layer of 2.7 m × 1.2 m type X gypsum boards 16 mm thick. The panels were attached with 32 mm long self-tapping, self-drilling sheet metal screws with shank diameter of 4.2 mm. The screw spacing was 152 mm on center on the board perimeter and 305 mm on center in the field. The seams were parallel to the studs (vertical) and were not staggered on the front and back side of the wall. The panel seams on the side of the wall to be fire tested were mudded and taped and the screw heads were mudded over. These wall assemblies have a 60-minute fire resistance rating per ASTM E119 [9] in the undamaged condition.

Test Setup

The test setup was informed by ASTM E2126-11 [10], but deviations were made as required by this test program. A controlling condition for the setup was that a burn compartment on a rolling platform needed to be rolled into position and removed multiple times during testing. This necessitated clear access to the specimen from one side. Details can be found in [6].

Mechanical Tests—The test specimens were loaded mechanically by holding the base of the wall fixed and applying a prescribed in-plane deformation to the top of the wall (Fig. 3). The top of the wall could rotate. Out-of-plane movement of the wall was limited by four structural steel guide frames placed perpendicular to the wall. The wall was attached to the

bottom framing beam (W16×26) by two rows of 16 mm A325 structural bolts (20 total) each with an applied torque of 136 N-m. In between the compression stud packs at each end of the walls, two 16 mm diameter Grade 8 steel threaded rods ran from the top of the wall to the bottom. At the bottom the rods passed through a 114 mm × 152 mm × 19 mm plate washer and were torqued to 136 N-m to the bottom framing beam. The attachments at the top of the wall were similar, however, the threaded rods were nutted only from above the top channel (MC6×15.3) and torqued to 68 N-m. The top of wall loading channel was additionally stiffened against bending at the actuator end using a W6×12 steel section.

Mechanical load was applied to the specimen using a servo-hydraulically controlled actuator (MTS 201.35TS) with a load capacity of 240 kN in tension and 365 kN in compression. The maximum stroke of the actuator was ±381 mm. The actuator was controlled using a MTS Flextest 100 controller. Axial loading to the wall was limited to the self-weight of the specimen, actuator and top loading beam. Vertical loading of the wall was neglected to avoid the secondary effects of uplift and gravity on the stud packs.

Fire Tests—The thermal load on the test specimens was provided by a natural gas diffusion burner located in a compartment (interior dimensions: 2.9 m × 3.5 m × 1.2 m) designed to approximate a portion of the corridor in the six-story building tested at UCSD. The constructed compartment is shown in Fig. 4a. The compartment was lined with two layers of 16 mm type C gypsum board. The open side of the compartment that mated with the test specimen was lined with 25 mm thick thermal ceramic blanket to provide a seal against smoke and flame leakage. The sides and top of the compartment overlapped the edges of the wall specimen approximately 0.1 m and the top of the burner was slightly above the bottom of the wall (Fig. 4b). The openings at the ends of the compartment were 1.7 m high by 1.2 m wide.

Testing Procedure

Cyclic Loading—ASTM E2126-11 [10] Method C (CUREE Basic Loading Protocol) was used with a reference deformation equal to 38.1 mm for all cyclic tests (CFS02 to CFS06). This protocol is widely used in the United States to characterize the seismic horizontal load resistance of vertical elements intended to form the lateral force resisting system in buildings. The reference deformation was based on the deformation at peak load during the monotonic test (CFS01a). The loading procedure involved displacement cycles grouped in phases at incrementally increasing displacement levels. The loading history starts with a series of six initiation cycles at small amplitudes. Subsequently, each phase of the loading history consists of a primary cycle with amplitude expressed as a fraction (percent) of the reference deformation and subsequent trailing cycles with amplitude of 75 % of the primary one. The rate of displacement was selected to be 1.52 mm/s to minimize inertial influences, while maintaining efficient test duration. The use of a quasi-static loading rate is reasonable because the mechanical behavior of the wall assembly is deformation-driven, rather than force-driven; i.e., behavior is not governed by inertial forces in the wall. The schedule of amplitude increments along with the step durations is given in Table 2.

Fire Loading—The fire load was delivered by a single natural gas diffusion burner. The mass flow rate of the natural gas was controlled to approximate the predicted time versus temperature curves for upper gas layer temperatures in the 2nd floor corridor of the six-story building at UCSD. This was achieved by rapidly increasing (rise time less than 60 seconds) the Heat Release Rate (HRR) from the burner to 1900 kW and nominally holding the HRR constant for 800 s (13 m 20 s); in test CFS06b the burn duration was doubled. The upper layer gas temperature during the heating phase predicted by the Consolidated Model of Fire and Smoke Transport (CFAST) [11] two-zone fire model is shown in Fig. 5. While the fire was more severe than the “standard fire curve” in International Organization for Standardization (ISO) standard ISO 834 [12], it was representative of conditions present in residential fires in modern buildings; for example, see [13]. Additionally, the burn duration for the UCSD tests, which the presented component tests were intended to inform, was limited to about 15 minutes by the local fire authorities due to concerns about performing an outdoor burn close to the dry season in Southern California. Additional details about the fire load development are provided in [6].

Instrumentation

Data were acquired using two National Instruments cDAQ-9188 chassis: one for channels located on test specimen or the loading frame and the other for channels on the burn compartment. The sensor locations are illustrated in Fig. 6 and Fig. 7. The arrows in Fig. 6 indicate the directions of the actions in the reported data (actuator compression = positive). Data were reported at a sampling frequency of 5 Hz during the cyclic loading tests and 1 Hz during the fire tests.

Load applied to the top of the specimen was measured at the actuator with a calibrated load cell with standard uncertainty of $\pm 0.15\%$. Longitudinal displacement at the top of the specimen (drift) was measured using a UniMeasure string potentiometer (PA30) with a standard uncertainty of $\pm 0.10\%$. Temperatures were measured using 24-gage bare-bead K-type thermocouples with standard uncertainty of the greater of $\pm 0.75\%$ or $2.2\text{ }^{\circ}\text{C}$. Correction for radiation to the thermocouple beads was neglected. The remaining channels shown are not reported in this paper but can be found in [6].

RESULTS

The results are examined first for an example case for combined cyclic loading and fire exposure. Subsequently, series-to-series comparisons are made. Test results for the monotonic loading case (CFS01) are not reported due to a problem with the stiffness of the setup that was corrected in the subsequent tests.

Typical Case

Specimen 3 represents a typical loading case in this study. The specimen was first cycled to a prescribed drift (CFS03a), subjected to fire load (CFS03b) and then, after the wall cooled, mechanical cycling was continued until the wall failed (CFS03c). The resulting load-displacement curve is shown in Fig. 8. The local maxima (load) for each step in the loading pattern are indicated for compression (circles) and tension (squares) excursions. The curves

defined by these maxima show an abrupt reduction of force after the specimen was subjected to fire.

Photographs of the gypsum-steel sheathed side of the specimen after each of the three stages of the test are shown in Fig. 9. It can be seen in Fig. 9a that the damage to the wall after cycling to a 1 % target drift ratio was limited to cracking and tearing of the tape and mud along the panel seams. The extent of the damage to the walls at 1 % interstory drift ratio was similar to that later observed in the six-story building tested at UCSD during the design level earthquake [2] and was consistent with the expectations of the panel manufacturer based on extensive (proprietary) seismic qualification testing of similar wall assemblies. The fire test caused severe damage to the gypsum adhered to the sheet metal (Fig. 9b), effectively eliminating any fire protection and structural capacity that the gypsum provided. During the post-fire cycling, global buckling of the sheet steel and tear-through of the screws through the sheet steel around the perimeters of the panel controlled the post-peak behavior (Fig. 9c).

A fire test (CFS03b) is depicted in Fig. 10. As shown by Fig. 11, the upper layer gas temperatures (TC1 to TC5) rose rapidly to about 800 °C and continued to increase to between 900 °C and 1000 °C over the 13 m 20 s duration of the test. The thermocouple trees (with TC1 25 mm from the compartment ceiling and subsequent thermocouples at 305 mm spacing below) show a symmetric heating of the compartment at both ends. A significant vertical gradient in the gas phase temperatures is present. Although the shape of this gradient was specific to the geometry of the investigated compartment, transition from a hot upper layer to a cool lower layer is characteristic of a ventilated compartment in a real fire. The influence of this thermal gradient is reflected in the greater damage to the gypsum at the top of the wall after the fire loading (Fig. 9b). Data from the thermocouples located inside of the wall cavity are shown in Fig. 12. A heating plateau is visible at 100 °C as the water is driven off from the gypsum board. The duration of the plateau is about 10 minutes for thermocouples placed on the sheet metal in the center of the panel (Wall), but is shorter at the stud (only top thermocouple available in this test), where the tape and mud was damaged during the previous mechanical cycling to 1 % drift ratio.

Comparisons

The enveloping curves defined by the local maxima of applied force versus drift for the five cyclic tests are compared in Fig. 13. Displacements (drifts) measured at the top of the specimens are converted to drift ratios by dividing by the specimen height (2.7 m). The portions of the curves indicated by dashed lines represent the mechanical response in the post-fire test stage. It is noted that the measured drift ratios are slightly smaller than the prescribed primary cycle drift amplitudes, which were controlled by the actuator displacement.

Test CFS02 represents the stiffness and capacity of the wall under ambient conditions. Test CFS05 represents the stiffness and capacity of the specimen after the steel sheathed side has been subjected to the investigated fire load for 13 m 20 s. The reduction in peak load capacity was 35 % (in compression) and the response was roughly symmetric for tension and compression cycles. The reduction in the peak load was accompanied by a shift in failure mode of the specimens from local buckling of the sheet steel (Fig. 14a) to global

buckling of the sheet steel (Fig. 14b) for the unburned (CFS02) and burned (CFS05) walls, respectively. The fire severely damaged the gypsum on the burn side reducing the stiffness of the shear panels out-of-plane and creating a 16 mm standoff between the screw heads and the sheet steel; the thickness of the lost gypsum. This, in effect, changed the specimen to a plain sheet steel shear wall with reduced constraint around the panel boundaries. The global buckling mode is consistent with observations of the seismic response of laterally loaded steel sheathed steel stud walls (i.e. walls without gypsum), where the added bonded gypsum is not present to restrain the buckling evolution [14]. Pre-damaging the specimen by reversed shear cycling to 1 % (CFS03) or 1.8 % (CFS04) drift ratio prior to the fire loading had no noticeable influence on the residual load bearing capacity of the wall. The fire load alone was the trigger to shift the load-displacement behavior between that for a sheet steel wall with adhered gypsum (CFS02) to that for a sheet steel wall without adhered gypsum (CFS05).

Doubling the burn time to 26 m 40 s (CFS06) caused additional reduction (11 % to 18 % in compression and tension, respectively) of the post-fire lateral load bearing capacity. This is likely due to the damage to the nonstructural gypsum board on the back side (cold side) of the wall during the longer burn, which was not present in the shorter tests.

Although the investigated time-temperature curves were more aggressive than a standard furnace test curve, the observed influence of temperature on the subsequent mechanical behavior of the wall assemblies is expected to be similar after a furnace test. This is because the observed changes in mechanical behavior are caused by loss of adhesion between the sheet steel and the gypsum board and deterioration of the gypsum during heating. The duration of heating to achieve comparable degradation of the gypsum, however, would likely be greater in a furnace test. Furthermore, uniform heating of the wall, as opposed to the vertical gradient in temperatures present in our tests, might affect the spatial distribution of the gypsum degradation and consequently the shape of the global buckling pattern. Additional testing is recommended to generalize these findings.

CONCLUSIONS

Six cold-formed steel framed shear walls (2.7 m × 3.7 m) sheathed on one side with sheet steel adhered to gypsum board were experimentally investigated to study the interaction between lateral load capacity and fire damage. The work was undertaken to complement a series of system-level tests of a six-story building subsequently performed at the University of California, San Diego in which a building was subjected to strong earthquake motions, fire and then earthquake aftershocks. The results in this paper indicate that the fire loading representative of modern furnished residential spaces, which is more demanding than the ISO standard fire curve, caused a shift in the failure mode of the walls under lateral loading from local to global buckling of the sheet steel with an accompanying 35 % reduction of the load capacity. Despite the demanding conditions placed on the specimens, the shear walls maintained a predictable, but reduced, lateral load capacity compared to ambient conditions. This information can be useful for first responder judgements regarding safety when entering this type of structure following an earthquake, as well as post-earthquake assessment of the structural integrity. Further testing is needed to investigate the

repeatability of these findings as well as to study the impact of different fire load scenarios, fire resistance ratings and other common methods of shear wall construction.

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Fig. 1. Cold-formed steel framed building at the LHPOST facility: (a) Photograph of the building on the shake table (looking at south west corner); (b) Typical floor plan (after [2]).

Fig. 2.
Specimen geometry (units in meters unless noted).

(a)

(b)

Fig. 3. Test setup: (a) Photograph of mechanical loading setup from the West (cold side); (b) Schematic drawing of longitudinal cross-section.

(a)

(b)

Fig. 4.

(a) Photograph of compartment with natural gas burner; (b) Schematic drawing of transverse cross-section with wall specimen in place.

Fig. 5. Target upper layer temperature versus time during heating phase predicted by CFAST for the NIST tests with ISO 834 standard fire curve shown for reference.

Fig. 6. Sensor locations on specimen and loading frame. Thermocouples (TCs) are located on the back of the steel sheathing (inside the wall cavity).

Fig. 7. Sensor locations within the interior of the burn compartment (left = longitudinal; right = transverse).

Fig. 8.
Lateral load versus drift during mechanical loading.

Fig. 9.

Sequence of photographs of steel sheathed (corridor) side of wall: (a) After mechanical loading to 1.0 % drift ratio (CFS03a); (b) After fire loading for 13 m 20 m (CFS03b); (c) After mechanical loading to 2.8 % drift ratio (CFS03c).

Fig. 10.
Fire loading: (a) View from back of compartment; (b) Video snapshot inside compartment.

Fig. 11.
Compartment temperatures during fire loading (CFS03b): (a) North thermocouple tree; (b) South thermocouple tree.

Fig. 12.
Specimen temperatures during fire loading (CFS03b).

Fig. 13.
Enveloping curves for CFS02 to CFS06.

Fig. 14.
Photograph of back of steel sheathed side of wall after mechanical loading to 2.8 % drift ratio with the nonstructural gypsum removed: (a) Unburned wall after cycling; (b) Cycling after burning.

Table 1

Test program.

Test name	Specimen	Description	Loading rate / Amplitude
CFS01a	CFS01	Monotonic pushover	Push @ 2.54 mm/minute
CFS01b		10 m burn	Multiple steps to 1900 kW
CFS02	CFS02	Cycling to failure	1.52 mm/second
CFS03a	CFS03	Cycling to 1 % drift	1.52 mm/second
CFS03b		13 m 20 s burn	Step to 1900 kW
CFS03c		Continue cycling until failure	1.52 mm/second
CFS04a	CFS04	Cycling to 1.8 % drift	1.52 mm/second
CFS04b		13 m 20 s burn	Step to 1900 kW
CFS04c		Continue cycling until failure	1.52 mm/second
CFS05a	CFS05	13 m 20 s burn	Step to 1900 kW
CFS05b		Cycling to failure	1.52 mm/second
CFS06a	CFS06	Cycling to 1 % drift	1.52 mm/second
CFS06b		26 m 40 s burn	Step to 1900 kW
CFS06c		Continue cycling until failure	1.52 mm/second

Table 2

Amplitude of primary cycles and step durations.

Pattern	Step	Number of cycles	Amplitude of primary cycle, %	Amplitude, mm	Drift, %	Step duration, m
1	1	6	5	1.9	0.07	10
2	2	7	7.5	2.9	0.10	12
	3	7	10	3.8	0.14	12
3	4	4	20	7.6	0.28	7
	5	4	30	11.4	0.42	7
4	6	3	40	15.2	0.56	5
	7	3	70	26.7	0.97	5
	8	3	100	38.1	1.39	5
	9	3	130	49.5	1.81	5
	10	3	170	64.8	2.36	5
	11	3	200	76.2	2.78	5