GROUND MOTION SELECTION AND SCALING CONSIDERATIONS FOR THE CFS-NHERI CAPSTONE 10-STORY BUILDING SPECIMEN

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Abstract: *Lightweight structures constructed of cold-formed steel (CFS) framing have demonstrated robust structural performance under shake table testing. However, the use of CFS framing for taller structures (>65-ft) in the North American construction industry is prohibited due to code restrictions. To contribute to understanding the seismic performance of tall CFS-constructed buildings, a 10-story full-scale CFS-framed building is planned to commence construction and preparation for shake table testing in late summer 2024. This unique specimen serves as the capstone effort to the NSF-funded Collaborative Research Program entitled: Seismic Resiliency of Repetitively Framed Mid-Rise Cold-Formed Steel Buildings (CFS-NHERI). This benchmark test, coined the CFS-NHERI 10-story building, will be conducted atop the newly upgraded 6-DOF Large High-Performance Outdoor Shake Table (LHPOST6) at NHERI@UC San Diego Experimental Facility. This test will provide a unique opportunity to investigate the system-level performance of tall CFS buildings under multi-directional seismic excitation.*

The selection of the test motion protocol during the planning stages is crucial to ensure that the building specimen is subject to a range of performance scenarios, most notably considering hazards scaled towards service, design, and maximum considered events with varying motion characteristics. To this end, a review of ground motion candidates and testing protocols adopted in previous large-scale tests is conducted. Performance targets of the CFS-NHERI 10-story capstone building test are subsequently defined. Candidate ground motions include twenty-two far-field and twenty-eight near-field ground motion suites from FEMA P695 and near-field ground motions as well as motions from the most recent 2023 Türkiye/Syria Earthquake sequence. The characteristics of these ground motion records are studied and different scaling methodologies, accounting for the characteristics of the test building, are employed. The final set of ground motions and scaling factors will ultimately be selected based on a comprehensive review of the available data, cross-comparison, consultation with the LHPOST6 facility, and results of a sensitivity analysis. At this stage, a preliminary testing protocol is developed to target the performance levels of the 10-story test building.

1. Introduction

1.1. CFS-NHERI 10-story Building Specimen

A 10-story CFS-framed building is scheduled for construction and testing atop the newly upgraded 6-DOF Large High-Performance Outdoor Shake Table Facility (LHPOST6) at NHERI@UC San Diego Experimental Facility (Van Den Einde et al., 2021), presently scheduled to commence in late Summer 2024. This program serves as the capstone effort to the NSF-funded Collaborative Research: *Seismic Resiliency of Repetitively Framed Mid-Rise Cold-Formed Steel Buildings (CFS-NHERI)* project (see https://cfsrc.org/projects/cfs-nheri/).

This capstone experimental program will also provide a unique opportunity to evaluate the post-earthquake fire performance of the earthquake-damaged building. Notably unique to this tall building will be the integration of complete architectural finishes and a building height exceeding the height limitation set by current US design standards (ASCE/SEI 2016, 2022) for building systems designed with such seismic force resisting system. The experiments will provide vital full-scale system-level benchmark test data for a state-of-the-art CFS building under multi-directional seismic input leading to potential improvements to the seismic design codes. Complimentary live fire tests lead by Cal Poly San Luis Obispo will facilitate additional understanding of the thermal and smoke spread within seismically damaged compartments. Additional information regarding the building test program may be found at https://cfs10.ucsd.edu/ (UC San Diego, 2023).

The 10-story test building was designed as a CFS-framed building at a hypothetical location in a highly seismic region near Irvine, California with a Site Class C (very dense soil and soft rock) condition (Singh et al., 2022). The following design parameters were assumed in accordance with ASCE 7-16: Risk Category: II, spectral acceleration at short periods, S_S = 1.261 g, spectral acceleration at a period of 1 second, S_1 = 0.452 g, and design spectral accelerations, S_{DS} = 1.009 g and S_{D1} = 0.452 g. The test building is constructed of all steel CFS studs, CFS-steel sheathed shear walls, gypsum-board-sheathed gravity walls, an exterior face with stucco on the north side, and a ledger-framed CFS diaphragm with cement panel. The lateral force resisting system of the test building consists of Type I shear walls (AISI, 2015) featuring single-sided steel sheet sheathing and ledger framing for floor-to-wall connections. The structure was designed to have a total building height of 30.5 m, which exceeds the height limitation of 19.8 m set by the current ASCE 7-16 and 7-22 design standards. The design effective seismic weight, W, of the building was estimated as 1575.0 kN, and the design seismic base shear force, V, was calculated as 173.2 kN in both orthogonal directions. Singh et al. (2022) summarized the structural design narrative of the test building and provided additional details. More recently, Singh et al. (2024) provides additional design updates regarding the egress system, the addition of a resilient modular stairs tower with drift-release connections. Figure 1 provides a 3D rendering of the proposed CFS-NHERI building specimen on top of the LHPOST6.

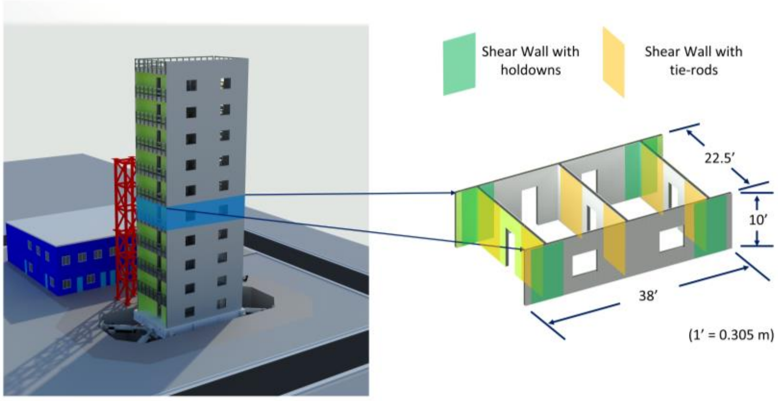

Figure 1. Rendering of CFS-NHERI 10-story test building atop LHPOST6 (view from northeast)

1.2. Design-level Building Period and Site-specific Spectrum

A conventional equivalent lateral force analysis was performed in the design of this test specimen. Accordingly, an approximate fundamental period (T_a) that was required for design purposes and can be determined from the equation as Eq. (1) per ASCE 7-16:

$$T_a = C_t h_n^x \tag{1}$$

where h_n is the structural height (30.5 m) and the coefficients C_t and x are determined as 0.0488 and 0.75. Therefore, the approximate fundamental period of the test building was determined as 0.632 s in both the E-W and the N-S directions. Figure 2 shows the design and MCE_R spectra of the test building, based on the selected Irvine location (S_{DS} = 1.009 g, S_{D1} = 0.452 g, S_{MS} = 1.513 g, S_{M1} = 0.678 g) with an overlay of the design fundamental period as well as the estimated periods of the first three predominant modes from the eigenvalue analysis of the numerical models as presented in Zhang et al. (2024). The comparison of the ASCE 7-16 and 7-22 design spectra of the Irvine site indicates insignificant spectral acceleration differences, with a maximum difference of 0.05 g at the four aforementioned periods. In addition, the Irvine location design spectrum closely resembles the design spectrum at the testing location (UC San Diego). Changing site class, however, will have a substantial impact on the design spectrum.

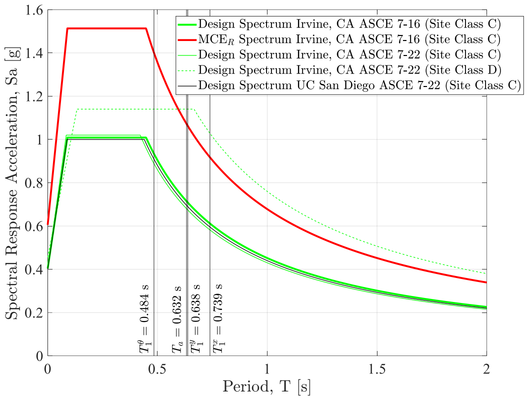

Figure 2. Design and MCE_R spectra of the CFS-NHERI 10-story test building with the overlaid codified estimated building predominant period, and three building periods from the simplified numerical model.

1.3. Ground Motion Protocols Adopted in Prior CFS Building Shake Table Test Programs

Investigation of the overall system-level performance of CFS buildings under earthquake excitation has remained limited. To the authors' knowledge, only two projects thus far have investigated the system-level seismic performance using shake table tests in North America. More recently, another 6-story full-scale CFS building has been tested under bidirectional excitations in China, marking it the tallest CFS building ever tested so far under bidirectional shaking. In these three test programs, test motions were scaled targeting increasing intensity levels with accruing damages in the building. Table 1 summarizes the earthquake motions used in these aforementioned tests, while each is discussed in greater depth in the following.

Peterman et al. (2016) conducted shake table testing on a full-scale two-story CFS-framed building, within the CFS-NEES project, at the NEES@Buffalo site. This project marked the first test to investigate the system-level response of a CFS-framed building in North America. The CFS-NEES test building had a floor plan dimension of 15.2 m x 7 m. Canoga Park and Rinaldi Receiving Station motions from the 1994 Northridge earthquake were selected for seismic testing, representing far-field and near-field motions. The Canoga Park motion at a scale of 100% was defined as the design basis earthquake (DBE), while the Rinaldi motion at 100% was defined as the maximum considered earthquake (MCE) by comparing the response spectrum of these motions against the site characteristic design spectrum. These ground motions were scaled targeting varying hazard levels and were applied following the order of 1D-2D-3D and increasing intensity throughout the three phases. In total 33 earthquake tests were conducted on this test specimen.

Hutchinson et al. (2021) performed seismic and post-earthquake fire testing on a mid-rise full-scale 6-story CFS wall-braced building, coined the CFS-HUD project, at the NEES@UC San Diego shake table facility, which at the time had uni-directional shaking capability. The CFS-HUD test building had a uniform floor plan with a dimension of 10.4 m x 7.3 m and a consistent story height at all levels of 3.1 m. Three motions representative of strong earthquakes in California, namely Canoga Park and Rinaldi Receiving Station motions from the 1994 Northridge earthquake, Rio Dell Overpass motion from the 1992 Cape Mendocino earthquake, and the Curico motion from the 2010 Maule subduction earthquake event in Chile were selected for use in the motion protocol for this program. The Curico record had a significantly longer duration than the other three motions and the Rinaldi record represented the near-fault pulse effect. A total number of seven earthquake tests of increasing motion intensity, scaled from these motion records, sequentially targeting service level (SLE), DBE level, and MCE level earthquakes, were applied uni-directionally along the longitudinal direction of the test building, followed by live fire tests at two selected floors. The building was then uniquely subjected to two post-fire aftershock tests: one SLE motion and one near-fault MCE motion.

Zhou et al. (2022) conducted shake table tests on a full-scale 6-story CFS-framed building in China using bidirectional earthquake input. The test building had a uniform floor plan with a dimension of 5.6 m x 4.2 m and a consistent story height of 2.7 m. Ground motion records from the Wenchuan and Imperial Valley (El Centro) earthquakes as well as a pair of artificial earthquake records were selected as the input excitations.

These ground motions were scaled targeting increasing intensity levels (SLE for regions with higher risk, DBE for regions with medium risk, and MCE for regions with lower risk) and applied in two stages.

Table 1. Summary of earthquake motions used in CFS building tests

Test Program	**Earthquake**				**Recording Station**	**Target Performance**	**# of Tests**
	M_w	**Year**	**Earthquake Event**	**Fault Mechanism**	**Station Name**		
CFS-NEES (Peterman et al.)	6.7	1994	Northridge, California, USA	Reverse	Canoga Park	16%DBE, 44%DBE, DBE	31
	6.7	1994	Northridge, California, USA	Reverse	Rinaldi Receiving Station	16%MCE, MCE	2
CFS-HUD (Hutchinson et al.)	6.7	1994	Northridge, California, USA	Reverse	Canoga Park	SLE, 50%DBE, DBE, MCE	5
	7.1	1992	Cape Mendocino, California, USA	Reverse	Rio Dell Overpass	SLE	2
	8.8	2010	Maule, Chile	Thrust	Curico	SLE	1
	6.7	1994	Northridge, California, USA	Reverse	Rinaldi Receiving Station	MCE	1
6-story CFS-framed building bidirectional tests in China (Zhou et al.)	6.9	1940	Imperial Valley, California, USA	Strike-Slip	El Centro	SLE, DBE, MCE	15
	7.9	2008	Wenchuan, China	Thrust	Wolong	SLE, DBE, MCE	16
	N/A	N/A	Artificial Motions	N/A	N/A	SLE, DBE, MCE	30

Note: SLE: Service-Level Earthquake (43-year return period, 50% probability of exceedance in 30 years); DBE: Design Basis Earthquake (475-year return period, 10% probability of exceedance in 50 years); MCE: Maximum Considered Earthquake (2,475-year return period. 2% probability of exceedance in 50 years).

1.4. Ground Motion Protocols Adopted in Prior Building Shake Table Test Programs of Varying Construction Types

Ground motions used in large/full-scale building programs, considering structures of varying construction types were also reviewed, with emphasis on: a) buildings with similar wall-bracing conceptual details as CFS-framed structures; b) tests programs with similar ground motion protocol targets; and c) tests that were conducted on the recently upgraded LHPOST6 facility with all 6-DOF earthquake input capability. Table 2 summarizes the ground motions used in these selected test programs. In what follows, each program will be discussed with an aim on synthesizing the motion sequence ultimately defined.

In a unique program at the E-Defense facility in Japan, van de Lindt et al. (2010) performed shake table testing on a full-scale 6-story light-frame wood building as part of the NEESWood project. The NEESWood Capstone building had a plan dimension of approximately 18 m x 12 m and a total height of 17 m. The Canoga Park record from the 1994 Northridge earthquake was selected as the input motion and carried through this program, with scaling representing seismic hazard levels of 50%, 10%, and 2% probability of exceedance in 50 years. The test building was subjected to scaled ground motions in all three directions.

Chen et al. (2016) conducted uni-directional shake table testing of a full-scale 5-story reinforced concrete building equipped with a variety of non-structural components and systems. This program, coined the BNCS project, was conducted at the NEES@UC San Diego site. Two motions representative of earthquake events in California: Canoga Park and LA City Terrace from 1994 Northridge earthquake; two motions that had substantially long duration from South American subduction zone events: San Pedro from 2010 Maule, Chile earthquake and ICA from 2007 Pisco, Peru earthquake; and one motion representative of central Alaska events: TAPS Pump Station #9 from 2002 Denali earthquake were selected as seed motions in this program. These ground motions were scaled and applied to the test building while it was in two configurations, namely base-isolated and fixed at its base. For the base-isolated configuration, applied motions were intended to impose minimal damage with a performance target not to exceed 0.5% maximum inter-story drift ratio. For the fixed-base configuration, the test building was sequentially subjected to ground motions scaled to achieve SLE, DBE, and MCE levels.

Table 2. Summary of earthquake motions used in selected building tests with LFRS other than CFS framing

Test Program	**Earthquake**				**Recording Station**	**Target Performance**	**# of Tests**
	M_w	**Year**	**Earthquake Event**	**Fault Mechanism**	**Station Name**		
NEESWood (van de Lindt et al.)	6.7	1994	Northridge, California, USA	Reverse	Canoga Park	50%/50 yr, DBE, MCE	3
BNCS (Chen et al.)	6.7	1994	Northridge, California, USA	Reverse	Canoga Park	SLE	2
	6.7	1994	Northridge, California, USA	Reverse	LA City Terrace	SLE	3
	8.8	2010	Maule, Chile	Thrust	San Pedro	SLE	1
	8.0	2007	Pisco, Peru	Thrust	ICA	SLE	5
	7.9	2002	Denali, Alaska, USA	Strike-Slip	TAPS Pump Station #9	DBE, MCE	2
MTB[2] (Morano et al.)	6.7	1994	Northridge, California, USA	Reverse	Rinaldi Receiving Station	SLE, DBE, Above DBE	19
	6.9	1995	Kobe, Japan	Strike-Slip	Takatori	SLE	7
	7.6	1999	Chi-Chi, Taiwan	Reverse-Oblique	TCU065	SLE	4
TallWood (Pei et al.)	6.6	2004	Niigata, Japan	Thrust	NIG023	See note	
	9.1	2011	Tohoku, Japan	Thrust	CHBH04		
	6.5	2010	Ferndale, California, USA	Strike-Slip	1746		
	7.6	1999	Chi-Chi, Taiwan	Reverse-Oblique	TCU075		
	6.4	1980	Victoria, Mexico	Strike-Slip	SAHOP Casa Flores		
	6.7	1994	Northridge, California, USA	Reverse	Compton - Castlegate St		
	8.0	2003	Tokachi, Japan	Reverse	HKD127		
	6.5	2010	Ferndale, California, USA	Strike-Slip	89486		
	7.0	1989	Loma Prieta, California, USA	Reverse-Oblique	Fremont - Emerson Court		

Note: Data for the TallWood test program is forthcoming as it was being summarized at the time of writing this paper. See the TallWood project website (2023) or Wichman (2023) for more information.

Morano et al. (2023) performed the very first multi-directional shake table testing of a modular re-configurable testbed steel building on the recently upgraded LHPOST6 at NHERI@UC San Diego Experimental Facility. Three ground motions were selected from the suite of motions that were used in the design and acceptance of the upgraded shake table, namely the Rinaldi Receiving Station recording from the 1994 Northridge earthquake, the Takatori recording from the 1995 Kobe, Japan earthquake, and motion TCU065 recorded during the 1999 Chi-Chi, Taiwan earthquake. These ground motions were scaled targeting building performance objectives ranging from elastic, quasi-elastic, design level, and above-design levels and applied to the test building while it was arranged with three different lateral force resisting system (LFRS) configurations. In this case LFRS component limit states were selected in the scaling protocol.

Pei et al. (2023) conducted a series of multi-directional shake table testing on a full-scale 10-story resilient mass timber rocking wall building at NHERI@UC San Diego Experimental Facility. The test building had a floor plan of 9.8 m x 10.5 m and a total height of 35.5 m, which makes it the tallest full-scale building ever tested on a shake table in the world. The test building also featured critical non-structural components including exterior façades, interior walls, and a 10-story resilient stair tower. Eleven ground motions were selected based on the uniform hazard targeted spectrum that was developed for this building's hypothetical design site in Seattle, WA with the inclusion of a considerable amount of subduction zone events. These ground motion candidates were then scaled to hazard levels with 43 yr, 225 yr, 475 yr, 975 yr, and 2,500 yr return periods (Wichman, 2023). As of the end of Phase 1, the test building has been subjected to a total of 100 ground motions. At the time of writing, this project team was compiling findings from this effort, though interested readers may find useful the project website (TallWood, 2023) and related contributions of Wichman (2023) for specifics regarding motion selection and scaling.

1.5. Review of Current Practice on Ground Motion Selection and Scaling Procedure

There have been several important documents developed in the past two decades or so guiding design engineers on the selection and scaling of earthquake motions for use in seismic design and in particular nonlinear response history analysis. A notable synthesis is offered in NIST (2011), which emphasizes at the time current ASCE 7 suggestions. Tall buildings however are unique, often designed with high-performance materials and framing systems outside of current height limits within building codes. For this reason, there are alternative provisions (e.g. SEAONC, 2007; PEER, 2017; and LATBSDC, 2020, 2023) whereby design courtesy of nonlinear time history analysis can be utilized to justify the structural system design. The latter two most recent guidance will be discussed herein.

The Guidelines for Performance-Based Seismic Design of Tall Buildings (PEER, 2017) require characterization of ground motions at Service-Level Earthquake (SLE) and Risk-Targeted Maximum Considered Earthquake (MCE_R). For sites located at close proximity to strong earthquakes, near-fault effects including rupture directivity effects and fling-step effects should be considered. The minimum number of horizontal record pairs for MCE_R level target spectrum is eleven. Seven record pairs should be selected if nonlinear response history analyses are to be performed for SLE earthquakes or three pairs for linear response analyses. The selected ground motions for a particular target spectrum should include an appropriate percentage of motions with near-fault pulse characteristics. For sites where pulse-type ground motions are considered, no less than five records in the pulse or no-pulse subsets should be used.

Similarly, the Los Angeles Tall Buildings Structural Design Council (2020, 2023) recommends that site-specific seismic hazard analysis be used to compute acceleration response spectra for SLE and MCE_R shaking levels. Both probabilistic and deterministic seismic hazard analyses shall be used for the determination of MCE_R level shaking. Ground motion records shall be selected and modified per Chapter 16 of ASCE 7-16; or as applicable per Chapter 16 of ASCE 7-22. The minimum number of horizontal record pairs for MCE_R is eleven. Similarly, no less than five records in the pulse subsets of the ground motion suite shall be used for sites where this type of ground motions are considered.

ASCE 7-16 and 7-22 require that a suite of no less than eleven ground motions be selected for each target spectrum. The selected ground motions shall be modified either through amplitude scaling or spectral matching in the period range of interest. The period range shall have an upper bound greater than or equal to twice the largest first-mode period in the principal horizontal directions and shall have a lower bound such that the period range includes at least the number of elastic modes necessary to achieve 90% mass participation in each horizontal direction. The lower bound period shall not exceed 20% of the smaller first-mode period for the two horizontal directions. Engineers often choose to use amplitude scaling for ground motion modification, which requires the average of the maximum-direction spectra from all the ground motions to not fall below 90% of the target response spectrum within the period range of interest.

2. Candidate Ground Motions from FEMA P695

FEMA P695 (2009) provides a set of far-field and near-field ground motion records each for the evaluation of building collapse performance. The far-field ground motion records include twenty-two motion pairs recorded at sites located at least 10 km from the fault rupture. The near-field record set includes twenty-eight motion pairs recorded at sites located less than 10 km from the fault rupture and includes two subsets: (1) near-field pulse record subset featuring ground motions with strong pulses, and (2) near-field no pulse record subset featuring ground motions that do not have such strong pulses.

As mentioned, the design site class of the CFS-NHERI 10-story test specimen is Site Class C. Based solely on the site class filter criterion, six ground motion records from the far-field ground motion sets (Table 3), five ground motion records from the near-field pulse records subset and ten ground motion records from the near-field no pulse records subset (Table 4) are identified as the candidate ground motions to be considered for the CFS-NHERI capstone 10-story building specimen.

Table 3. Summary of candidate ground motions selected from the FEMA P695 ***far-field*** *set*

ID No.	Earthquake			Recording Station	Site Data			Fault Type
	M	**Year**	**Name**	**Name**	**V_{s30} (m/s)**	**V_{s30} (ft/s)**	**ASCE 7-16 Site Class**	
4	7.1	1999	Hector Mine	Hector	685	2,247	C	Strike-slip
7	6.9	1995	Kobe, Japan	Nishi-Akashi	609	1,998	C	Strike-slip
10	7.5	1999	Kocaeli, Turkey	Arcelik	523	1,716	C	Strike-slip
15	7.4	1990	Manjil, Iran	Abbar	724	2,375	C	Strike-slip
20	7.6	1999	Chi-Chi, Taiwan	TCU045	705	2,313	C	Reverse-Oblique
22	6.5	1976	Friuli, Italy	Tolmezzo	425	1,394	C	Thrust

Table 4. Summary of candidate ground motions selected from FEMA P695 ***near-field*** *set*

ID No.	Earthquake			Recording Station	Site Data			Fault Type
	M	**Year**	**Name**	**Name**	**V_{s30} (m/s)**	**V_{s30} (ft/s)**	**ASCE 7-16 Site Class**	
				Pulse Records Subset				
5	6.9	1989	Loma Prieta	Saratoga - Aloha	371	1,217	C	Reverse-Oblique
7	7.0	1992	Cape Mendocino	Petrolia	713	2,339	C	Reverse
8	7.3	1992	Landers	Lucerne	685	2,247	C	Strike-slip
10	6.7	1994	Northridge-01	Sylmar - Olive View	441	1,447	C	Reverse
13	7.6	1999	Chi-Chi, Taiwan	TCU102	714	2,343	C	Reverse-Oblique
				No Pulse Records Subset				
15	7.0	1984	Gazli, USSR	Karakyr	660	2,165	C	Thrust
18	6.8	1985	Nahanni, Canada	Site 1	660	2,165	C	Blind Thrust
19	6.8	1985	Nahanni, Canada	Site 2	660	2,165	C	Blind Thrust
20	6.9	1989	Loma Prieta	BRAN	376	1,234	C	Reverse-Oblique
21	6.9	1989	Loma Prieta	Corralitos	462	1,516	C	Reverse-Oblique
22	7.1	1992	Cape Mendocino	Cape Mendocino	514	1,686	C	Reverse
23	6.7	1994	Northridge-01	LA - Sepulveda VA	380	1,247	C	Reverse
26	7.6	1999	Chi-Chi, Taiwan	TCU067	434	1,424	C	Reverse-Oblique
27	7.6	1999	Chi-Chi, Taiwan	TCU084	553	1,814	C	Reverse-Oblique
28	7.9	2002	Denali, Alaska	TAPS Pump Sta. #10	553	1,814	C	Strike-slip

The scaling method specified in FEMA P695 involves two elements: normalization and scaling. First, each individual record is normalized by their respective peak ground velocities to remove unwarranted variability between records. The normalized record set will be then scaled so that the median value is anchored to the MCE demand level of shaking at the fundamental period of the building. Figures 3 and 4 present the normalized and scaled spectra for the selected ground motion records from the far-field set and near-field set, respectively. Tables 5 and 6 summarize the normalization and scaling factors used in this process.

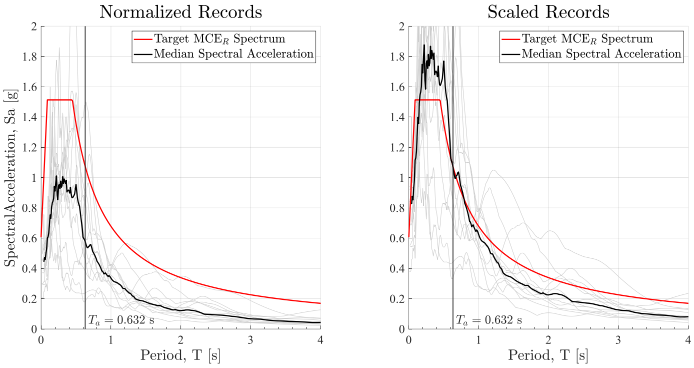

Figure 3. Normalized and scaled spectra for selected FEMA P695 ***far-field*** *ground motions (traces in grey are individual recordings, N = 6)*

Table 5. Summary of normalization factors and scaling factors for candidate ground motions from FEMA P695 ***far-field*** *set*

ID No.	**NF**	**SF**	**SF$_{Final}$**	**ID No.**	**NF**	**SF**	**SF$_{Final}$**
4	1.09	1.86	2.02	15	0.79	1.86	1.47
7	1.03	1.86	1.91	20	0.96	1.86	1.78
10	1.36	1.86	2.52	22	1.44	1.86	2.67

Note: NF: Normalization Factor; SF: Scaling Factor (from normalized); SF$_{Final}$ = NF*SF

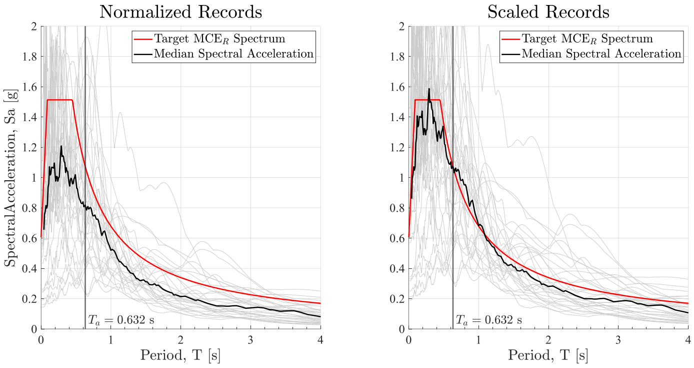

Figure 4. Normalized and scaled spectra for selected FEMA P695 ***near-field*** *ground motions (traces in grey are individual recordings, N = 15)*

Table 6. Summary of normalization factors and scaling factors for candidate ground motions from FEMA P695 ***near-field*** *set*

ID No.	**NF**	**SF**	**SF$_{Final}$**	**ID No.**	**NF**	**SF**	**SF$_{Final}$**
Pulse Records Subset				**No Pulse Records Subset**			
5	1.63	1.31	2.14	15	0.86	1.31	1.13
7	1.08	1.31	1.42	18	1.27	1.31	1.67
8	0.77	1.31	1.01	19	1.95	1.31	2.56
10	0.80	1.31	1.05	20	1.15	1.31	1.51
13	0.86	1.31	1.13	21	1.17	1.31	1.54
				22	0.66	1.31	0.87
				23	0.77	1.31	1.01
				26	0.78	1.31	1.02
				27	0.62	1.31	0.81
				28	0.57	1.31	0.75

Note: NF: Normalization Factor; SF: Scaling Factor (from normalized); SF$_{Final}$ = NF*SF

3. Site-Specific Ground Motion Suite Development

While the FEMA P695 subset offers a useful motion suite for consideration in this study, particularly developed for use in collapse analysis studies, a site-specific ground motion suite is also of interest. The 10-story test specimen was designed as a CFS-framed building at a hypothetical location near Irvine, California. Considering the regional seismic hazard, a site-specific probabilistic seismic hazard analysis was available for an actual building at the nearby University of California Irvine campus with a similar site class (i.e., Site Class C) with $S_{MS} = 1.513\,g$, $S_{M1} = 0.678\,g$. Utilizing this hazard analysis, a suite of eleven site-specific ground motion records was selected. This set of ground motion records serves as a basis for the site-specific ground motion suite development. At the time of preparation of this paper, the site-specific ground motion development is still progressing, however, Table 7 provides a summary of the aforementioned motion suite and Figure 5 presents the scaled spectra of the selected ground motion suite overlaid with the UC Irvine location MCE_R spectrum.

Table 7. Summary of site-specific ground motion suite for a nearby UC Irvine location with Site Class C

Rec #	**NGA RSN #**	**Earthquake**			**Recording Station**	**Site Data**		**Fault Mechanism**
		M_w	**Year**	**Name**	**Name**	**V_{s30} (m/s)**	**D_{rup} (km)**	
1	6	7.0	1940	Imperial Valley-02	El Centro Array #9	213	6.09	Strike Slip
2	139	7.4	1978	Tabas, Iran	Dayhook	472	13.94	Reverse
3	767	6.9	1989	Loma Prieta	Gilroy Array #3	350	12.82	Reverse-Oblique
4	802	6.9	1989	Loma Prieta	Saratoga - Aloha Ave	381	8.50	Reverse-Oblique
5	1148	7.5	1999	Kocaeli, Turkey	Arcelik	523	13.49	Strike Slip
6	1158	7.5	1999	Kocaeli, Turkey	Duzce	282	15.37	Strike Slip
7	1511	7.6	1999	Chi-Chi, Taiwan	TCU076	615	2.74	Reverse-Oblique
8	1549	7.6	1999	Chi-Chi, Taiwan	TCU129	511	1.83	Reverse-Oblique
9	1602	7.1	1999	Duzce, Turkey	Bolu	294	12.04	Strike Slip
10	5678	7.2	2008	Iwate-Miyagi, Japan	MYGH02	399	11.12	Reverse
11	6893	7.0	2010	Darfield, New Zealand	DFHS	344	11.86	Strike Slip

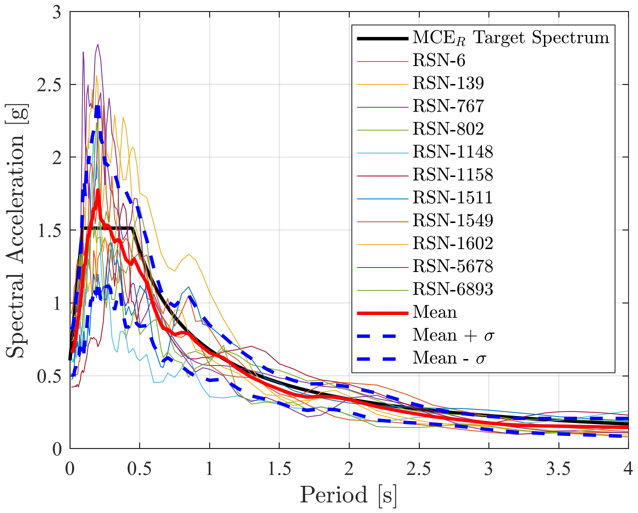

Figure 5. Spectra of scaled site-specific ground motion suite overlay with UC Irvine location MCE_R spectrum.

4. Considering Earthquake Sequence Effects

The catastrophic 2023 Türkiye/Syria earthquake sequence has resulted in a significant concentration of reinforced concrete and CMU-infill collapsed buildings. The Disaster and Emergency Management Presidency of Türkiye (AFAD) has installed an extensive network of recording stations (tadas.afad.gov.tr). Based on the preliminary reconnaissance report from Middle East Technical University (2023), eight stations were selected for consideration in the present study, those selected were based on: (1) same site class as the 10-story test specimen (Site Class C or TBEC Site Class ZC (AFAD, 2019)); and (2) with distances smaller than 100 km from the rupture location of the two Türkiye/Syria earthquakes: M_w 7.8 Kahramanmaraş Earthquake and M_w 7.5 Elbistan Earthquake. These records will provide a valuable opportunity to investigate the performance of the 10-story CFS building considering an earthquake sequence. Table 8 summarizes the rupture distances for both events to the selected stations. The locations of those stations can be seen in Figure 6.

Table 8. Summary of selected stations for Türkiye/Syria earthquake sequence (METU, 2023)

Station #	R_{rup1} (km)	R_{rup2} (km)	V_{s30} (m/s)	TBEC Site Class	V_{s30} (ft/s)	ASCE 7-16 Site Class
127	95.8	73.7	583	ZC	1,913	C
4405	94.5	85.3	579	ZC	1,900	C
4611	18.5	32.3	731	ZC	2,398	C
4615	10.3	70.2	484	ZC	1,588	C
4616	2.3	67.8	390	ZC	1,280	C
4617	22.2	44.6	574	ZC	1,883	C
4620	19.3	45.4	484	ZC	1,588	C
8002	15.2	89	430	ZC	1,411	C

Note: R_{rup1}: R_{rup} for M_w 7.8 Kahramanmaraş Earthquake; R_{rup2}: R_{rup} for M_w 7.5 Elbistan Earthquake

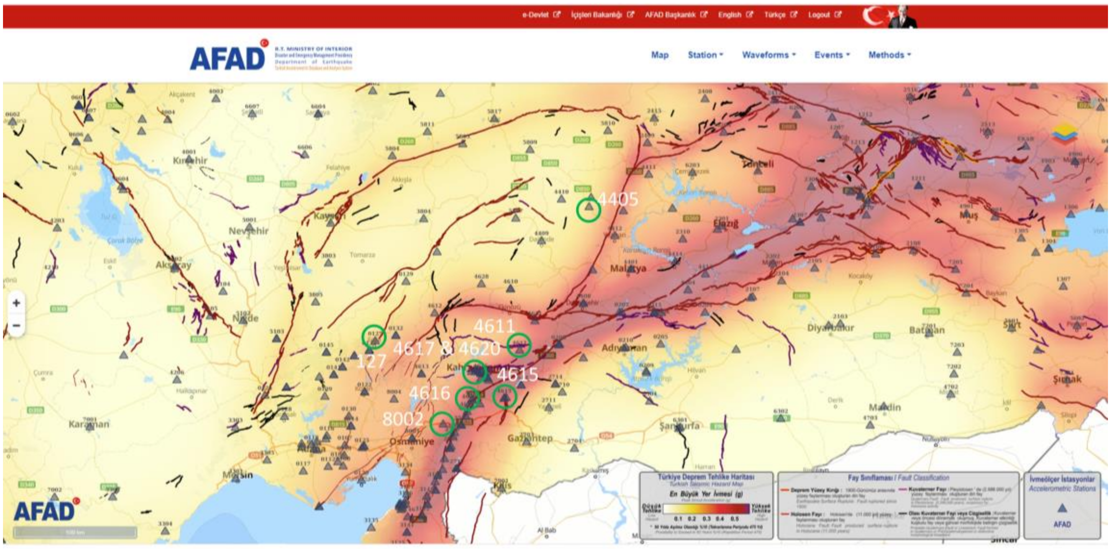

Figure 6. Station locations and active faults on AFAD map (https://tadas.afad.gov.tr/map).

5. Summary and Future Steps

In preparation for a shake table test program of a full-scale 10-story CFS-framed building considering multi-directional earthquake excitation, a review of ground motion candidates and testing protocols adopted in previous large-scale tests is conducted. In particular, the present paper focuses on: (1) prior CFS-framed building tests; (2) similar wall-braced building system tests; and (3) tests performed at the same experimental facility. Review of current ground motion selection and scaling practice as adopted for use in nonlinear time history analysis is also performed. Notably candidate motion suites being considered are from three sources: (1) the FEMA P695 motion suite, (2) a site in close proximity that selected for the design-basis spectrum (i.e., site-specific ground motions) and (3) motions recorded from the 2023 Türkiye/Syria earthquake. The latter is being considered to allow investigation of the impact of accruing damage due to sequences of earthquakes. At the time of preparation of this paper, the scaling factors and testing protocols are under development. Next steps involve performing nonlinear response history analyses utilizing the numerical models developed for this project (Zhang et al., 2024). The predicted responses will then be evaluated to target the performance levels of the 10-story test building. Additional limits from the shake table facility will also be considered in the selection of final motions for use in the test program.

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