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Overview of the CFS10 Program: Full-scale Shake Table (Earthquake and Fire) Testing of a 10-Story Cold-Formed Steel Building

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ABSTRACT

This paper is the first in a pair presented within the session: *Findings from the CFS10 Multi-Hazard Test Program*. The emphasis within this article is to, in brevity, describe the scope of a landmark full-scale 10-story cold-formed steel (CFS) framed building tested under multi-hazard (earthquake and fire) scenarios at the UC San Diego 6-DOF Large High-Performance Outdoor Shake Table (LHPOST6). Coined CFS10, this unique building specimen is designed beyond current code height limits, adopting advances in cold-formed steel shear wall detailing, varied construction modalities, and enriched with nonstructural components and systems. The landmark CFS10 building specimen was subjected to extreme multi-hazard (earthquake and fire) loading conditions.

This paper sets the framework for presentations to be shared at the Congress, while also aiding in ongoing documentation of findings from the program.

INTRODUCTION

In recent years, lightweight structures constructed of load bearing CFS-framing have demonstrated robust structural behavior during system-level shake table testing. Amongst the most notable are full-scale investigations of CFS-framed buildings by Peterman et al. (2016) and Hutchinson et al. (2021). In the former, a 2-story all CFS-framed building was tested at SUNY-Buffalo under increasing intensity multi-directional earthquake simulations (Fig. 1a); while the later involved a 6-story all CFS-framed building finished with gypsum sheathing tested at UC San Diego, under increasing intensity uni-directional earthquake simulations (Fig. 1b). Both programs embraced all-CFS gravity and lateral seismic resisting systems, with sheathed (oriented strand board for the former and steel sheet sheathing for the later) shear walls and ledger-framed CFS-joist framed diaphragms adopted. Robust seismic behavior was observed in each of these programs, with overstrength contributions from non-seismically designated components and architectural finishes evident.

Despite these unique datasets and complementary component experimental and numerical evidence, the use of CFS framed systems for taller buildings [$>19.8\text{m}$ ($>65\text{ft}$), or 6-stories] remains prohibited due to code restrictions (e.g., ASCE 7-22, 2022). To contribute to understanding the performance of tall CFS-constructed buildings under multi-hazard, a 10-story full-scale CFS-framed building was recently constructed and tested under increasing intensity multi-directional earthquake shaking and post-earthquake live fire testing. Coined CFS10 (cfs10.ucsd.edu), this building specimen was constructed and tested atop the upgraded 6-DOF Large High-Performance Outdoor Shake Table (LHPOST6) at the NHERI@UC San Diego Experimental Facility (esec.ucsd.edu), see Fig. 2. This multi-university-industry collaborative contribution intends to advance knowledge of the performance of tall CFS-framed buildings by serving as a benchmark for validating and advancing modeling tools, design codes and practices.

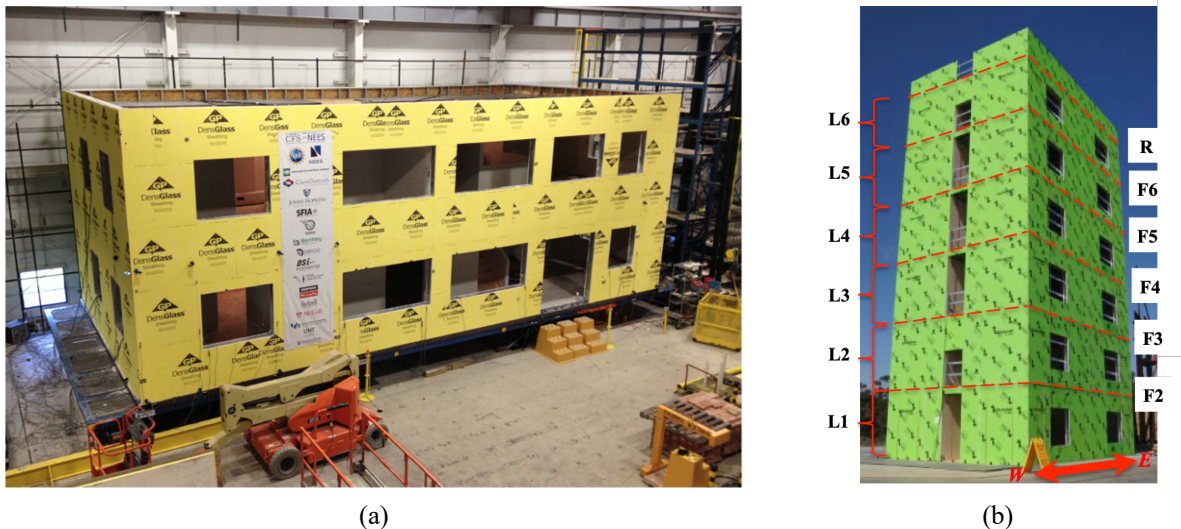


Figure 1. CFS-NEES (2-story, Peterman et al., 2016) and (b) CFS-HUD (6-story, Hutchinson et al., 2021)

EXPERIMENTAL PROGRAM

Building Design & Construction

The landmark CFS10 building specimen was designed above-code height limits with 10 stories and a total height of 30.5 m (to the roof level) and a floor plan measuring 6.86 m $\times$ 11.58 m, see Fig. 2. The primary goal of this test program was to evaluate the seismic performance of a tall CFS building with steel-sheet-sheathed shear walls exceeding the ASCE 7-22 (2022) height limit of 19.8 m, or 6 stories. Characterized as a slice of a full building archetype, the test specimen's footprint resulted in three wall lines in the Y-direction with two steel sheet-sheathed shear walls along each wall line, and two wall lines in the X-direction, also with two steel sheet-sheathed shear walls, detailed to provide lateral seismic resistance, see Fig. 3. This layout resulted in two room-space compartments, and two “balconies”, small portions of the adjacent rooms in the archetype. In addition to the load-bearing CFS studs and CFS-steel sheet-sheathed shear walls, the test building was detailed with gypsum-board-sheathed gravity walls, an exterior face with a stucco-based finish exterior insulation and finishing system (EIFS) on its north side (the south side is a corridor in the full building), and a ledger-framed CFS floor framing system with a dry cementitious panel diaphragm. Additional details regarding the structural design are summarized in Singh et al. (2024) and Hutchinson et al. (2026).

The building was outfitted with various nonstructural components and systems, including a seismic drift resilient stair system (Sorosh et al., 2025), a fire riser and sprinkler system, suspended ceiling (Singh et al., 2026a), a gas piping system, windows, and rooftop equipment (Zhang X. et al, 2026), see Fig. 4. A holistic description of the various nonstructural components and systems integrated within the specimen is provided in the companion paper by Singh et al. (2026b). Construction, instrumentation, and initial findings regarding the seismic performance of the building are provided in Hutchinson et al. (2026).

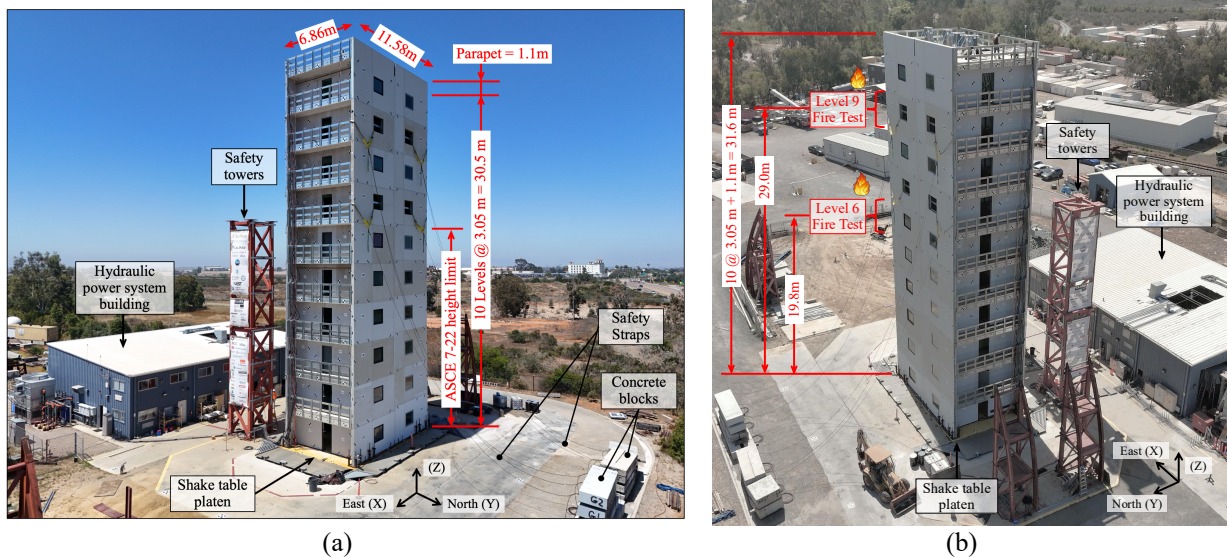


Figure 2. Annotated photograph of the CFS10 specimen: (a) Northeast view (annotated features for seismic test phase); and (b) Northwest view (annotated fire test floors).

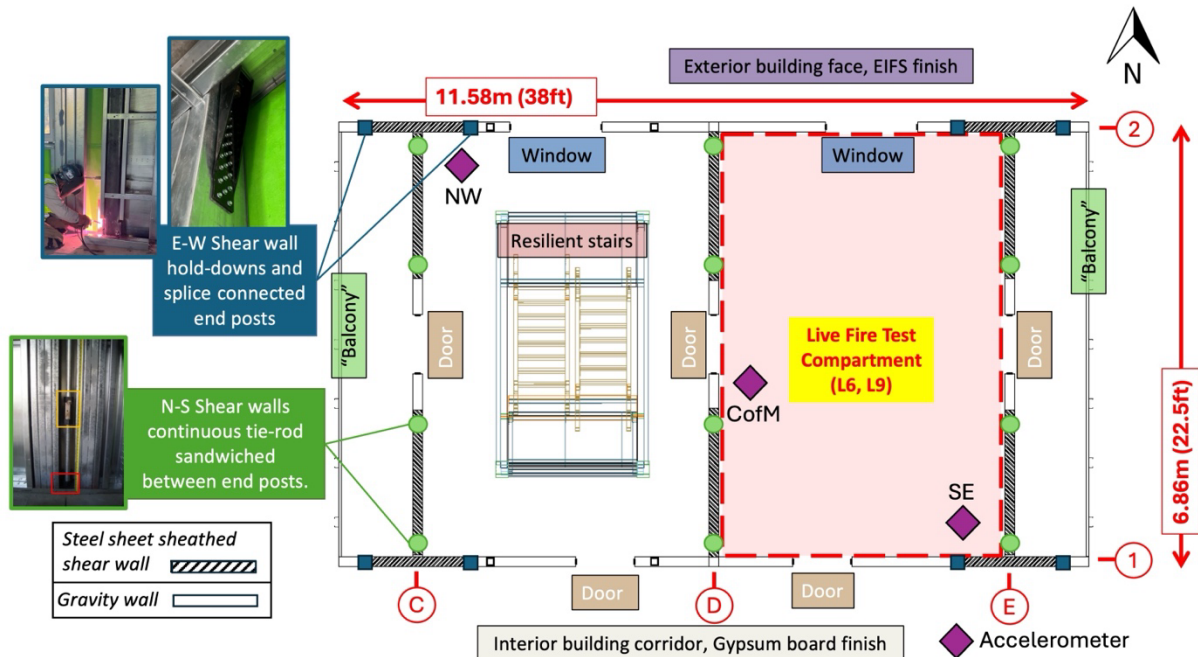


Figure 3. Annotated plan view and photographs of various shear wall detailing features.



Figure 4. Nonstructural components and systems integrated within the test building: (a) suspended ceilings, (b) flexible and rigid gas piping, (c) fire sprinkler system, (d) swing doors, (e) roll-up door, (f) interior and exterior finishes, (g) windows, (h) roof-mounted mechanical equipment, and (i) prefabricated modular steel stairs.

Earthquake Test Protocol

A total of 18 multi-directional earthquake ground motions, scaled to increasing hazard levels, were applied to the specimen, see Table 1. These included 15 service-level earthquakes (SLE), one design-level earthquake (DE), and two risk-targeted maximum considered earthquakes (MCE_R). Two distinct SLE suites (MID1-10 and MID11-15) were designed considering seismological settings characterized by short duration far-field and long duration events, denoted *SLE1* and *SLE2*, respectively. As a result, while one carry-through increasing intensity motion (1989 Loma Prieta, California – Gilroy Array #3) was utilized across all three hazard levels, a wide range of amplitudes, durations, and directionalities were ultimately imposed on the specimen, see Table 1. Additional details regarding motion selection criteria for this program, including the source selection and scaling procedures adopted, are discussed in Zhang, J. et al. (2026).

Table 1. Earthquake test protocol.

Hazard level	MID	Earthquake event				Input DOF	Components		D_{5-95} [sec]	PIA _x [g]	PIA _y [g]	PIA _z [g]
		Name, location	Year	M _w	Station		X	Y				
SLE1	1*	Loma Prieta, California	1989	6.9	Gilroy Array #3	XYZ	90°	0°	11.4	0.126	0.122	0.082
	2					XYZ	90°	0°	11.4	0.118	0.114	0.096
	3					X	90°		11.4	0.133	0.041	0.025
	4					Y		0°	6.4	0.042	0.109	0.024
	5	Northridge, California	1994	6.7	LA City Terrace	XYZ	90°	180°	11.9	0.125	0.121	0.081
	6					XYZ	180°	90°	11.9	0.184	0.094	0.084
	7					X	180°		11.9	0.168	0.027	0.023
	8					Y		90°	11.6	0.026	0.093	0.021
	9	Cape Mendocino, California	1992	7.0	Rio Dell Overpass	XYZ	270°	360°	15.3	0.084	0.109	0.099
	10					XYZ	360°	270°	15.3	0.078	0.122	0.151
SLE2	11**	Kahramanmaraş, Türkiye	2023	7.8	4611	XYZ	E	N	43.4	0.095	0.075	0.050
	12					XYZ	E	N	43.4	0.117	0.110	0.064
	13	Elbistan, Türkiye	2023	7.5	4611	XYZ	E	N	35.1	0.100	0.061	0.052
	14	Maule, Chile	2010	8.8	Curico Hospital	XY	EW	NS	45.2	0.097	0.091	0.033
	15	Kahramanmaraş, Türkiye	2023	7.8	4611	XYZ	E	N	43.4	0.119	0.089	0.063
DE	16	Loma Prieta, California	1989	6.9	Gilroy Array #3	XY	90°	0°	11.4	0.454	0.388	0.014
MCE_R	17	Loma Prieta, California	1989	6.9	Gilroy Array #3	XY	90°	0°	11.4	0.723	0.619	0.016
	18	Cape Mendocino, California	1992	7.0	Rio Dell Overpass	XY	360°	270°	15.3	0.782	0.336	0.025

SLE = service level earthquake, DE = design earthquake, and MCE_R = risk-targeted maximum considered earthquake.

PIA = peak input acceleration (achieved) using shake table platen center of mass accelerometers

D_{5-95} = significant duration of strong shaking computed as the time interval between 5% and 95% Arias Intensity per original record (max component)

*MID1 was conducted with the stairs in a locked configuration

**MID11 target to 75% SLE

Distribution of Peak Seismic Demands

Acceleration and interstory drift response as a function of height in a building are important indicators of the distribution of seismic demand during earthquake shaking. In addition to guiding the structural design, these demands are important to the design of nonstructural components and systems. The later correlations are synthesized in the companion paper of Singh et al. (2026b). In this regard, select results are presented herein to lend context to their distribution within the building. Figs. 5 and 6 present the floor acceleration response histories and the peak floor accelerations (PFA) distributed with height, in the X- and Y- directions, respectively. Results are presented for the target scaled MCE_R test considering the carry-through motion (1989 Loma Prieta, California – Gilroy Array #3). Floor acceleration response histories demonstrate the gradual filtering of the transient response with increasing height, and subsequent amplification of accelerations. The largest PFA are consistently observed at the roof of the building in each

direction of response, with a maximum PFA of 1.68g, measured at the center of mass in the X-direction, where the resulting amplification factor (PFA/PIA) is 2.75. The largest amplification factor during these tests is 3.3, measured at the roof southeast corner of the building, also in the X-direction of response. Higher mode effects are evident in the PFA profiles of each direction, with non-uniform PFA/PIA ratios particularly evident at the upper levels of the building.

Interstory drift ratios (IDR) are equally important demands of interest in building seismic response. Figure 7 presents the computed peak IDR (PIDR) profiles during the target scaled DE and MCE_R tests considering the carry-through motion from the 1989 Loma Prieta, California earthquake (Gilroy Array #3 record). Floor drift histories are computed by double integrating unidirectional accelerometers located at the northwest (NW), southeast (SE), and center of mass (CoM) locations of the building's floor plan. Subsequently, interstory drift values are computed by subtracting sequential floor-floor drift histories at similar locations. IDRs are then computed by normalizing by the floor-floor height and filtered with a Butterworth filter to remove numerically produced artifacts, with PIDRs in the positive and negative directions extracted. Results indicate an approximate doubling of PIDR demands when comparing the DE to MCE_R test results, with larger PIDRs observed in the X-direction of excitation. The largest PIDRs in the X-direction are approximately 1%, concentrated at Level 8, while the largest PIDRs in the Y-direction are approximately 0.7%, concentrated at Level 4. These were lower than anticipated ($<1.5\%$ PIDRs at MCE_R).

Evolution of Modal Properties

To capture the evolution of dynamic characteristics of the building, low-amplitude uni-directional white noise (WN) tests in each orthogonal direction were imposed following each earthquake test. Data from 63 acceleration measurements, distributed across all floors at NW, SE, and CoM locations were used to determine the evolution of the building's modal characteristics. Specifically, the vibration periods of the first three modes corresponding to the first flexural-torsional mode in the longitudinal direction (1-L-To), the flexural-dominant mode in the transverse direction (2-T), and the torsional mode (3-To) were identified using the Stochastic Subspace Identification (SSID) technique (Peeters and De Roeck, 2001). Table 2 and Fig. 8 summarize the evolution of the building's vibration periods across eight distinct structural states. In this analysis, to estimate the vibration periods corresponding to the flexural-torsional mode in the X-direction and the torsional mode, data from white noise tests conducted in the X-direction were used. Similarly, to estimate the period of the flexural-dominant mode in the Y-direction, data from white noise tests in the Y-direction were used. An elongation of the building's vibration periods was observed as testing progressed from smaller-level earthquake tests (SLE motions) to relatively larger ones (MCE_R motions). Based on these results, it was found that unlocking the stairs (an initial test with the mid-story stair connections of Fig. 4i locked) increased the vibration periods of the first three modes by less than 1% (Sorosh et al., 2026). A maximum cumulative increase of 3% and 7% in vibration period was observed after the *SLE1* and *SLE2* motions, respectively. Furthermore, the DE motion resulted in a cumulative increase of approximately 15% in the building's period compared to the virgin state of the building after the stairs were unlocked (S0). After the first MCE_R motion, the building's period elongated by about 28% relative to the virgin state, and after the second MCE_R motion, the period increased by approximately 35%. The largest

increase was observed in the vibration period corresponding to the flexural-torsional mode in the longitudinal direction.

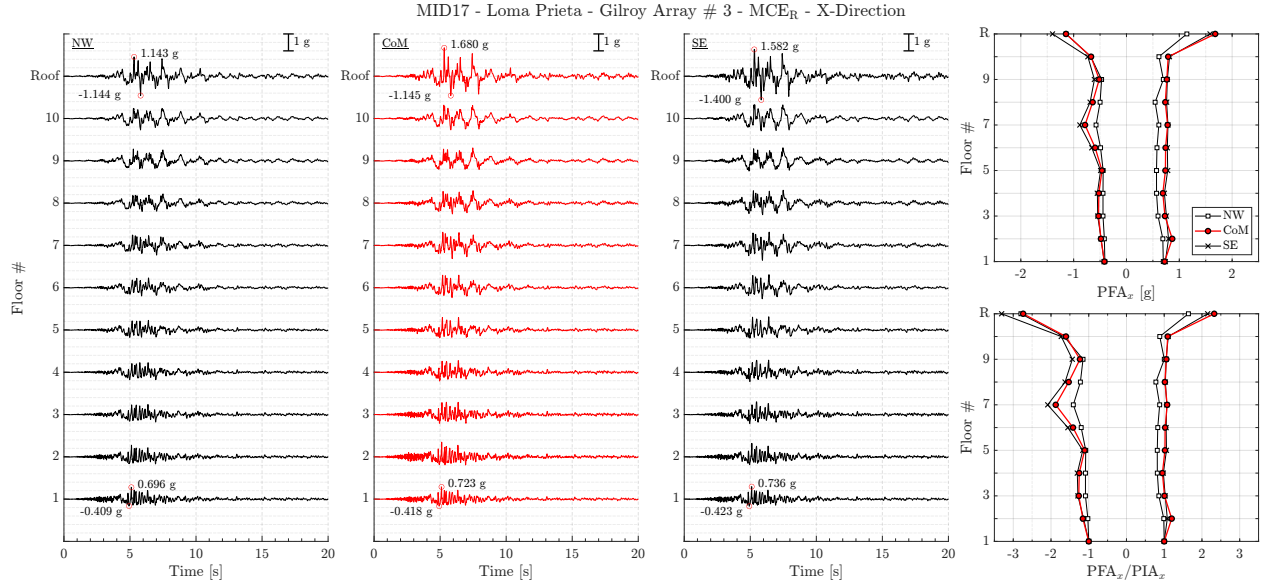


Figure 5. Floor acceleration response histories in the X-direction and peak floor acceleration profile during the risk-targeted maximum considered earthquake (MCE_R, MID17) event, distribution across floor plan.

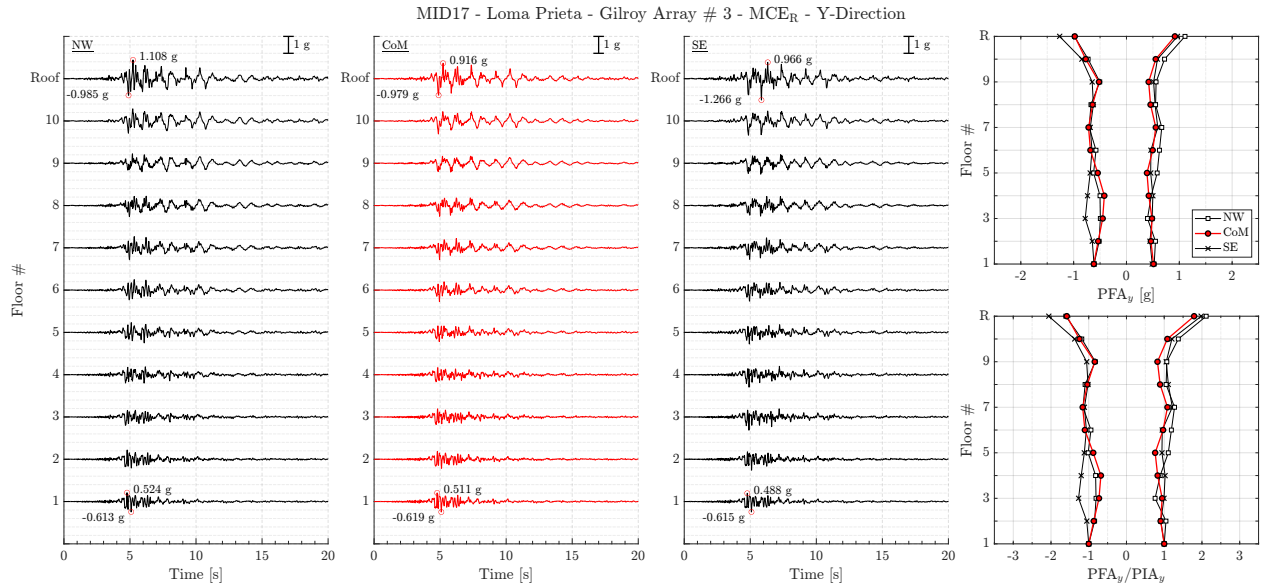


Figure 6. Floor acceleration response histories in the Y-direction and peak floor acceleration profile during the risk-targeted maximum considered earthquake (MCE_R, MID17) event, distribution across floor plan.

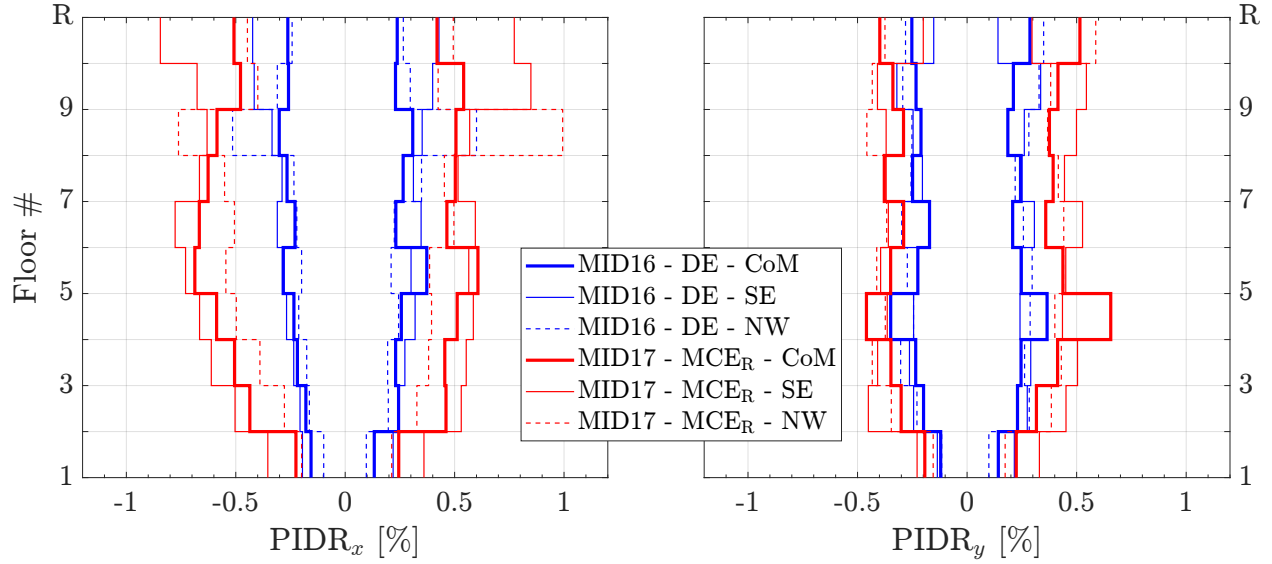


Figure 7. Peak interstory drift ratio profile during the sequential design earthquake (DE, MID16) and risk-targeted maximum considered earthquake (MCE_R, MID17) events, distribution across floor plan.

Table 2. Vibration period and change in period corresponding to the first three modes – as estimated using the SSID method.

State	State Description	Period [s]			Period Change [%]		
		1-L-To	2-T	3-To	1-L-To	2-T	3-To
SL0	Virgin state, stairs locked	0.78	0.74	0.51	0.0	0.0	0.0
SL1	Stairs locked, after MID1	0.81	0.76	0.53	3.8	3.7	5.2
S0	Building's virgin state, stairs unlocked	0.81	0.75	0.53	0.0	0.0	0.0
S1	After SLE1 motions (MID10)	0.82	0.77	0.53	2.0	3.1	1.1
S2	After SLE2 motions (MID15)	0.87	0.78	0.55	7.2	3.7	3.9
S3	After DE motions (MID16)	0.93	0.82	0.61	15.5	9.7	14.4
S4	After MCE _R -1 (MID17)	1.03	0.87	0.65	27.9	16.2	23.3
S5	After MCE _R -2 (MID18)	1.09	0.91	0.63	35.2	20.6	19.1

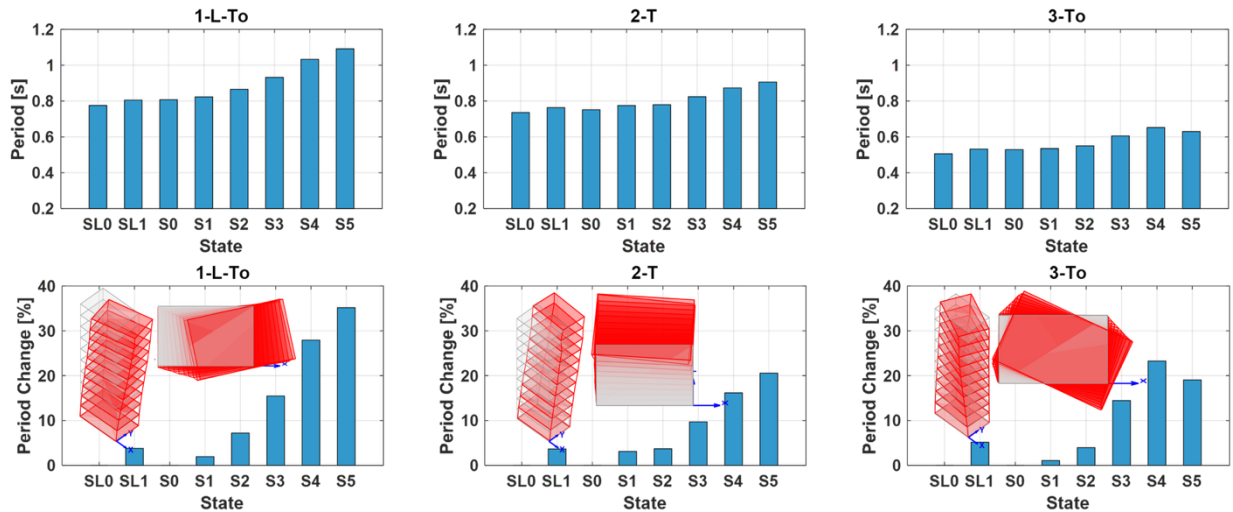


Figure 8. Vibration period and change in period corresponding to the first three modes - as estimated using the SSID method.

Fire Test Protocol & Impressions

Following the earthquake test phase, two live fire compartment tests were conducted to capture the specimen's fire-following-earthquake performance. The Level 9 (L9) Test was performed on the ninth story, followed by the L6 Test on the sixth story, both in the east compartment. The compartment layout was identical for the two tests, see Fig. 3. It contained a single window opening (1.22 m by 1.42 m), corresponding to an opening factor of $0.017 \text{ m}^{1/2}$. During the tests, the window remained open, while the two doors were closed. The third door on the south wall line was framed with a double layer of fire rated gypsum. The fuel load consisted of stacked wood pallets, with total masses of 625 kg (L6) and 533 kg (L9). Assuming a wood calorific value of 17.5 MJ/kg, this corresponds to 391 MJ/m^2 and 334 MJ/m^2 , respectively. For reference, according to the Eurocode EN1991-1-2 (CEN, 2002), the 80% fractile of the fuel load density is 377 MJ/m^2 in a hotel room and 511 MJ/m^2 in an office. The fire test protocol and select results are given in Table 3.

Table 3. Fire test program and key results.

Fire Test	Date	Fuel load	Opening factor	Growth phase	Fully developed phase	Peak gas temperature
Level 9 (L9)	July 17, 2025	533 kg	$0.017 \text{ m}^{1/2}$	17.5 min	17 min	1013 °C
Level 6 (L6)	July 29, 2025	625 kg	$0.017 \text{ m}^{1/2}$	12 min	29 min	1050 °C

A key objective of the fire tests was to assess the thermal-structural response and multi-hazard resiliency of cold-formed steel structures. Therefore, thermal measurements were collected throughout the compartment and on the CFS framing members using Type K thermocouples. Thermocouples were notably placed on the two flanges of wall studs at three heights, on the steel sheathing, and on the bottom flanges of ceiling joists. The vertical gas temperature profiles near the walls were also measured.

Numerical modeling with the fire zone model OZone and the finite element software SAFIR was carried out before the tests (Nui et al., 2025, 2026) to support the test design and predict the expected thermal-structural response. The simulations indicated that, even under unfavorable assumptions regarding the seismic-induced damage, the wall studs were expected to maintain stability provided that steel temperatures remained below 600 °C.

Fire Compartment Behavior

Both tests exhibited typical compartment fire behavior, including a growth phase, flashover, a fully developed phase, and a decay phase (Fig. 9). In both tests, the fire was left to develop and eventually decay (due to fuel consumption) for about 90 minutes in total with no fire service intervention. Flashover occurred after approximately 17.5 minutes in L9 and 12 minutes in L6, with the higher fuel load in L6 producing a slightly longer and hotter fully developed phase (Table 3). Extensive gypsum board damage was observed at the end of both tests. For the wall systems, temperature data showed that wall stud temperatures remained below about 550 °C, which suggests the integrity of the single Type X gypsum layer during most of the test and late-stage detachment during the decay phase. In L6, however, gypsum fall-off from the ceiling occurred

during the fully developed phase and led to joist temperatures exceeding 800 °C. This resulted in significant residual deflections in several CFS ceiling and floor joist members. The tests also led to valuable observations on the floor integrity, door system integrity, and performance of other nonstructural systems, select results of which are presented in the companion paper to this session (Singh et al. 2026b). Overall, the structural system maintained stability throughout the two fire tests, and the recorded thermal responses showed good agreement with numerical model predictions, except in the case of L6 Test floor and ceiling joists, where temperatures were affected by gypsum fall-off.

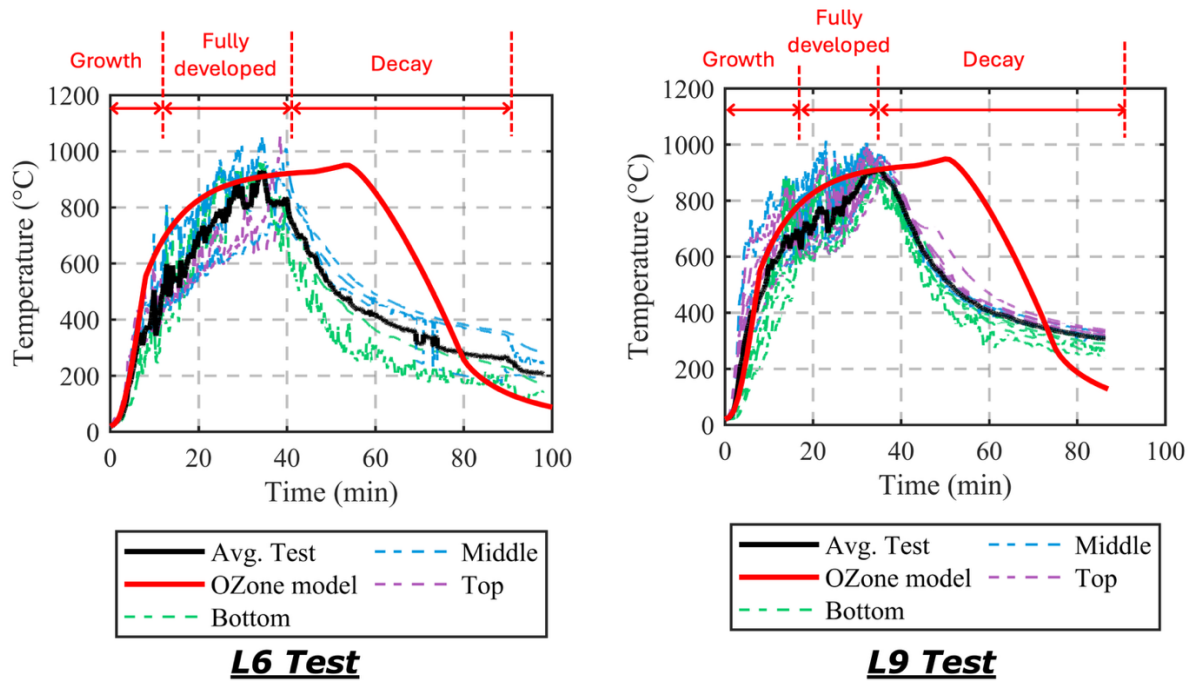


Figure 9. Fire development in the CFS10 post-earthquake fire tests.

CONCLUDING REMARKS: SCOPE OF SESSION

In 2024-2025, a landmark 10-story cold-formed steel building specimen was constructed and tested under multi-hazard (earthquake and fire) scenarios at the UC San Diego Large High-Performance Outdoor Shake Table. Coined CFS10, this unique building specimen is designed beyond current code height limits, adopting advances in cold-formed steel shear wall detailing, varied construction modalities, and enriched with nonstructural components and systems. This paper is the first in a pair presented within a session dedicated to sharing findings from this unique multi-university-government-industry partnership program.

This session will involve six presentations led by the CFS10 team aimed at sharing the essence of results and the diversity of contributions from this landmark program. Within this session, the project team will share a brief overview of the program (1 - Hutchinson); and the related NCS damage-response correlations (2 - Singh). These opening discussions will offer a framework for subsequent presentation of findings from the team, including the following:

- (3) Zhang, J. will share a presentation entitled: “Practice-oriented Numerical modeling and Preliminary Model-Experiment Comparison of a 10-story Shake Table Tested Cold-Formed Steel Building” (Fig. 10).
- (4) Eladly, M. will share a presentation entitled: “Performance Evaluation of a Ten-Story Cold-Formed Steel-Framed Building under Extreme Seismic Events Exceeding MCE_R ” (Fig. 11).
- (5) Emberley et al. will share a presentation entitled: “Post-Earthquake Fire Testing of a Full-Scale 10-Story Cold-Formed Steel Building”.
- (6) Ji, R. will share a presentation entitled: “UAV-Based Remote Sensing Tools for Structural Behavior Monitoring under Earthquake and Fire Hazards” (Fig. 12).

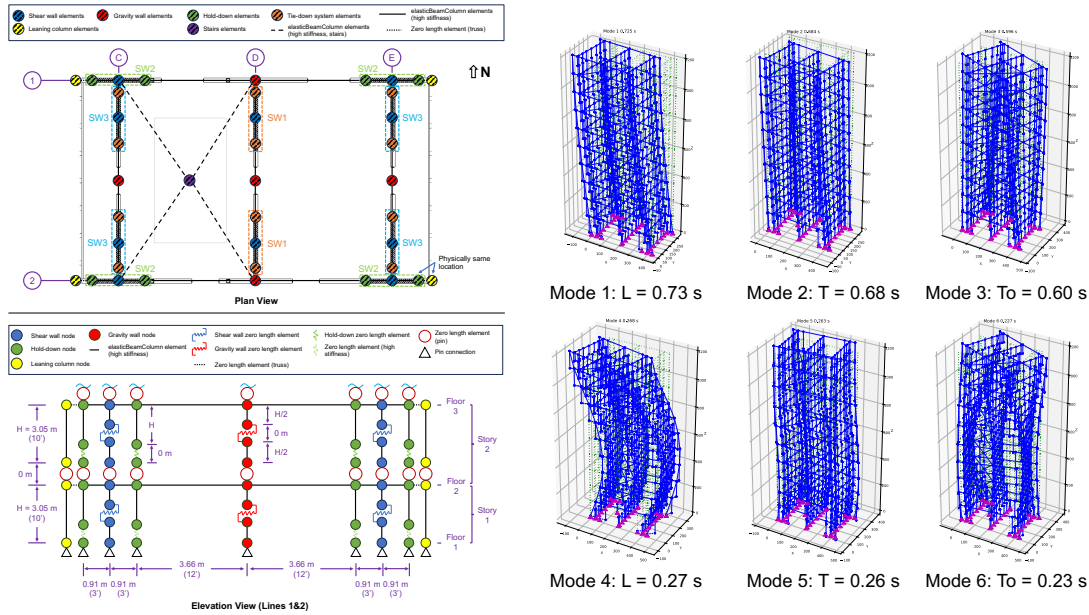


Figure 10. Presentation by Zhang, J. “Practice-oriented Numerical modeling and Preliminary Model-Experiment Comparison of a 10-story Shake Table Tested Cold-Formed Steel Building”

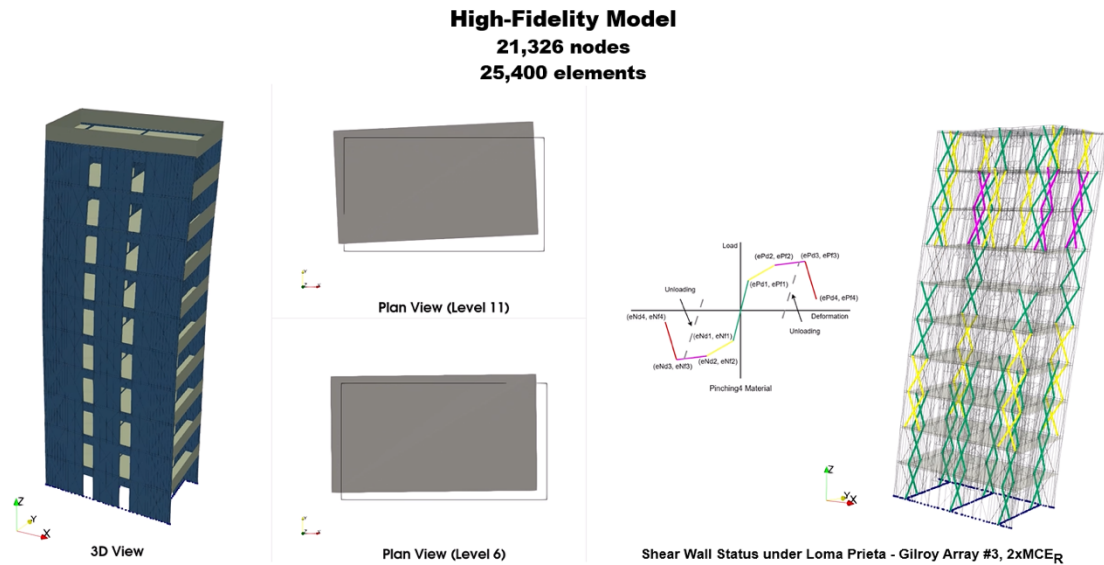


Figure 11. Presentation by Eladly, M. “Performance Evaluation of a Ten-Story Cold-Formed Steel-Framed Building under Extreme Seismic Events Exceeding MCE_R ” (Eladly et al., 2025)

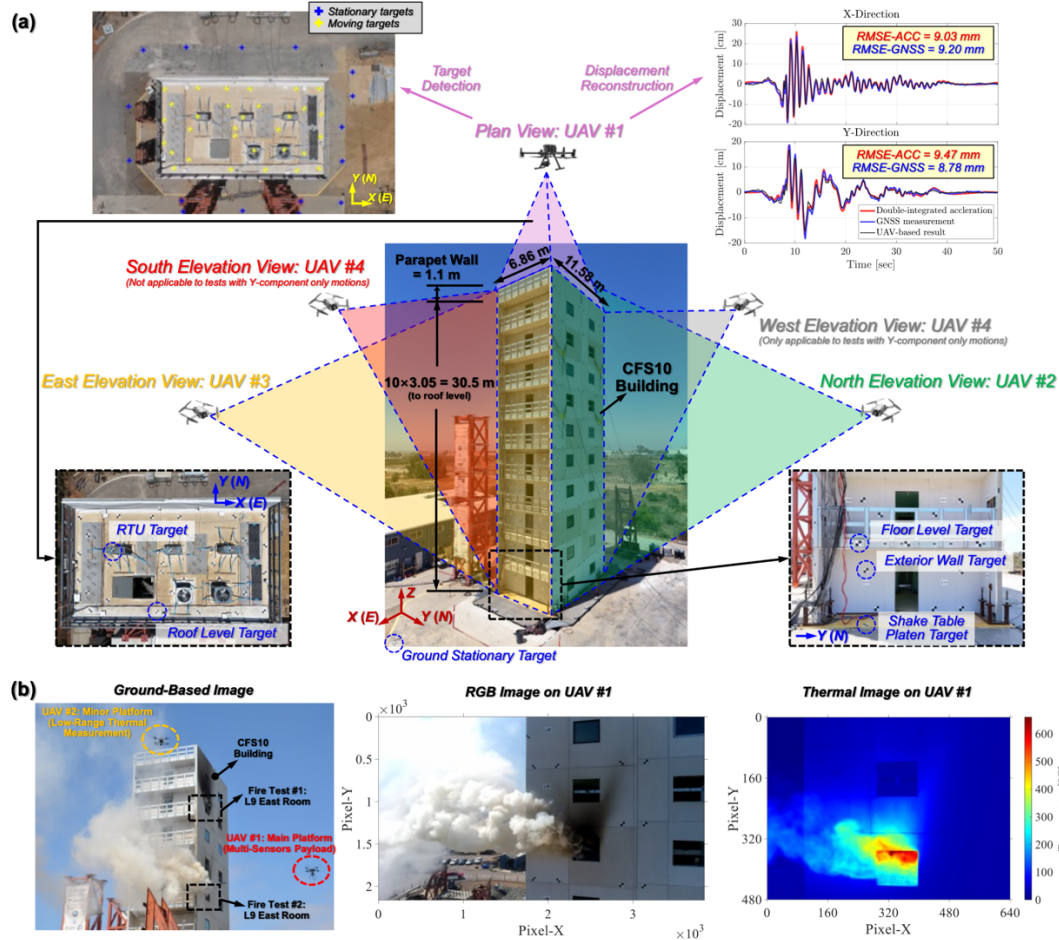


Figure 12. Presentation by Ji, R. “UAV-Based Remote Sensing Tools for Structural Behavior Monitoring under Earthquake and Fire Hazards”

ACKNOWLEDGMENTS

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