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PHYSICAL DAMAGE EVOLUTION DURING EARTHQUAKE AND POST-EARTHQUAKE FIRE TESTING OF A MID-RISE COLD-FORMED STEEL FRAMED BUILDING

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ABSTRACT

A unique research collaboration between academia, government, and industry was formed to contribute to understanding the earthquake and post-earthquake fire behavior of mid-rise cold-formed steel wall-braced buildings. Led by the University of California, San Diego (UCSD), with partnerships from Worcester Polytechnic Institute (WPI), government and state agencies, and more than 15 industry sponsors, the centerpiece of this project involved full-scale earthquake and fire testing of a full-scale six-story CFS wall braced building. The test building was constructed on the world's largest outdoor shake table – the Large High Performance Outdoor Shake Table (LHPOST) at UCSD. Within a three-week test program, the building was subjected to seven earthquake tests of increasing motion intensity. Subsequently, live fire tests were conducted on the earthquake-damaged building at two select floors. Finally, for the first time, the test building was subjected two post-fire earthquake tests, including a low-amplitude 'aftershock' and an extreme near-fault motion. In addition, low-amplitude vibration data were collected during construction and testing phases to support identification of the dynamic state of the building system. This paper offers an overview of the earthquake and fire test program and summarizes key experimental results (i.e., building response, physical damage features) during each of the earthquake, post-earthquake fire, and post-fire earthquake test phases.

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A unique research collaboration between academia, government, and industry was formed to contribute to understanding the earthquake and post-earthquake fire behavior of mid-rise cold-formed steel wall-braced buildings. Led by the University of California, San Diego (UCSD), with partnerships from Worcester Polytechnic Institute (WPI), government and state agencies, and more than 15 industry sponsors, the centerpiece of this project involved full-scale earthquake and fire testing of a full-scale six-story CFS wall braced building. The test building was constructed on the world's largest outdoor shake table – the Large High Performance Outdoor Shake Table (LHPOST) at UCSD. Within a three-week test program, the building was subjected to seven earthquake tests of increasing motion intensity. Subsequently, live fire tests were conducted on the earthquake-damaged building at two select floors. Finally, for the first time, the test building was subjected two post-fire earthquake tests, including a low-amplitude 'aftershock' and an extreme near-fault motion. In addition, low-amplitude vibration data were collected during construction and testing phases to support identification of the dynamic state of the building system. This paper offers an overview of the earthquake and fire test program and summarizes key experimental results (i.e., building response, physical damage features) during each of the earthquake, post-earthquake fire, and post-fire earthquake test phases.

Introduction

Growth in the use of cold-formed steel (CFS) framed construction has been substantial in recent years, perhaps most notably in high seismic regions in the western United States. Structural systems of this kind consist of repetitively framed light-gauge steel members (e.g., studs, tracks, joists) attached with sheathing materials (e.g., wood, sheet steel) to form wall-braced component. CFS-framed structures can offer lower installation and maintenance costs than other structural types, particularly when erected with prefabricated assemblies. They are also durable, formed of an inherently ductile material of consistent behavior, lightweight, and manufactured from recycled materials. Compared to other lightweight framing solutions, CFS is non-combustible, an important basic characteristic to minimize fire spread. While these lightweight systems provide the potential to support the need for resilient and sustainable housing, the state of understanding

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regarding their structural behavior in response to extreme events, in particular earthquakes and ensuing hazards, remains relatively limited.

In the past few decades, a number of experimental investigations have been devoted to advancing understanding regarding the seismic response of CFS-framed shear wall components. The work conducted by Serrette et al. [1] represents one of the first efforts of its kind in North America to study the seismic response of CFS-framed shear walls. This effort largely formed the initial basis for codified design of CFS systems (e.g., [2,3]). Rogers and colleagues extended their research to investigate CFS wall behavior with varied sheathing materials or framing details [4]. Their experimental studies included pseudo-static tests of CFS-framed steel strap shear walls [5] and steel-sheet shear walls [6], as well as pseudo-dynamic tests of two-story steel-sheet shear wall assemblies [7]. In addition, recent experimental studies involved testing of CFS shear walls sheathed with sheet steel [8] or oriented strand board (OSB) panels [9]. In contrast, there is a paucity of data regarding the seismic response of CFS-framed buildings configured in their system-level arrangement (whole building tests). The shake table testing of a low-rise (two-story) CFS-framed building within the NSF-supported NEES-CFS program represents the first and only system-level CFS-framed building test in the North America [10,11].

To address the need for understanding the earthquake and post-earthquake fire behavior of mid-rise CFS-framed buildings, a unique multidisciplinary test project was conducted on the LHPOST test facility at UCSD between April and July 2016. Central to this research is the system-level earthquake and live fire testing of a full-scale six-story CFS wall braced building [12-14]. In a three-week test program, the building was subjected to seven earthquake tests of increasing motion intensity. Earthquake motions were scaled to impose service, design, and maximum credible earthquake (MCE) demands onto the test building. Subsequently, live fire tests were conducted on the earthquake-damaged building at two select floors. Finally, the test building was subjected to two post-fire earthquake tests, including a low-amplitude 'aftershock' and an extreme near-fault target MCE-scaled intensity motion. This paper provides an overview of the earthquake and fire test program as well as summarizes the key experimental results (i.e., building global response, physical damage features) during each of the earthquake, post-earthquake fire, and post-fire earthquake test phases.

Building Design and Construction

Building Design

A full-scale cold-formed steel (CFS) test building was designed and erected on the large, high-performance outdoor shake table at UC San Diego (NHERI@UC San Diego) [12-14]. For the purposes of design, this six-story CFS framed test building (Fig. 1a) was assumed to be located in a high seismic region near downtown Los Angeles, with its design basis complying with current code provisions within ASCE 7-10 [15] (ASCE, 2010), AISI S100 (AISI, 2012), and AISI S213 (AISI, 2007). For simplicity, a uniform plan with dimension of 10.4 m × 7.3 m (34 ft × 24 ft) at each floor was adopted, allowing the specimen to occupy almost the entire 12.2 m × 7.6 m (40 ft × 25 ft) shake table footprint. The total height of the building was 19.2 m above the shake table platen (a floor-to-floor height of 3.1 m (10 ft) for all stories and a 1.2 m-tall (4 ft tall) parapet on the roof perimeter). The effective seismic design weight of the test building was assumed as 1420 kN (320 kips). According to ASCE 7-10 [15], the CFS wall braced building was designed with a response modification factor R of 6.5, an overstrength factor Ω of 3.0, and a deflection amplification factor C_d of 4.0. The code-based fundamental period of the test building T was determined as 0.43 sec considering a total building height of 18.3 m (60 ft) excluding the parapets. The base shear coefficient C_s of the test building was consequently determined as 0.236 and resulted in an effective seismic design base shear Vb of 334 kN (75 kips). It is noted that the weight of the building was directly determined using measurements recorded during the nine earthquake tests. From these measurements, the average building weight, including its nonstructural components was 1160 kN (260 kips). While this was ~260 kN (60 kips) lower than that used for the design, this was anticipated and accounts for the reduction of live loads (reduction factor of ~0.6) in the event of an earthquake. The estimated maximum inelastic story drift of the building was ~1.0% (with a deflection amplification factor C_d of 4.0, which was lower than the allowable story drift of 2.0% as prescribed in ASCE 7-10 [15].

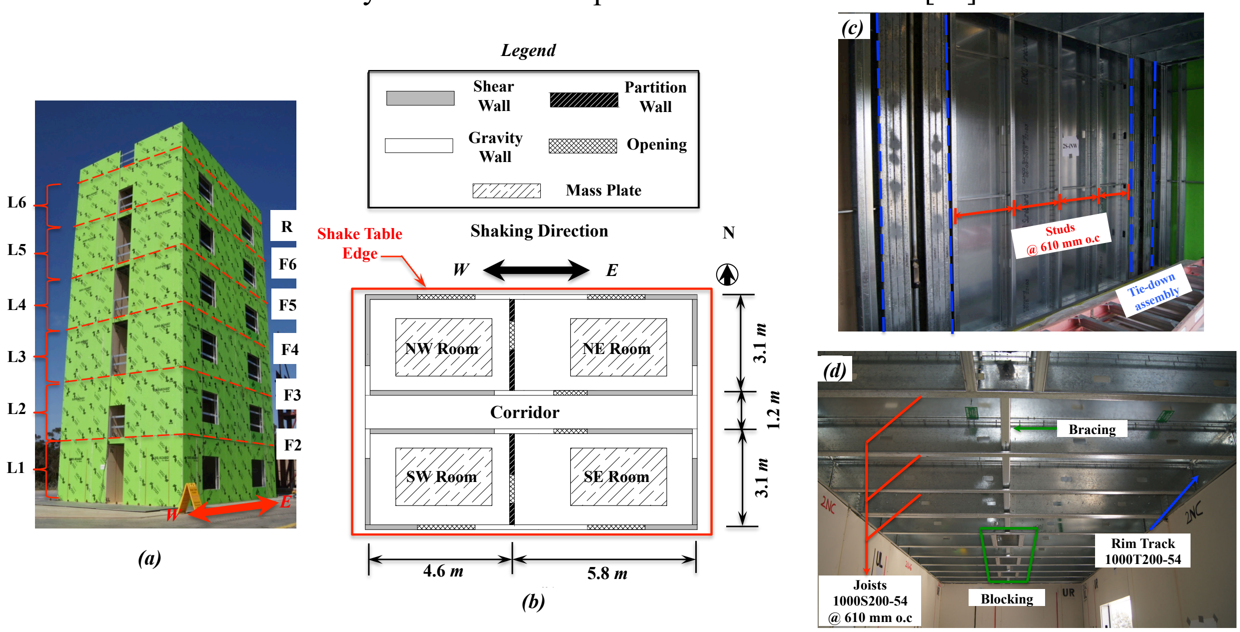

Figure 1. (a) Isometric view of test building, (b) building plan layout (typical of floor 2 to 6), (c) corridor shear wall steel framing, and (d) floor diaphragm steel framing.

Structural Component Details

The test building was detailed to carry lateral seismic loading using prefabricated repetitively framed CFS floors and walls with shear load resistance provided via steel sheathing. As illustrated in the plan layout of Fig. 1b, two longitudinal shear walls were placed along each (east and west) end of the corridor, with an associated wall length of 4.0 m (13 ft) for the walls at the west end and 3.3 m (11 ft). In addition, short shear walls with a length of ~1.6 m (5'-4") in the longitudinal direction and ~2.1 m (7 ft) in the transverse direction were placed at the four corners of the building. The total shear wall length per floor was 21.3 m (70 ft) in the longitudinal (shaking) direction and 8.6 m (28 ft) in the transverse direction. It is noted that the corridor shear walls were designed as the primary lateral load resisting elements in the direction of shaking, while the corner shear walls were assumed to resist transverse and torsion loads during the tests.

The shear walls were framed using standard framing members (e.g., studs, tracks as shown in Fig. 1c). Sheathing materials utilized load-resisting structural panels on the exterior (or corridor) side and 16 mm (5/8") thick regular gypsum boards on the room side. The structural panels were fabricated using 16 mm (5/8") thick gypsum boards (or) bonded with a layer of 0.686 mm (0.027") thick (22 ga.) sheet steel to provide shear resistance to the shear wall assemblies. For the corridor shear walls, vertical studs utilized 600S200-68 at 610 mm (24") o.c at the first level and 600S200-54 at 610 mm (24") o.c at all remaining levels. The (top and bottom) tracks were consistently constructed using 600T200-54, with the exception of the first level bottom tracks that used 600T200-97. In addition, the chord studs (in a double stud pattern) at the edge of the door and opening windows were constructed using 600S200-68. It is also important to note that corridor and corner shear walls contained a pair of tie-down subassemblies (consisting of tie rods and compression posts) as part of the building tie-down system.

The floor and roof diaphragms of the test building were all constructed using prefabricated CFS-framed panel systems, however individual edges of the diaphragm were attached using a ledger framing system (Fig. 1d). Namely, they were connected to the vertical structural system by attaching the diaphragm joists to the flange of the wall studs via a combination of rim track and clip angle solution. The floor sheathing consisted of fiber reinforced cement boards bonded with a layer of 0.838 mm (0.033”) thick (20 ga.) sheet steel. In addition, the underside of floor 3 and roof was sheathed with 16 mm (5/8”) thick regular gypsum panels to provide a compartmentalized fire-testing environment.

Construction

Construction of the test building commenced in April 15, 2016 with the shake table platen tie-down installation. Subsequently, the first-story wall system was fabricated in-situ for a total of four days. Following completion of the first-story wall system, the building construction significantly expedited as a result of the highly efficient panelized construction. Construction of the upper levels progressed at a rate of one level per day. The erection of the building skeleton was completed on April 27, 2016 (total of nine construction days). Interior construction commenced immediately following the completion of the building erection. Activities related to interior construction included installation of: 1) interior gypsum panels (structural walls, nonstructural walls, and ceiling), 2) interior partition walls, 3) door systems, and 4) appliance (on the first and sixth floors only). These activities spanned about an entire month and the interior installation was completed at the beginning of June 2016. Interested readers are referred to the video links [1,2] of the construction and demolition time lapses.

Test Protocol

The three-week test program consisted of a sequence of nine earthquake tests and six fire tests between June 13 and July 1, 2016. During the first week (pre-fire test phase), the building was subjected to seven earthquakes with increasing input motion intensity in three test days (June 13, 15, and 17, 2016). Subsequently, live fire tests were conducted on the earthquake-damaged building at the second and sixth levels of the building across a period of three consecutive days (June 27–29, 2016). The test program concluded with two post-fire earthquake tests on the final test day at the end of the third week (July 1, 2016). To complement the earthquake and fire test sequence, low-amplitude vibration tests in the form of white noise and tire (shock) tests as well as ambient vibration tests were conducted throughout the construction and test phase. It is noted that all of the earthquake and white noise test motions were applied in the east-west direction using the single-axis shake table, whose axis coincided with the longitudinal axis of the building.

Earthquake Test Protocol

The acceleration and displacement time histories of the achieved input earthquake motions and

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[1] Construction time lapse available at https://www.youtube.com/watch?v=IFq7Nv_020c.
[2] Demolition time lapse available at https://www.youtube.com/watch?v=ElOiksCJUKM.

the 5% damped elastic response spectra are shown in Fig. 2. It is noted that all the input motions were selected from shallow earthquake events in California (i.e., the 1994 M_w=6.7 Northridge earthquake, the 1992 M_w=7.0 Cape Mendocino earthquake), with the exception of EQ3 that represented a large-magnitude subduction event (the 2010 Mw=8.8 Maule earthquake in Chile). Consequently, the strong motion duration of EQ3 was substantially longer than those of the remaining motions in the earthquake test sequence. It is also notable that the first seven earthquake motions (pre-fire test sequence) were applied at increasing intensity to progressively damage the building, as the peak input accelerations of the motions increased from 0.15 g to 0.9 g and the spectral accelerations at the fundamental period of the test building $S_a(T_1,5\%)$ increased from 0.3 g to 2.0 g. The last two test motions (post-fire test sequence) were intended to represent a service-level aftershock event (EQ8 – a replicate of test motion EQ1) and a near-fault extreme event (EQ9) with a peak input acceleration above 1.0 g.

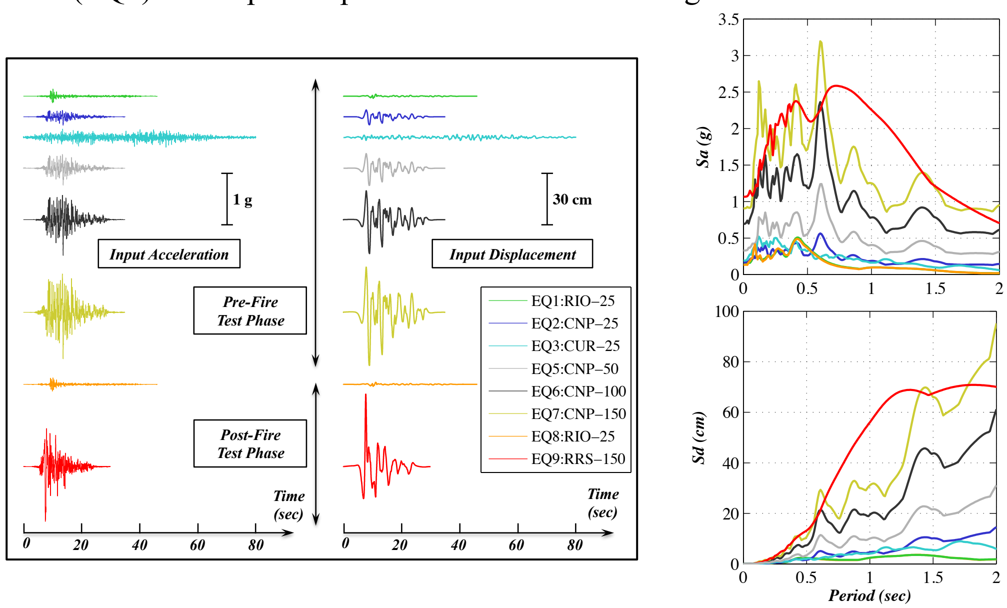


Figure 2. Achieved input motions: acceleration and displacement time histories of the (left); response spectra (ξ = 5%): pseudo-acceleration spectra (top-right), and displacement spectra (bottom-right).

Fire Test Protocol

Following the completion of pre-fire test sequence (EQ1-EQ7), the earthquake-damage building was subjected to six compartment fire tests on three consecutive days, including four tests at level 2 and two tests level 6 (Fig. 3). The fire test compartments were constructed with architectural features representative of a 60-minute fire resistance rating construction details. To ensure the attainment of post-flashover condition at each compartment, a set of six stainless steel burner pans, each filled with 12 liters of n-heptane fuel, was used to create the fire loads (with a heat release rate of 2.16 megawatt) for each test. It is noted that the compartment ventilation characteristics and the extant of damage to the interior gypsum boards induced by prior earthquake test sequence were the major variables considered in the fire test sequence.

Building Response to Earthquake Motions

To facilitate discussion of the test building behavior of the during the pre- and post-fire earthquake tests, the system-level building response in the direction of direction (along the longitudinal axis of the building), including peak floor accelerations (PFAs), peak inter-story drift ratios (PIDRs), peak roof drift ratios (PRDRs), and residual roof drift ratios (RDR_{res}), are summarized in Table 1. While relatively small in amplitude during the service level earthquakes (PIDRs < 0.1%), the PIDRs achieved about 1% during the design event (EQ6) and exceeded 1.5% during the MCE event (EQ7). As shown in Fig. 4, the story shear vs interstory drift (IDR) response indicates that the test building remained quasi-linear during the service level test (EQ2), but became highly nonlinear as the drift demands reached about 1.0% during the design event (EQ6) and exceeded 1.5% during the MCE event (EQ7). The shear wall behavior observed in the pre-fire test sequence correlates well with those of the wall-component cyclic loading tests [16]. During the post-fire test phase, the final near-fault extreme event (EQ9) induced excessively large drift demands at level 2 of the test building (transient PIDR > 12% and RDR_{res} > 1%), resulting in extremely severe structural damage to the wall systems at level 2.

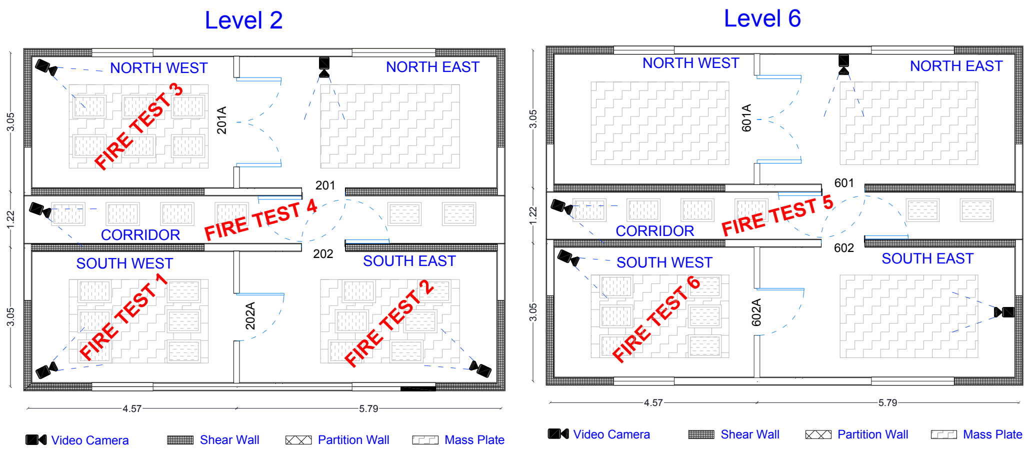


Figure 3. Locations of the fire compartment tests as shown in the building plan layout (level 2 and 6).

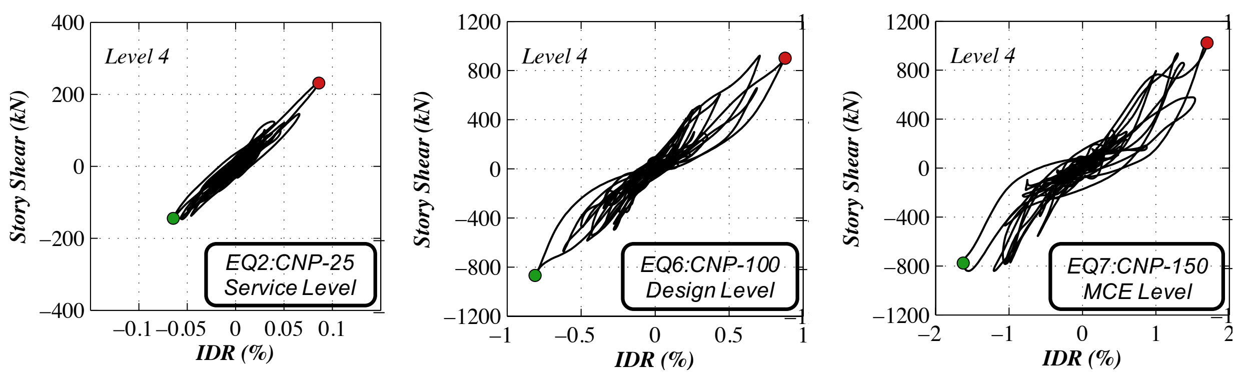


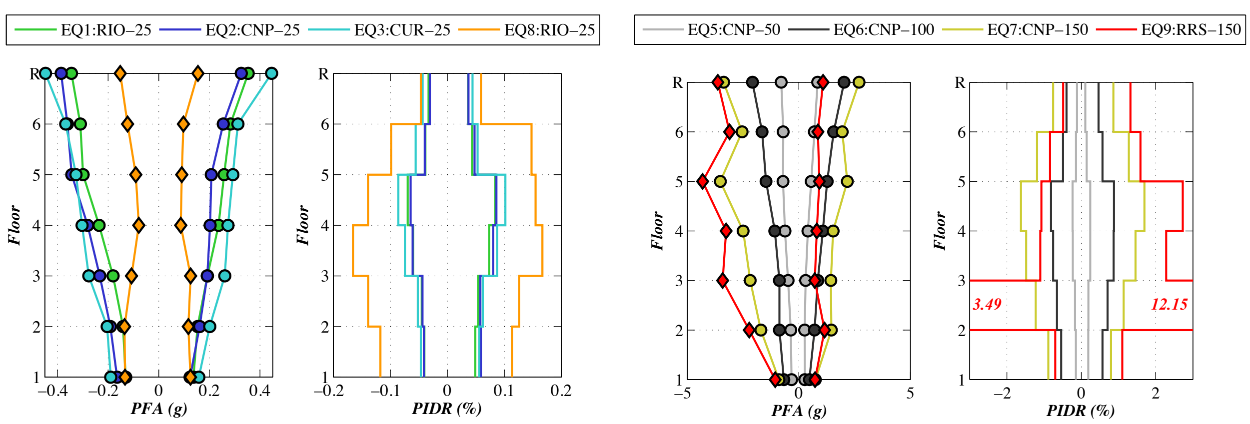
Figure 4. Story shear vs interstory drift response at level 4 during three pre-fire earthquake tests.

To demonstrate the effect of prior earthquake and fire damage on the behavior of the test building, the PFA and PIDR responses during the service level pre- and post-fire tests (i.e., tests EQ1—EQ3 and EQ8) are compared in Fig. 5a. Although the seismic demands on the building were relatively low during these service-level earthquakes, the building observed apparent acceleration attenuation effects and larger interstory drifts during the post-fire test (EQ8). This is due to the fact that building sustained substantial stiffness deterioration due to the damage accumulated during the prior earthquake and fire tests. Consequently, the PIDRs achieved during the post-fire service level test (EQ8) were about twice as large as those attained during the post-fire service level test sequence (EQ1—EQ3). In contrast, Fig. 5b compares the PFA and PIDR responses during the *above-service-level* pre- and post-fire tests (i.e., tests EQ5—EQ7 and EQ9). As the motion intensity increased, the largest PIDR occurred at the mid-height of building (level 3 and 4) throughout the pre-fire earthquake test sequence. In addition, the PFA increased almost monotonically up the height of the building during the pre-fire earthquake test sequence, indicating a fundamental-mode dominant structural response in these tests. The final earthquake test (near-fault MCE event EQ9) subjected the building to extremely large drift demands (PIDR exceeded 12% at level 2) and resulted in a near-collapse condition of the test specimen. This is partially attributed to the fire-induced damage to the gypsum sheathing at level 2, which significantly reduced the shear capacity of the shear walls [16], encouraging formation of a soft-story mechanism during the final near-fault earthquake (EQ9) at this level.

Table 1. Summary of the test sequence and the associated peak building responses

Test Date	*Test Motion*	*Performance Target*	*PFA (g) (Floor #)*	*PIDR (%) (Level #)*	*PRDR (%)*	*RDR_{res} (%)*
Day 1 (June 13, 2016)	EQ1:RIO-25	Service level	0.35 (R)	0.08 (L4)	0.05	0.0
	EQ2:CNP-25		0.38 (R)	0.09 (L4)	0.07	0.0
	EQ3:CUR-25		0.45 (R)	0.10 (L4)	0.08	0.0
Day 2 (June 13, 2016)	EQ4:CNP-25		0.43 (R)	0.10 (L4)	0.09	0.0
	EQ5:CNP-50	50% Design	0.85 (R)	0.24 (L3)	0.19	0.0
	EQ6:CNP-100	Design	2.07 (R)	0.89 (L4)	0.70	0.0
Day 3 (June 13, 2016)	EQ7:CNP-150	MCE	3.77 (F5)	1.70 (L4)	1.49	0.1
Fire Test Sequence (June 27–29, 2016)						
Day 4 (June 13, 2016)	EQ8:RIO-25	Service level	0.16 (R)	0.17 (L3)	0.12	0.0
	EQ9:RRS-150	MCE	4.43 (F5)	12.15 (L2)	2.84	1.2

Notes: PFA= peak floor acceleration; PIDR = peak interstory drift ratio; PRDR = peak roof drift ratio; RDR_{res} = residual roof drift ratio; MCE = maximum considered earthquake.



(a) (b)

Figure 5. Building peak responses during (a) *service-level* tests and (b) *above-service-level* tests. Peak floor accelerations (left) and peak interstory drift ratios (right).

Physical Observations

Pre-fire Earthquake Tests

Due to the low seismic demands during the service level tests at all levels of the test building (PFA < 0.5 g, PIDR < 0.1%), the interior sheathing sustained only a few instances of minor damage in the form of incipient screw withdrawal and localized gypsum crushing at bulging at level 3 and 4, while no visible damage to interior sheathing occurred at all other levels. The extant of sheathing damage, however, was considered inadequate to classify the walls into an appreciable damage state. Damage to the interior sheathing however did continue to develop as the seismic drift demand increased during the 50% design event (EQ5) and design event (EQ6). Screw withdrawal and gypsum crushing of the corridor shear walls and gravity walls became more pervasive at all levels (except level 6), in particular at the corridor shear wall–gravity wall boundaries as well as the window and door openings (Fig. 6a). In contrast, damage to the corner shear walls remained minor, as they occurred only in the form of localized gypsum crushing at the corner and formation of gaps between gypsum panels. Following the completion of the pre-fire earthquake test sequence, the gypsum panels of the northwest compartment at level 4 (the level with the largest drift demands during the pre-fire test sequence) were removed to allow for inspection of the shear wall framing and sheathing steel. As shown in Fig. 6b, localized buckling of the sheathing steel was detected at the top of corridor shear wall, while the framing studs and tracks did not sustained apparent damage. In contrast, the wall framing and sheathing steel of the corner shear wall in the same compartment sustained no apparent damage. In addition, loosening of the bolts at the floor bearing connections was detected, resulting in very loose tie rods at the end of the pre-fire sequence.

(a) (b)

Figure 6. Shear wall damage during the pre-fire earthquake test sequence: (a) sheathing damage following the design level test (EQ6), and (b) corridor shear wall framing following the MCE test (EQ7).

Fire Tests

Following the pre-fire earthquake tests, live fire tests were conducted at the six pre-determined compartments at level 2 and 6. The elevated temperature caused dehydration and shrinkage of the wall and ceiling gypsum boards and fiber-reinforced cement boards on the floor. The damage to the shear wall and floor sheathing resulted in significant strength and rigidity loss of these structural components. As shown in Fig. 7, the fire-induced structural damage occurred in the form of: (1) partial detachment of gypsum ceilings, and (2) significant deflections (about 1.5 cm) of the floor diaphragm at the second floor as a result of deteriorated cement boards on top of the sheet steel. In addition, the building egress was compromised following the fire tests as a result of loss of functionality of the doors due to the fire-induced damage to the door components.

Figure 7. Flame and smoke extension during the corridor fire test (left) and fire-induced damage: partially detached ceiling gypsum board (top middle), shrinkage and cracking of the cement board on the floor (bottom middle), excessive floor deflection (top right), and melted door closer (top bottom).

Post-fire Earthquake Tests

Due to the extremely large drift demands at level 2 of the test building (transient PIDR > 12% and residual IDR ~ 6%) during the post-fire extreme event EQ9, the building developed a soft-story mechanism at the completion of the test program (Fig. 8a). The excessive interstory drift demands are associated with severe damage to the structural components of level 2 and, importantly, revealed the ultimate damage mechanism of the lateral loading resisting system. The test building, however, resisted collapse largely due to redistribution of loads and the framing action of the continuous rod tie-down system. Evident in the post-event physical inspection was the near complete tearing of a large extent of the structural sheathing, partial or full-delamination of sheathing face gypsum and associated detachment of gypsum on the opposing face (Fig. 8b). Consequently, structural components of the wall (framing members) suffered extensive global and local buckling.

(a) (b)

Figure 8. Building damage following the extreme event test (EQ9): (a) north elevation, and (b) interior views (corridor and SE room, left and right, respectively, level 2).

Concluding Remarks

To advance knowledge regarding the multi-hazard performance of mid-rise CFS construction, a full-scale six-story cold-formed steel building was constructed and tested on the UCSD LHPOST test facility between April and July 2016. During this program, the test building was first subjected to a suite of seven earthquake motions with progressively increasing motion intensity, followed by live fire tests in six strategically selected rooms at level 2 and 6 of the test building. Finally, for the first time, the test building was subjected two post-fire earthquake tests, including a low-amplitude 'aftershock' and an extreme near-fault target MCE-scaled motion. Key findings regarding the three distinct test phases include the following:

- *Pre-Fire Earthquake Tests:* The test building suffered minimal damage during the service level earthquake tests and remained largely in the quasi-linear range, with very low drift demands imposed on the specimen (< 0.2%). During the design level earthquake test, the corridor shear and gravity walls at level 3 and 4 suffered damage in the form of gypsum panel crushing and fastener withdrawal when the interstory drifts at these two levels reached about 1.0%. Damage continued to progress as the interstory drift exceeded 1.5% during the maximum considered earthquake (MCE) test, however observed damage to the building remained readily repairable, with the structural shear walls at the lower floors (those that could be inspected) developing their intended local steel sheathing buckling mechanism near attachment points along framing member perimeters. The building structural components performed satisfactorily throughout the pre-fire earthquake test sequence. The most significant damage to the structural system, as noted, occurred in the form of buckled sheet steel on the corridor shear walls composite panels.
- *Fire Tests:* Post-flashover conditions were achieved in all six compartment fire tests at the given ventilation conditions, with the corresponding maximum compartment temperatures ranging between 800 - 1100 °C. The elevated temperature caused significant degradation of interior fire rated gypsum boards on sheet steel and plain fire rated gypsum boards, leading to loss of structural strength. Loss of rigidity in floor sheathing due to degradation of cement board on top of the sheet steel caused significant floor deflections (about 1.5 cm). Thermal bowing of floor joists was observed after the test suggesting that there was a significant flow of heat from the floor system under consideration. The dehydrated and detached ceiling panel cause overhead hazards in the case of an aftershock event, and the extended flames through the building exterior openings also emphasize the high likelihood of travelling fire hazards.
- *Post-Fire Earthquake Tests:* The low-amplitude aftershock following the fire tests significantly attenuated seismic demands in the building as a result of the elongated period caused by the pre-fire earthquake sequence. In contrast, the extreme near-fault earthquake test (EQ9) developed a full soft story mechanism at level 2 and caused severe damage to the buildings structural system (complete loss of structural integrity of corridor and exterior longitudinal shear walls). It is highly laudable that the test building resisted collapse due to redistribution of loads and framing action of the building rod tie-down system.

Acknowledgments

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