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Seismic Analysis of the 10-Story CFS-NHERI Building

Jiachen Zhang¹, Amanpreet Singh², Tara C. Hutchinson³, Xiang Wang⁴

Abstract

A 10-story full-scale cold-formed steel building is scheduled to be tested on the newly upgraded 6-DOF Large High-Performance Outdoor Shake Table (LHPOST6) at the University of California San Diego serving as the capstone effort to the NSF-funded Collaborative Research: Seismic Resiliency of Repetitively Framed Mid-Rise Cold-Formed Steel Buildings (CFS-NHERI) project. In preparation for this test program, a simplified pancake model is developed in OpenSees to predict the dynamic characteristics and to simulate the behavior of this 10-story test building under nonlinear dynamic response history analysis. The pancake model is composed of rigid diaphragms and nonlinear shear springs representing the hysteretic behavior of the lateral load resisting shear walls. Pinching4 material properties calibrated using past experiments defined the properties of shear springs with scaling based on geometric properties. To evaluate the robustness of the approach, a similar model of a 6-story CFS building, tested under uni-directional shaking was also developed. For the 6-story modeling effort, the dynamic characteristics and response under the same series of ground motions used in the shake table tests are compared with the test results and good agreement is observed. Thus, for the 10-story CFS-NHERI building (pre-test) analyses, a suite of 22 far-field ground motion pairs provided in FEMA P695 are chosen to perform nonlinear response history analysis. The responses under these scaled ground motions are compared to investigate the effect on the seismic response of the 10-story CFS-NHERI building. In future work, the responses of the pancake model will be compared with other numerical models being developed by fellow researchers within the CFS-NHERI team using more complex wall line representations for example. At present however, it is recognized that the simplified pancake model provides a fast computational turnaround time, while effectively capturing the characteristics and providing reasonable estimates of the responses, which makes it useful for pre-test analyses to identify a candidate suite of ground motions to be used for the future shake table test program.

1. Introduction

Three-dimensional structural analysis models are essential to simulate the behavior of building systems under varying loading conditions. However, the complexity and redundancy in the structural systems can be more prominent at the building level. The level of modeling detail greatly affects the required computational resources and using highly refined models can overwhelm the computing capability. Thus, model reduction techniques must be implemented to offer efficient numerical models that provide reasonable results as well as reasonable computational overhead. To this end, simplified pancake model ideas have been employed in the realm of wood structures. For example, Folz and Filiatrault

[1][2] developed a simple numerical model to predict the dynamic characteristic, quasi-static pushover, and seismic response of the woodframe test building within the CUREE-Caltech Woodframe Project. This model was composed of rigid horizontal diaphragms and nonlinear shear spring elements representing the lateral load resisting shear walls. The predictive capabilities of the model were then compared with the experimental results and shown to provide reasonably accurate estimates of the response. Along a similar strategy, van de Lindt et al. [3] developed a SAPWood model within the NEESWood Project. Predictions derived from the SAPWood model were compared against the experimental seismic responses of a full-scale two-story light-frame wood townhouse building and good agreement was observed. In the aforementioned and other efforts with the woodframe research contributions, pancake models developed were also shown to provide fast computational time and require minimal computational resources and as a result may be useful tools for practicing engineers and researchers for seismic analysis and design of such building systems.

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Research conducted on various components of buildings using cold-formed steel (CFS) framing solutions has significantly contributed to the advances and development of codes and standards such as AISI S100[4], S240[5], and S400[6], and to the better understanding of the behavior and CFS framing components. However, investigation of the system-level performance of cold-formed steel buildings under earthquake excitation has been very limited. To the authors' knowledge, only two projects thus far have investigated the system-level seismic performance using large shake tables in Northern America. Peterman et al. [7]–[9] conducted a two-story low-rise CFS-framed building shake table test at the NEES@Buffalo site within the CFS-NEES project. This test program involved two phases with various construction details. The test building was subjected to a series of white noise vibrations and earthquake excitation until destructive testing at the maximum considered earthquake level in the final stage. Hutchinson et al. [10]–[12] conducted shake table testing of a six-story mid-rise CFS-framed building at the NEES@UC San Diego site within the CFS-HUD project and investigated the earthquake and post-earthquake fire performance of the building system.

The upcoming 10-story CFS-NHERI building benchmark test will provide a unique opportunity to investigate the system-level performance of now much taller CFS buildings under multi-directional seismic excitation. In preparation for this effort, a simplified numerical pancake model was developed in OpenSees [13]–[15] to predict the dynamic characteristics and to simulate the nonlinear behavior of the test building. To evaluate the robustness of the modeling approach, a similar model of the 6-story CFS-HUD test building was also developed and responses of the pancake model under the same series of ground motions were compared with the test data.

1.1 Available Shake Table Test Data

The full-scale shake table testing of a six-story mid-rise CFS-framed building conducted by Hutchinson et al. [10]–[12] at the NEES@UC San Diego site within the CFS-HUD project was the tallest CFS building ever tested at that time and the first of its kind that underwent live fire testing and subsequent post-fire earthquake tests. It provided a unique opportunity to evaluate the earthquake and post-earthquake fire performance of CFS building systems. The test building was subjected to a sequence of white noise vibrations and seven earthquake tests of increasing intensity targeting service, design, and maximum considered level earthquake demands during the pre-fire test stage. Live fire tests were conducted subsequently at two selected floors and finally the test building was subject to two post-fire earthquake tests including a low-amplitude motion and a near-fault maximum considered level earthquake motion. Figure 1 shows the isometric view and the building plan layout for the CFS-HUD test building.

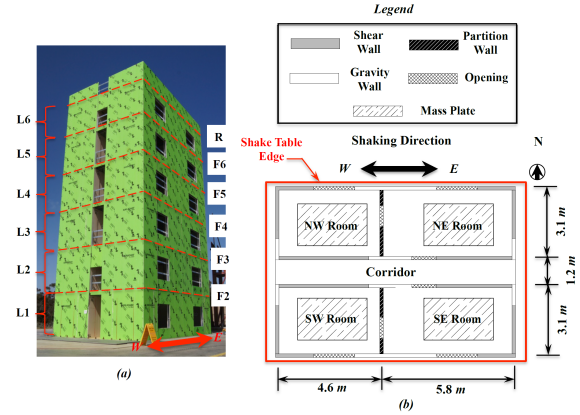


Figure 1: CFS-HUD test building: (a) isometric view, and (b) building plan layout [10][12].

1.2 Upcoming CFS-NHERI Shake Table Test Program

The multi-university-industry project entitled: *Seismic Resiliency of Repetitively Framed Mid-Rise Cold-Formed Steel Buildings*, funded through the National Science Foundation (NSF), referred to as CFS-NHERI, was undertaken to advance knowledge of the seismic performance of mid-rise CFS-framed building systems and to use this knowledge to support improvements in the seismic design codes for such systems. Thus far within CFS-NHERI, a set of experiments have been conducted to better understand and capture nonlinearities at the component and connection level within the CFS-NHERI project (Singh et al. [16]–[21], Zhang et al. [22]–[24], Castaneda et al. [25]). Singh et al. [16]–[21] tested wall line assemblies with various configurations utilizing shake table and quasi-static testing. Specimen details include unfinished or finished, symmetrical or unsymmetrical, Type I or Type II shear wall detailing configurations, tension tie rods or holdowns as anchorage detailing, and presence of window openings. A significant increase in the wall line lateral strength and stiffness caused by finish application on the exterior wall line has been observed. Zhang et al. [22]–[24] conducted connection tests to investigate the screw connection behavior to support the use of steel sheet sheathing with HSS sections. Castaneda [25] recently tested floor diaphragm assemblies using CFS joists and various panels to characterize the lateral performance of such floor systems at the component level. The system-level tests will build on the learnings from these connection- and component-level tests performed within the CFS-NHERI project. This unique tall CFS building will adopt the use of prefabricated CFS wall-lines and floor diaphragms for rapid construction at the test site.

A 10-story CFS-framed test building will be constructed and tested atop the newly upgraded 6-DOF Large High-Performance Outdoor Shake Table Facility (LHPOST6) at

NHERI@UC San Diego Experimental Facility [26]. This 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 7-22 [27]). 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 seismic design codes improvements. Complimentary live fire tests performed by Cal Poly San Luis Obispo will facilitate additional understanding of the thermal and smoke spread within seismically damaged compartments.

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 corresponding Site Class C (very dense soil and soft rock) condition (see Singh et al. [28]). The following design parameters were assumed in accordance with ASCE 7-16 [29]: risk category: II, spectral acceleration at short periods, $S_s = 1.261$ g, spectral acceleration at a period of 1 s, $S_1 = 0.452$ g, and design spectral accelerations, $S_{DS} = 1.009$ g and $S_{D1} = 0.452$ g. The lateral force resisting system of the test building consists of Type I shear walls with 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 (100 ft), which exceeds the height limitation of 19.8 m (65 ft) set by the current ASCE 7-16 [29] and 7-22 [27] design standards. Singh et al. [28] summarized the structural design narrative of the test building and provided additional details. Figure 2 provides a simplified 3D rendering of the proposed CFS-NHERI test building.

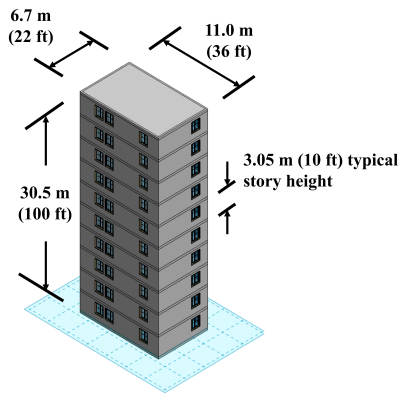


Figure 2: Simplified rendering of the 10-story CFS-NHERI test building (note: actual floor plan varies slightly).

2. Pancake Model Development and Analysis for CFS-HUD Project

2.1 Model Description

The six-story CFS-HUD test building (see Figure 1) had a uniform plan dimension of 10.4 m x 7.3 m (34 ft x 24 ft) for each floor. Note that base input excitation is imposed along the long axis of this building. Each story had a floor-to-floor height of 3.1 m (10 ft). Two longitudinal shear walls were placed along the east and west ends of the corridor, with a wall length of 4.0 m (13 ft) for the walls at the west end and 3.3 m (11 ft) for those at the east end. L-shaped 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 each corner of the building. The total length of shear wall was 21.3 m (70 ft) in the longitudinal direction and 8.6 m (28 ft) in the transverse direction [10]–[12].

OpenSeesPy [15] was used in the development of the numerical pancake model. The general solution strategies involved applying gravity first, assigning uniform excitation, and performing nonlinear transient analysis using Newton-Raphson solution algorithm with a Newmark integrator of $\gamma = 1/2$ and $\beta = 1/4$.

Each shear wall segment had a node to which a zero-length shear spring was connected. Figure 3 shows the locations of the shear springs adopted for this simplified model. In total, there are 8 shear springs in the longitudinal direction (blue) and 4 shear springs in the transverse direction (green). It should be noted that the contributions of the longitudinal shear springs to the rotational stiffness remain the same regardless of the exact locations of the nodes for the longitudinal direction as long as they remain along the same horizontal line in plan view, due to the decoupling modeling approach. Therefore, these nodes were moved to the edge as shown in Figure 3.

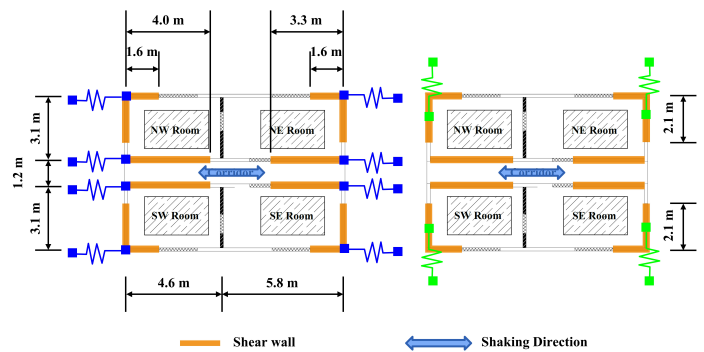
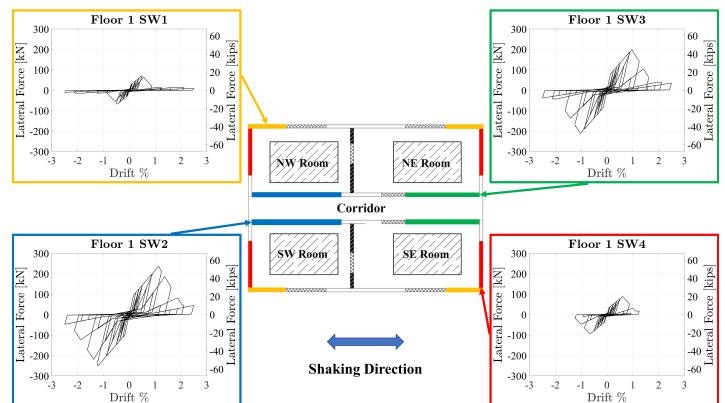


Figure 3: Zero-length shear spring locations assumed for CFS-HUD building model.

2.2 Hysteretic Modeling of Shear Walls

Figure 10 consists of four sub-figures labeled (a) through (d).
 (a) A photograph of the 1/4 scale specimen, a rectangular frame structure. Dimensions are indicated: height is 2.7 m (9 ft) and width is 3.7 m (12 ft).
 (b) A plot of Lateral Force [kN] (left y-axis, -200 to 200) versus Drift % (x-axis, -3 to 3). It shows a hysteresis loop. The legend indicates 'Experimental' as a solid blue line and 'Pinching4' as a dashed black line.
 (c) A plot of Dissipated energy per cycle [kN-mm] (left y-axis, 0 to 10000) versus Cumulative drift [mm] (x-axis, 0 to 2500). The legend shows 'Experimental' as a solid blue line with diamond markers and 'Pinching4' as a dashed black line with square markers.
 (d) A plot of Cumulative dissipated energy [kN-mm] (left y-axis, 0 to 4, with a multiplier of $\times 10^4$) versus Cumulative drift [mm] (x-axis, 0 to 2500). The legend shows 'Experimental' as a solid blue line with diamond markers and 'Pinching4' as a dashed black line with square markers.

The pinching4 material properties for each shear wall were then scaled based on the actual geometric properties and detailing for each shear wall within the CFS-HUD building. Figure 7 shows the scaled pinching4 material properties of level 1 shear springs for the CFS-HUD model.



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Table 1: Comparison of building periods for CFS-HUD model and test. (L = longitudinal, T = transverse, and To = torsional mode)

Mode	1-T	1-L	1-To	2-T	2-L	2-To
Model	0.48 s	0.31 s	0.27 s	0.16 s	0.11 s	0.09 s
Test	0.45 s	0.26 s	0.23 s	0.13 s	0.08 s	0.07 s

*1-T, 1-L, 1-To: 1st transverse, longitudinal mode, and torsional mode;
2-T, 2-L, 2-To: 2nd transverse, longitudinal mode, and torsional mode;

Table 2: Summary of ground motions.

Short Name	Earthquake - Station (Performance Target)
EQ1:RIO-25	1992 Cape Mendocino - Rio Dell Overpass (Service Level)
EQ2:CNP-25	1994 Northridge - Canoga Park (Service Level)
EQ3:CUR-25	2010 Maule, Chile - Curico (Service Level)
EQ4:CNP-25*	1994 Northridge - Canoga Park (Service Level)
EQ5:CNP-50	1994 Northridge - Canoga Park (50% Design Level)
EQ6:CNP-100	1994 Northridge - Canoga Park (Design Level)
EQ7:CNP-150	1994 Northridge - Canoga Park (MCE Level)

*Note: EQ4 is a repeat of EQ2, therefore not considered in future analyses

2.3 Summary of Analyses and Results

2.3.1 Eigen Analysis

Eigen analysis was first performed using the aforementioned numerical model of the CFS-HUD test building. Table 1 summarizes the comparison of building periods for both the numerical model and the shake table test. Good agreement is observed in this comparison, both in terms of sequence of fundamental modes and amplitude of associated periods. However, a consistent trend of nominally larger periods ($\lesssim 10\%$) from the numerical model is also observed. This could result from a lower stiffness in the numerical model caused by the assumption that only shear walls contribute to the lateral stiffness.

2.3.2 Nonlinear Response History Analysis

The recorded shake table platen motion obtained during testing were then used as the input ground motions records for nonlinear response history analysis for the CFS-HUD numerical model. The input earthquake ground motion records used in the pre-fire test stage were selected from three historical earthquake events, namely Rio Dell Overpass from the 1992 Cape Mendocino earthquake, Canoga Park from the 1994 Northridge earthquake, and Curico from the 2010 Maule earthquake in Chile. Table 2 summarizes the input ground motions for the shake table tests used during the pre-fire stage and their respective performance targets. Figure 8 shows the acceleration time histories of these motions which were directly input to the numerical model obtained from the shake table test results. It can be clearly observed that the first seven ground motion inputs used in the pre-fire test stage were applied at increasing intensity. Figure 9 shows the 5% damped elastic pseudo-spectral acceleration of the input ground motions. Note that the intensity of the input motions was sequentially scaled to span from service

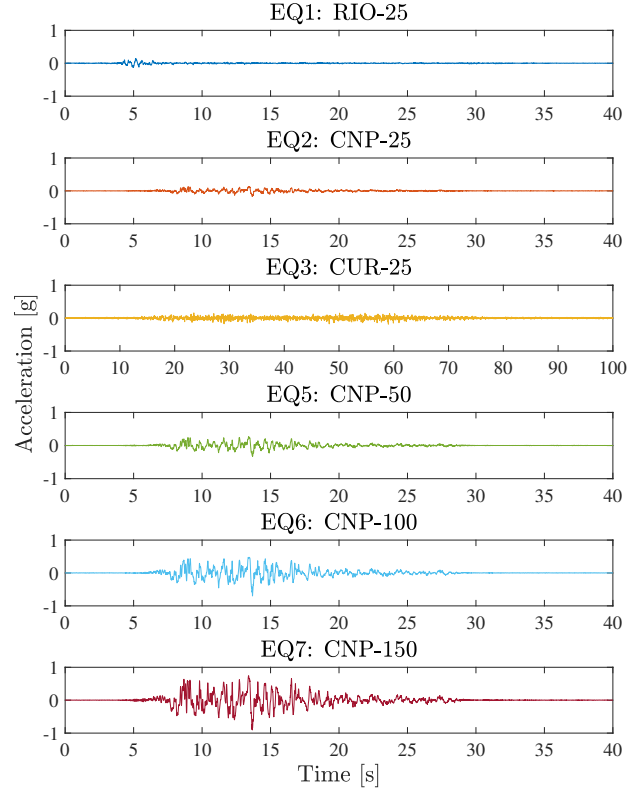


Figure 8: Acceleration time histories of input ground motions. Note that test motion EQ3: CUR-25 is of significantly longer duration, therefore its x-axis range differs from all others.

through to MCE demands, resulting in the peak acceleration of the ground motions increasing from 0.14 g to 0.9 g, while the spectral accelerations at the fundamental period (period for the 1st longitudinal mode) increased from 0.3 g to 1.7 g with the increasing intensity.

Figure 10 shows a comparison of the peak floor acceleration (PFA) distribution obtained from the numerical model with that measured during testing considering three ground motions that target service, design and MCE performance objectives. In general, the comparison is reasonable. The PFA distributions obtained from the numerical model and the shake table tests match significantly well for service and design level ground motions. Less favorable comparison between the test and numerical simulation PFAs are realized at the MCE level earthquake, with an overestimation as large as approximately 2-fold in the negative PFA (floor 5). However, the remainder of the floor peak acceleration are within 30-50% of those in tests.

The floor response spectra (FRS) for the pre-fire earthquake

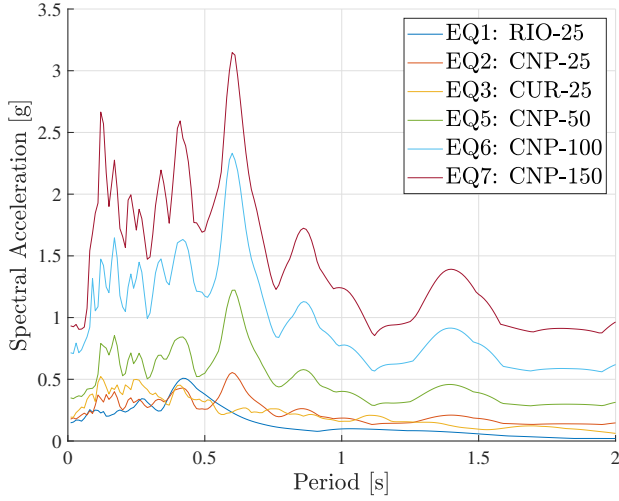


Figure 9: Elastic 5% damped pseudo-spectral acceleration of input ground motions.

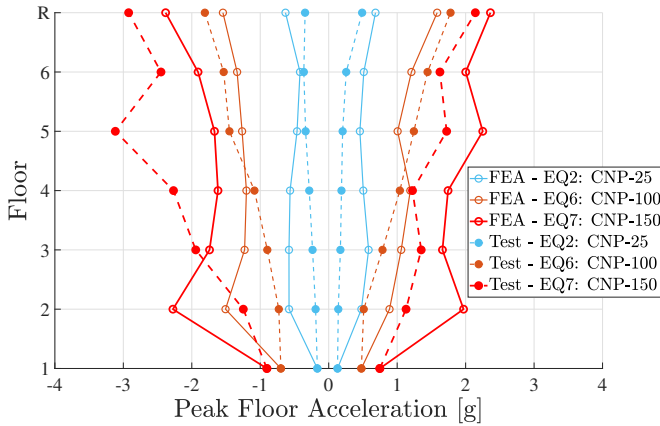


Figure 10: Comparison of peak floor acceleration (PFA) distribution.

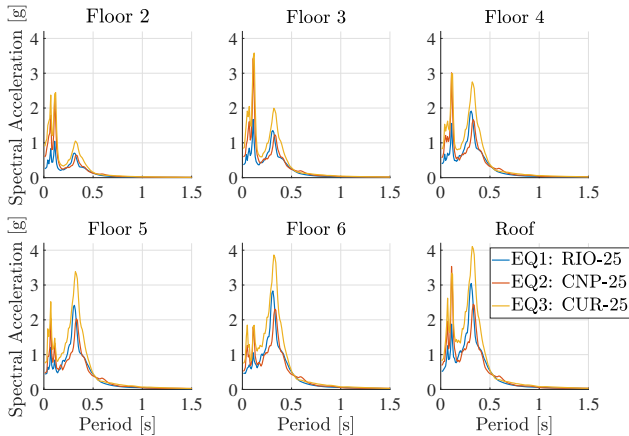


Figure 11: Floor 5% damped pseudo-acceleration response spectra for service-level earthquakes (EQ1-3).

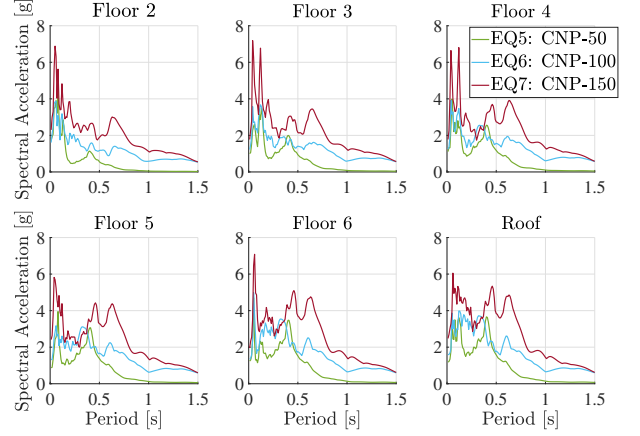


Figure 12: Floor 5% damped pseudo-acceleration response spectra for above-service-level earthquakes (EQ5-7).

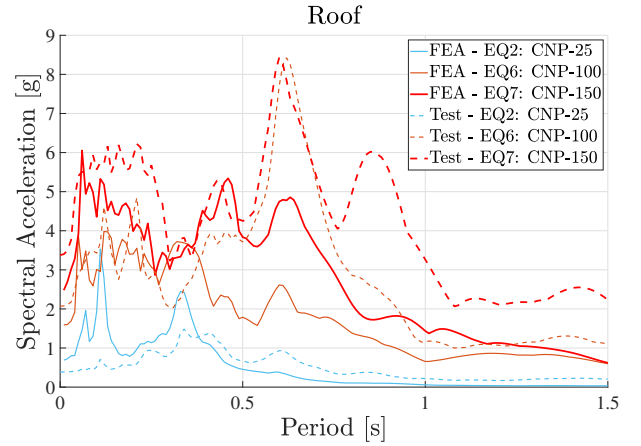


Figure 13: Comparison of roof level 5% damped pseudo-acceleration response spectra.

sequences are illustrated in Figures 11 and 12. The floor response spectra are defined as the 5% damped elastic pseudo-acceleration spectra calculated using the achieved floor acceleration as the input excitation in the longitudinal direction. It can be observed that for both the service-level earthquakes and above-service level earthquakes, the observed spectral peaks are tuned with the longitudinal periods of the building, which are about 0.3 seconds for the first longitudinal mode and about 0.1 seconds for the second longitudinal mode. The FRS peaks associated with the fundamental longitudinal period (about 0.3 seconds) increase with height in the building. These findings are consistent with those of Wang et al. [12]. However, the observed FRS peaks obtained from the model tend to have larger peak amplitudes for service-level earthquakes and smaller peak amplitudes for above-service-level earthquakes.

Figure 13 presents a comparison of the roof level 5%

damped pseudo-acceleration response spectra obtained from the numerical model and the shake table test results under three ground motions that target at service, design, and MCE level. Consistent with observations from the FRS of the service-level response, the observed spectral peaks are tuned with the longitudinal periods of the building. The spectral peaks associated with a period around 0.6 s have significantly large amplitudes obtained from the test results for both design and MCE level ground motions while the amplitudes for the same ground motion obtained from the numerical model results are relatively smaller. This might result from the fact that the building becomes less stiff with damage and thus has elongated period while the material properties in the numerical model did not consider explicitly the damaged accumulated during the test sequence.

3. Pancake Model Development and Analysis for CFS-NHERI Test Building

3.1 Model Description

Adopting a similar methodology, a numerical pancake model was developed for the aforementioned 10-story CFS-NHERI test building. The test building was designed to have four 1.83 m (6 ft) shear walls (four SW2) in the longitudinal direction and six 1.83 m (6 ft) shear walls (two SW1 and four SW3) in the transverse direction. Shear walls were designed with varying details. For example, all shear walls of level 1 used 0.838 mm (33 mil) thick steel sheet and #10 screws and were also fully blocked, but SW1, SW2, and SW3 of level 1 were designed with different minimum framing thicknesses (1.37 mm (54 mil), 2.46 mm (97 mil), and 1.37 mm (54 mil)) and different fastener spacing patterns (51 mm (2 in), 51 mm (2 in), and 102 mm (4 in)). Further details about the detailing of the shear walls can be found in the summarized narrative by Singh et al. [28]. Similarly, each shear wall segment had a pair of nodes to which a zero-length shear spring was connected. Figure 14 shows the locations of the nodes and shear springs. In total, there are 4 shear springs in the longitudinal direction (blue) and 6 shear springs in the transverse direction (green). It should be noted that the contributions of the transverse shear springs to the rotational stiffness also remain the same regardless of locations of the nodes as long as they remain along the same vertical line in plan view due to the decoupling modeling approach. Therefore, those nodes were moved to the edge as shown in Figure 14. Each rigid diaphragm then had one master node, to which a lumped mass with rotational inertia was assigned, and six slave nodes. The four slave nodes at four corners were each connected to one longitudinal shear spring and transverse shear spring. The response of the building was again similarly defined in terms of only three DOFs per floor. The building model shown in Figure 15 was produced by utilizing this overall modeling approach.

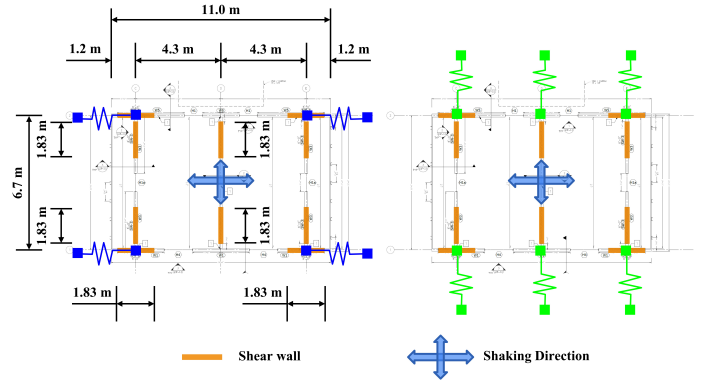


Figure 14: Zero-length nonlinear shear spring locations of 10-story CFS-NHERI model.

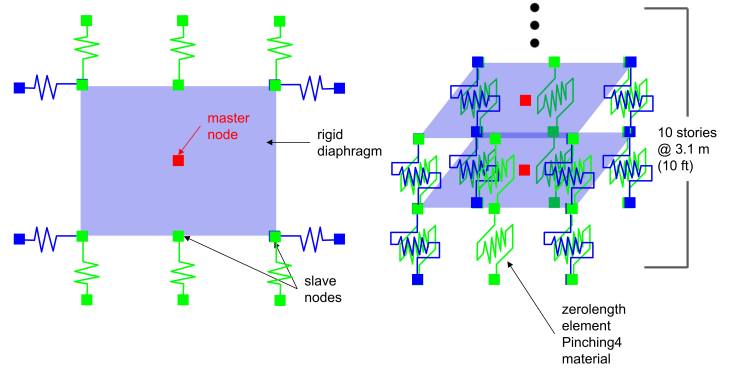


Figure 15: Conceptual representation of the 10-story CFS-NHERI building pancake model.

Determination of the shear spring material properties, as characterized using the pinching4 material, was undertaken courtesy of test results by Rizk and Rogers [32]. These experiments involved monotonic and cyclic loading of 28 specimens with 14 configurations to determine the influence of the framing thickness on the performance of the shear walls. Calibration for the pinching4 material properties was performed based on test results from W1 by Rizk and Rogers [32] (Figure 16a) and from the wall line test by Singh et al. [16]–[21], and the material properties were scaled based on geometric properties of each shear wall.

3.2 Summary of Analyses and Results

3.2.1 Eigen Analysis

Table 3 summarizes the building periods as estimations using the CFS-NHERI pancake numerical model. The 1st longitudinal mode (1-L), the 1st transverse mode (1-T), and the 1st torsional mode (1-To) are 1.32 s, 1.17 s, and 0.80 s, respectively. The design code (ASCE 7-16 [29]) based approximate fundamental period is $T_a = C_t h_n^x = 0.63$ s. A mis-

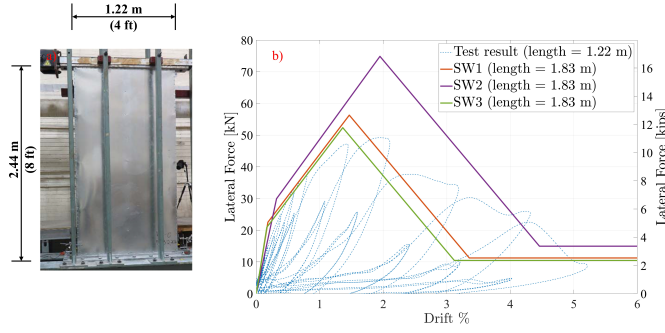


Figure 16: Pinching4 properties of level 1 shear springs for CFS-NHERI model. (Note part a photograph courtesy of Rizk and Rogers [32])

Table 3: Summary of building periods for the CFS-NHERI 10-story building model. (L = longitudinal, T = transverse, and To = torsional mode)

Mode	1-L	1-T	1-To	2-L	2-T	2-To
Period [s]	1.47	1.13	0.82	0.53	0.46	0.32
Mass Participation %	85.1	78.5	80.8	7.1	12.1	10.5

*1-L, 1-T, 1-To: 1st longitudinal, transverse, and torsional mode;
2-L, 2-T, 2-To: 2nd longitudinal, transverse, and torsional mode;

match is observed between the numerical model periods and the code-based period, indicating a more flexible behavior of the numerical pancake model. Table 3 also summarizes the mass participation ratio for each mode, which is the ratio between the effective modal mass and the total mass of the building. It is important to note that despite the tall height of the building, greater than 90% mass participation is associated with each of the first two modes in the longitudinal, transverse, and torsional direction.

3.2.2 Nonlinear Response History Analysis

Nonlinear response history analyses are performed to offer insight into the expected behavior of the 10-story CFS-NHERI building across a wide range of earthquake demand scenarios. For this purpose, motions from the FEMA P695 [33], provide two sets of ground motion records for collapse assessment. Record sets are divided into the Far-Field record set and the Near-Field record set. The Far-Field record set includes twenty-two ground motion pairs of horizontal ground motions from sites located greater than or equal to 10 km from fault rupture. This suite of twenty-two ground motion pairs were used for the nonlinear response history analysis for the 10-story CFS-NHERI test building model. The ground motion records were scaled per the FEMA P695 scaling procedure. Specifically, individual records in each set were normalized by their respective peak ground velocities and then normalized ground motions are collectively scaled to a specific ground motion intensity such that the median spectral acceleration of the record set

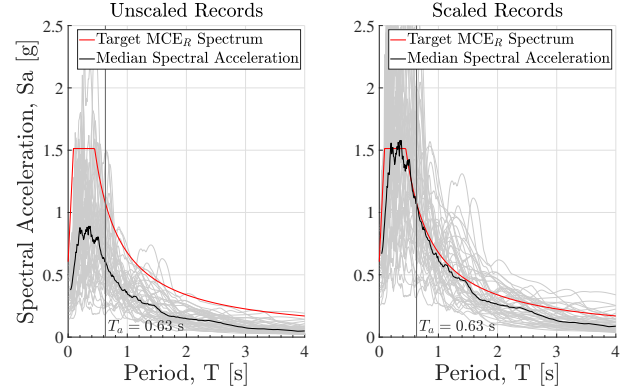


Figure 17: FEMA P695 scaling procedure [33].

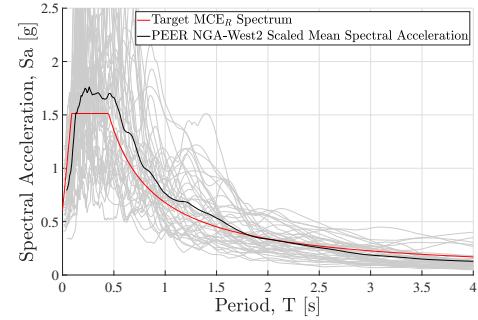


Figure 18: PEER scaled records [34].

matched the spectral acceleration at the fundamental period of the building. The scaling procedure is shown in Figure 17. Since the building design was using the code-based fundamental period (see Singh et al. [28]), the normalized ground motions were scaled using the same scaling factor such that the scaled median spectral acceleration at T_a matched the target spectrum acceleration value at T_a , where T_a is taken as the fundamental period of the 10-story test building calculated per ASCE 7-16 [29]. The scaling factor is the ratio of the target maximum considered earthquake level spectral acceleration value to the median spectral acceleration value at T_a .

The same suite of ground motions were also scaled according to the scaling procedure of the Pacific Earthquake Engineering Research Center (PEER) Enhancement of Next Generation Attenuation Relationships for Western US (NGA-West2) database [34]. Figure 18 shows the scaled ground motions according to PEER NGA-West2's scaling procedure.

Figure 19 presents a comparison between the scaled records using both the FEMA P695 and PEER NGA-West2 procedure. It is observed that the ground motions scaled per the PEER NGA-West2 scaling method have a larger mean spectral acceleration content than the ground motions scaled per

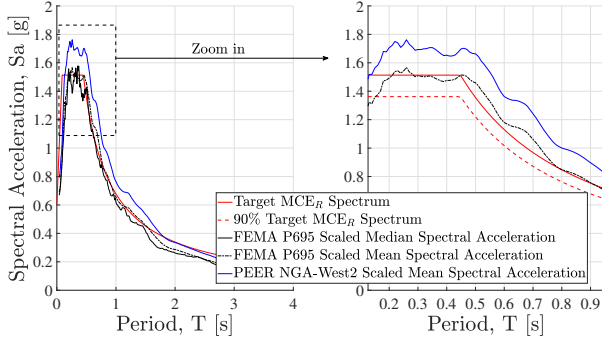


Figure 19: Comparison between FEMA P695 and PEER NGA-West2 scaled records.

FEMA P695. Note that while the FEMA p695 is using the median spectral acceleration for scaling, the mean spectral acceleration is used in Figure 19 for consistent comparison with the mean of PEER NGA-West2 scaled records. The right plot in Figure 19 shows the comparison in the period range of interest per ASCE 7-16 [29] Chapter 16. ASCE 7-16 Chapter 16 requires that the average of the spectra from all the scaled ground motions shall not fall below 90% of the target response spectrum for any period within the aforementioned period range of interest. It can be observed that the scaled ground motions per PEER scaling method satisfied this requirement while the mean spectrum of the scaled ground motions per the FEMA P695 fell below 90% of target spectrum near the lower bound of the period range.

Figure 20 presents the distribution of peak floor acceleration (PFA) for both longitudinal direction and transverse directions under ground motion pairs scaled per FEMA P695. Bi-directional ground motion pairs were imposed on the 10-story CFS-NHERI building pancake model for nonlinear response history analyses. Horizontal components within the same ground motion pair were switched with each other and then imposed again on the numerical model. In total, 44 ground motion pairs were used. Compared with the distribution for the longitudinal direction, the PFA distribution for the transverse direction have a larger mean amplitude. Larger dispersion is also observed for the PFA distribution for the transverse direction. Similar observation is found for the distribution of PFA under the suite of ground motions scaled per PEER NGA-West2, which is presented in Figure 21. In general, the PFA distributions under ground motions scaled per PEER NGA-West2 have larger amplitudes than the responses under ground motions scaled per FEMA P695.

Figures 22 and 23 present the 5% damped roof level response spectra under the same suite of ground motions scaled per FEMA P695 and PEER NGA-West2, respectively. It is observed in both figures that the response spectra have larger spectral acceleration amplitudes in the transverse di-

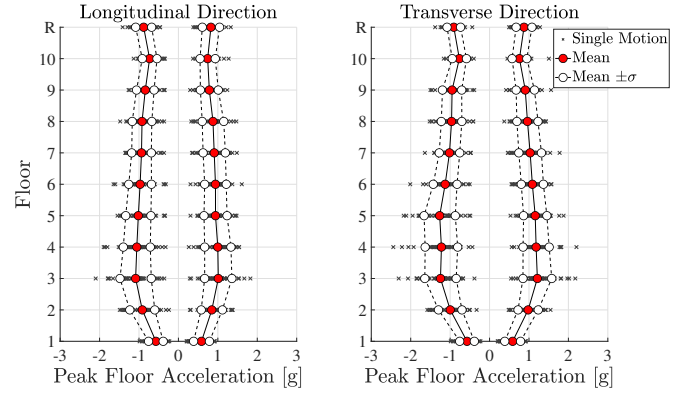


Figure 20: Peak floor acceleration (PFA) distribution under ground motion pairs scaled per FEMA P695.

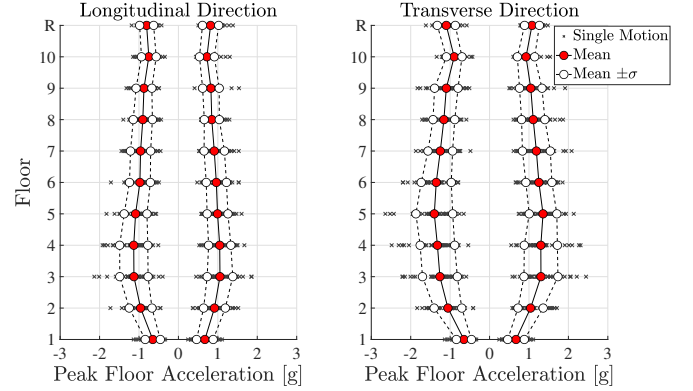


Figure 21: Peak floor acceleration (PFA) distribution under ground motion pairs scaled per PEER NGA-West2.

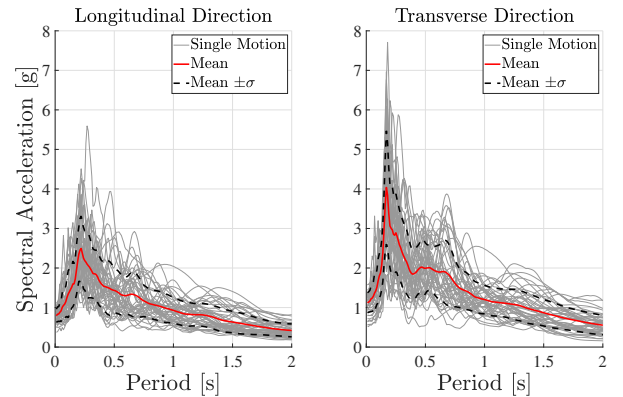


Figure 22: Roof level 5% damped pseudo-acceleration response spectra under ground motion pairs scaled per FEMA P695.

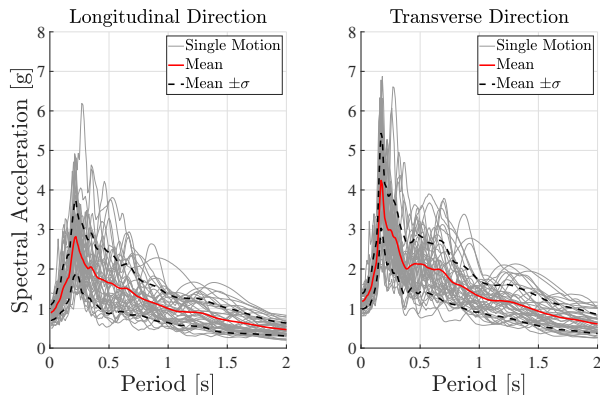


Figure 23: Roof level 5% damped pseudo-acceleration response spectra under ground motion pairs scaled per PEER NGA-West2.

rection than the longitudinal direction. The roof level response spectra under PEER NGA-West2 scaled ground motions tend to have larger peak spectral accelerations and larger standard deviation than FEMA P695 scaled ground motions. However, the peak spectral acceleration of FEMA P695 scaled ground motions is larger than that of PEER NGA-West2 scaled ground motions in the transverse direction.

4. Concluding Remarks and Outlook

The system-level benchmark test of the 10-story CFS-NHERI building at the newly upgraded LHPOST6 at NHERI@ UC San Diego Experimental Facility will provide a unique opportunity to evaluate the earthquake and post-earthquake fire performance of tall CFS-framed buildings under multi-directional earthquake excitation. During the planning of this test program, a variety of numerical models are being developed, including a simplified pancake model, first used to predict the dynamic characteristics and subsequently to simulate the nonlinear dynamic behavior of the designed specimen under different earthquake motions. To evaluate the robustness of the approach, a similar numerical model of the CFS-HUD 6-story building is also developed. Results from this model were compared with shake test results and good agreement is found. In future work, the responses of the pancake model will be compared with other numerical models being developed by fellow researchers within the CFS-NHERI team using more complex wall line representations for example, to further predict the test building's response. At present, however, the simplified pancake model provides a fast computational turnaround time, while effectively capturing the dynamic characteristics, which makes it a useful tool for extensive amount of analyses to determine the ideal suite of ground motions to be used for the shake table test. The model continues to be refined and may also be useful for incremental dynamic analysis and parametric studies in the future.

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References

- [1] B. Folz and A. Filiatrault, "Seismic analysis of wood-frame structures. I: Model formulation," *Journal of Structural Engineering*, vol. 130, no. 9, pp. 1353–1360, 2004. DOI: 10.1061/(ASCE)0733-9445(2004)130:9(1353).
- [2] B. Folz and A. Filiatrault, "Seismic analysis of wood-frame structures. II: Model implementation and verification," *Journal of Structural Engineering*, vol. 130, no. 9, pp. 1361–1370, 2004. DOI: 10.1061/(ASCE)0733-9445(2004)130:9(1361).
- [3] J. W. van de Lindt, S. Pei, H. Liu, and A. Filiatrault, "Three-dimensional seismic response of a full-scale light-frame wood building: Numerical study," *Journal of Structural Engineering*, vol. 136, no. 1, pp. 56–65, 2010. DOI: 10.1061/(ASCE)ST.1943-541X.0000086.
- [4] AISI S100-16, *North American Specification for the Design of Cold-Formed Steel Structural Members*. Washington, DC, U.S.A.: AISI, 2016.
- [5] AISI S240-15, *North American Specification for Cold-Formed Steel Structural Framing*. Washington, DC, U.S.A.: AISI, 2015.
- [6] AISI S400-15, *North American Specification for Seismic Design of Cold-Formed Steel Structural Systems*. Washington, DC, U.S.A.: AISI, 2015.
- [7] K. D. Peterman, M. J. J. Stehman, R. L. Madsen, S. G. Buonopane, N. Nakata, and B. W. Schafer, "Experimental seismic response of a full-scale cold-formed steel-framed building. I: System-level response," *Journal of Structural Engineering*, vol. 142, no. 12, 04016127, 2016. DOI: 10.1061/(ASCE)ST.1943-541X.0001577.

- [8] K. D. Peterman, M. J. J. Stehman, R. L. Madsen, S. G. Buonopane, N. Nakata, and B. W. Schafer, "Experimental seismic response of a full-scale cold-formed steel-framed building. II: Subsystem-level response," *Journal of Structural Engineering*, vol. 142, no. 12, 04016128, 2016. DOI: 10.1061/(ASCE)ST.1943-541X.0001578.
- [9] K. D. Peterman, "Behavior of full-scale cold-formed steel building under seismic excitations," Ph.D. dissertation, Johns Hopkins University, Baltimore, MD, 2014.
- [10] T. C. Hutchinson, X. Wang, G. Hegemier, P. Kamath, and B. Meacham, "Earthquake and postearthquake fire testing of a midrise cold-formed steel-framed building. I: Building response and physical damage," *Journal of Structural Engineering*, vol. 147, no. 9, 04021125, 2021. DOI: 10.1061/(ASCE)ST.1943-541X.0003097.
- [11] X. Wang and T. C. Hutchinson, "Earthquake and postearthquake fire testing of a midrise cold-formed steel-framed building. II: Shear wall behavior and design implications," *Journal of Structural Engineering*, vol. 147, no. 9, 04021126, 2021. DOI: 10.1061/(ASCE)ST.1943-541X.0003098.
- [12] X. Wang, T. C. Hutchinson, G. Hegemeir, S. Gunisetty, P. Kamath, and B. Meacham, "Earthquake and post-earthquake fire performance of a mid-rise cold-formed steel framed building—Test program and test results: Final Report (CFS Test Program Report #2)," University of California San Diego, La Jolla, CA, Structural Systems Research Project SSRP-16/08, 2016.
- [13] F. McKenna, G. L. Fenves, and M. H. Scott, *Open System for Earthquake Engineering Simulation*, <http://opensees.berkeley.edu>: University of California, Berkeley, 2000.
- [14] F. McKenna, M. H. Scott, and G. L. Fenves, "Nonlinear finite-element analysis software architecture using object composition," *Journal of Computing in Civil Engineering*, vol. 24, no. 1, pp. 95–107, 2010. DOI: 10.1061/(ASCE)CP.1943-5487.0000002.
- [15] M. Zhu, F. McKenna, and M. H. Scott, "OpenSeesPy: Python library for the OpenSees finite element framework," *SoftwareX*, vol. 7, pp. 6–11, 2018. DOI: <https://doi.org/10.1016/j.softx.2017.10.009>.
- [16] A. Singh, X. Wang, Z. Zhang, F. Derveni, H. Castaneda, K. D. Peterman, B. W. Schafer, and T. C. Hutchinson, "Steel sheet sheathed cold-formed steel framed in-line wall systems. I: Impact of structural detailing," *Journal of Structural Engineering*, 2022, in press.
- [17] A. Singh, X. Wang, Z. Zhang, F. Derveni, H. Castaneda, K. D. Peterman, B. W. Schafer, and T. C. Hutchinson, "Steel sheet sheathed cold-formed steel framed in-line wall systems. II: Impact of nonstructural detailing," *Journal of Structural Engineering*, 2022, in press.
- [18] A. Singh and T. C. Hutchinson, "Lateral response of cold-formed steel framed steel sheathed in-line wall systems detailed for mid-rise buildings. Part I: Shake table test phase," University of California San Diego, La Jolla, CA, Structural Systems Research Project SSRP-19/05, 2021.
- [19] A. Singh and T. C. Hutchinson, "Lateral response of cold-formed steel framed steel sheathed in-line wall systems detailed for mid-rise buildings. Part II: Quasi-static test phase," University of California San Diego, La Jolla, CA, Structural Systems Research Project SSRP-19/06, 2022.
- [20] A. Singh, T. C. Hutchinson, X. Wang, Z. Zhang, B. W. Schafer, F. Derveni, H. Castaneda, and K. D. Peterman, "Wall line tests: Phase 1 – Shake table tests," in *CFS-NHERI: Seismic Resiliency of Repetitively Framed Mid-Rise Cold-Formed Steel Buildings*, DesignSafe-CI, 2021. DOI: 10.17603/ds2-mvj8-8386.
- [21] A. Singh, T. C. Hutchinson, X. Wang, Z. Zhang, B. W. Schafer, F. Derveni, H. Castaneda, and K. D. Peterman, "Wall line tests: Phase 2 – Quasi-static tests," in *CFS-NHERI: Seismic Resiliency of Repetitively Framed Mid-Rise Cold-Formed Steel Buildings*, DesignSafe-CI, 2022. DOI: 10.17603/ds2-encf-vj32.
- [22] Z. Zhang, A. Singh, F. Derveni, S. Torabian, K. D. Peterman, T. C. Hutchinson, and B. W. Schafer, "Cyclic experiments on isolated steel sheet connections for CFS framed steel sheet sheathed shear walls with new configurations," *Engineering Structures*, vol. 244, 112805, 2021. DOI: 10.1016/j.engstruct.2021.112805.
- [23] Z. Zhang, A. Singh, F. Derveni, S. Torabian, K. D. Peterman, T. C. Hutchinson, and B. W. Schafer, "Cyclic experiments on steel sheet connections for standard CFS framed steel sheet sheathed shear walls," *Journal of Structural Engineering*, vol. 148, no. 2, 04021261, 2022. DOI: 10.1061/(ASCE)ST.1943-541X.0003233.
- [24] Z. Zhang, S. Torabian, A. Singh, F. Derveni, K. D. Peterman, T. C. Hutchinson, and B. W. Schafer, "Cyclic experiments on isolated steel sheet connections for CFS framed steel sheet sheathed shear walls," in *CFS-NHERI: Seismic Resiliency of Repetitively Framed Mid-Rise Cold-Formed Steel Buildings*, DesignSafe-CI, 2021. DOI: 10.17603/ds2-y1wv-yz51.

- [25] H. Castaneda, "Seismic performance of cold-formed steel diaphragms," Ph.D. dissertation, University of Massachusetts, Amherst, MA, 2022.
- [26] L. Van Den Einde, J. P. Conte, J. I. Restrepo, R. Bustamante, M. Halvorson, T. C. Hutchinson, C.-T. Lai, K. Lotfizadeh, J. E. Luco, M. L. Morrison, G. Mosqueda, M. Nemeth, O. Ozcelik, S. Restrepo, A. Rodriguez, P. B. Shing, B. Thoen, and G. Tsampras, "NHERI@UC San Diego 6-DOF Large High-Performance Outdoor Shake Table Facility," *Frontiers in Built Environment*, vol. 6, 580333, 2021. DOI: 10.3389/fbuil.2020.580333.
- [27] ASCE/SEI 7-22, *Minimum Design Loads for Buildings and Other Structures*. Reston, VA, U.S.A.: ASCE/SEI, 2022.
- [28] A. Singh, T. Hutchinson, S. Torabian, B. Schafer, K. PETERMAN, L. Padgett, and H. Jones, "Structural design narrative of the CFS-NHERI 10-story test building for multi-dimensional shake table testing," in *Proceedings of the Cold-Formed Steel Research Consortium Colloquium*, 17-19 October, 2022.
- [29] ASCE/SEI 7-16, *Minimum Design Loads for Buildings and Other Structures*. Reston, VA, U.S.A.: ASCE/SEI, 2016.
- [30] M. S. Hoehler and C. M. Smith, "Influence of fire on the lateral load capacity of steel-sheathed cold-formed steel shear walls - report of test," National Institute of Standards and Technology, Gaithersburg, MD, NIST Interagency/Internal Report (NISTIR) 8160, 2016. DOI: 10.6028/NIST.IR.8160.
- [31] L. N. Lowes, N. Mitra, and A. Altoontash, "A beam-column joint model for simulating the earthquake response of reinforced concrete frames," Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, PEER Report 2003-10, 2003.
- [32] R. Rizk, "Higher strength cold-formed steel framed / steel shear walls for mid-rise construction," McGill University, Montreal, Quebec, Canada, Research Report, 2017.
- [33] FEMA P695, *Quantification of Building Seismic Performance Factors*. Washington, DC: Federal Emergency Management Agency, 2009.
- [34] PEER, *PEER NGA-West2 Database*, <https://ngawest2.berkeley.edu/>: Pacific Earthquake Engineering Research Center, University of California, Berkeley.