Chapter 9
Evolution of Modal Characteristics and Model Comparison for a 10-Story Cold-Formed Steel Framed Building under Seismic Excitation

Jiachen (Charlie) Zhang, Shokrullah Sorosh, Amanpreet Singh, Benjamin W. Schafer, and Tara C. Hutchinson

Abstract A 10-story cold-formed steel-framed building, coined CFS10, was constructed and tested at the 6-DOF Large High-Performance Outdoor Shake Table (LHPOST6) at UC San Diego in the summer of 2025. Seismic tests targeted Service-Level Earthquake (SLE), Design Earthquake (DE) and risk-targeted Maximum-Considered Earthquake (MCE_R) hazards and were followed by live fire compartment tests. This landmark full-scale building specimen was designed beyond current code limits with advances in steel sheet sheathed shear walls and heavy chord stud details proofed in complementary component test programs, while also incorporating a wide range of realistic nonstructural systems (stairs, fire sprinkler, gas piping, glazing, etc.). The shake table testing captures the accumulation of damage to the building and its nonstructural elements during realistic seismic scenarios. In preparation for predicting building performance and to aid in ground motion selection and test protocols, a practice-oriented, phenomenological finite element (FE) model was developed. This model aimed for efficient simulation run speeds, with only essential degrees-of-freedom defined, while still seeking to capture the salient features of the specimen's behavior to earthquake excitation. This paper discusses the modal characteristics of the CFS10 building and how those characteristics evolve as damage accumulates during the seismic testing phase. Importantly, a preliminary comparison is made between the experimentally extracted modal characteristics, which evolve during the test program, and the aforementioned FE model predictions.

Keywords Cold-formed steel · Shake table test · Modal characteristics · Damage accumulation · Numerical model predictions

Introduction

As a capstone to the National Science Foundation (NSF) primary sponsored project on cold-formed steel (CFS) framed buildings, a full-scale 10-story CFS building, coined CFS10, was tested in the summer of 2025 at the 6-degree-of-freedom Large High-Performance Outdoor Shake Table (LHPOST6) [1] at the University of California San Diego. The landmark CFS10 building specimen was designed to above-code height limits with 10-stories and a total height of 30.5 m and a floor plan measuring 6.86 × 11.58 m, see Fig 1. 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 [2] height limit of 19.8 m, or 6 stories. A detailed discussion on the seismic performance of the building is provided in Hutchinson et al. [3]. A summary of the design details of the structural system of this building is provided in Singh et al. [4]. In addition, this building was outfitted with various nonstructural components and systems, including a resilient stair system [5, 6], a fire riser and sprinkler system, suspended ceiling [7], gas piping system, windows, doors, and rooftop equipment [8].

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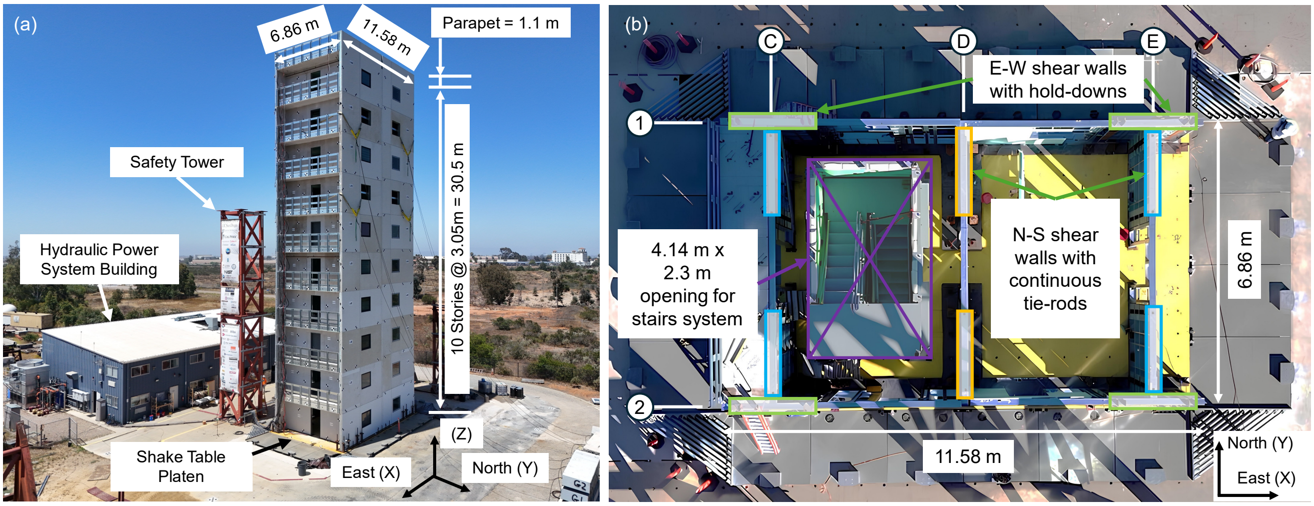

Fig. 1 CFS10 building (a) isometric view and (b) plan view

Seismic Test Protocol

Table 1 presents the seismic test protocol for the CFS10 program. A detailed discussion on the development of the seismic test protocol is provided in Zhang et al. [9]. Low-amplitude white noise (WN) tests were conducted between input ground motions to capture the evolution of the building's dynamic characteristics. Fig 2 presents the details of the test days, including WN and seismic excitations in chronological order. In total, 18 ground motions, including 15 targeting Service-Level Earthquake (SLE), one targeting Design Earthquake (DE), and two targeting Risk-Targeted Maximum-Considered Earthquake (MCE_R) hazards, as well as 37 WNs were applied to the test specimen over a duration of 7 test days.

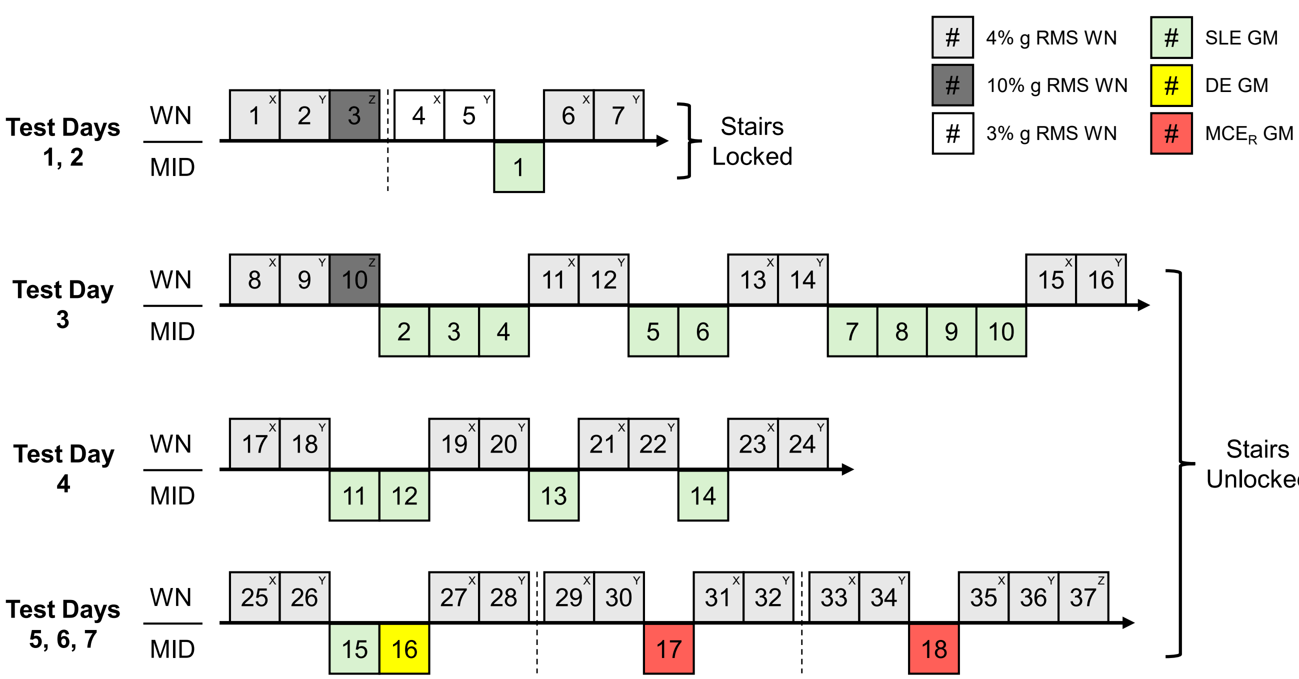

Fig. 2 CFS10 seismic test days in chronological order

Table 1 CFS10 seismic test protocol

Hazard level	MID	Earthquake event: Name, location	Year	M_w	Station	Input DOF	Components X	Components Y	$D_{5\text{-}95}$ [sec]	PIA_x [g]	PIA_y [g]	PIA_z [g]
SLE	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
	11**	Kahramanmaras, 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	Kahramanmaras, 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
$D_{5\text{-}95}$ = 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

Description of the Practice-oriented Finite Element Model

In preparation for predicting building performance and to aid in ground motion selection and test protocols, a pre-test practice-oriented, phenomenological finite element (FE) model, as shown in Fig 3, was developed. This model was developed with only essential degrees of freedom to facilitate efficient simulation run times, while also aiming to capture the salient features of the specimen's behavior to earthquake excitation. Shear walls and gravity walls were modeled to match full hysteretic behavior with both backbone degradation and pinching. A detailed description of this FE model, which was developed in OpenSeesPy [10], can be found in Zhang et al. [11]. In addition to the features presented in [11], zero-length shear springs that reflect the stiffness of the stairs were added to the center of the stairs to model the additional stiffness and strength contributions from the stairs. The details of modeling the stairs system can be found in [5].

FE Model Eigenvalue Analyses Considering Sequential Run-style Analysis

As-achieved acceleration histories of the input motions during the "stairs unlocked" configuration (i.e., MID2 through MID18) were used as ground motion inputs for the FE model and sequentially run to replicate the actual sequence of how these motions were applied to the building. That is, the model state was preserved following individual nonlinear time

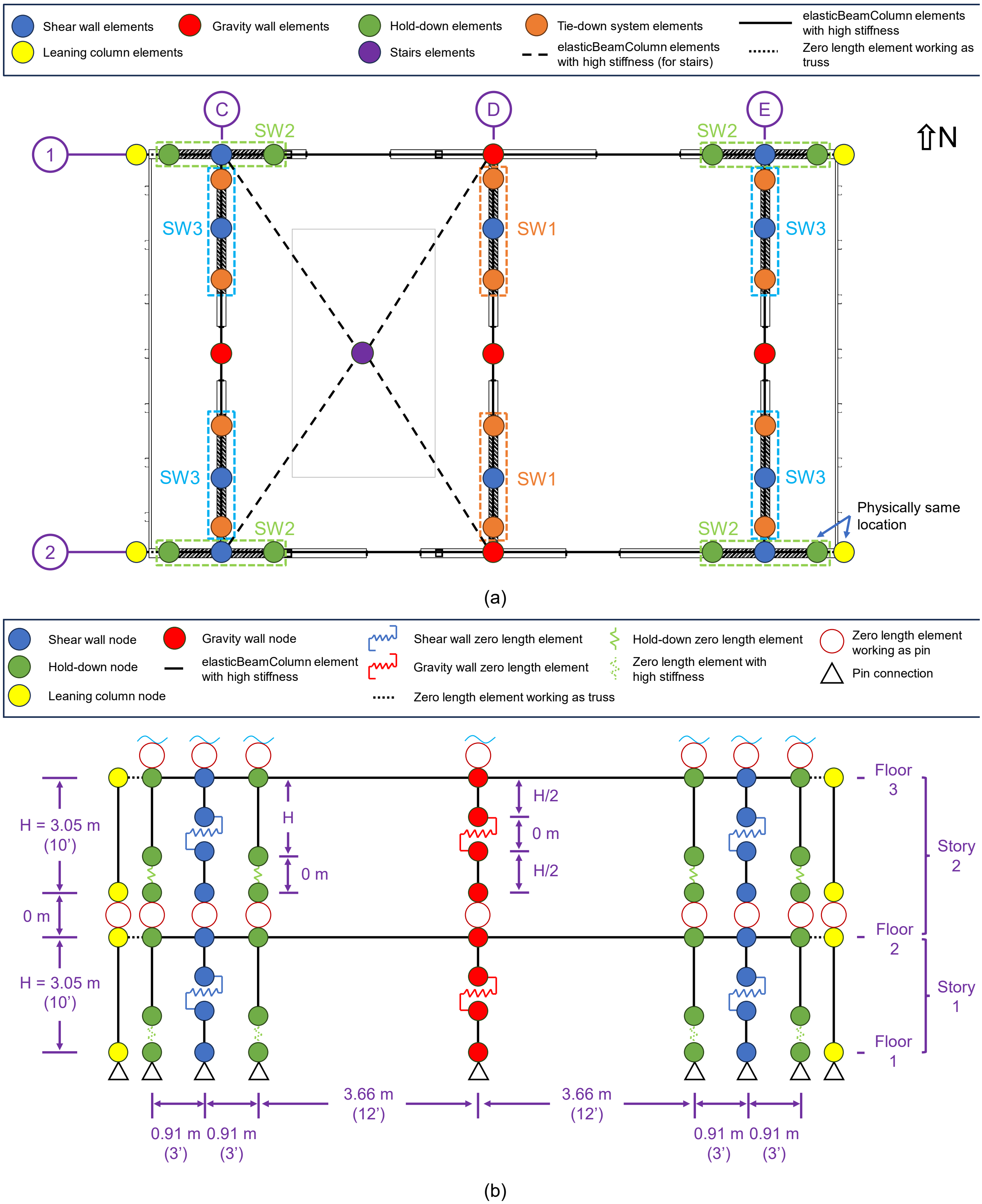

Fig. 3 Schematic (a) plan view and (b) representative elevation view of wall lines 1 and 2 of FE model

history analyses, allowing the accumulated damage to be captured through the modeling of the shear walls and gravity walls. Following each ground motion run, an additional 30 seconds were introduced in the numerical model to allow free vibration of the building model to dampen out prior to performing an eigenvalue analysis. The eigenvalue analyses were performed with the aim of replicating the system identification process, which during testing was achieved using low-amplitude unidirectional white noise tests, following seismic excitations, see Fig 2. Fig 4 summarizes the building modal period evolution for the first three fundamental modes, i.e., 1st longitudinal (1-X), 1st transverse (1-Y), and 1st torsional (1-θ). These analyses capture the general gradual increase in periods across the primary modes anticipated during sequentially increasing earthquake demand, with the largest increases, as anticipated, during the MCE_R simulated MID17 and MID18.

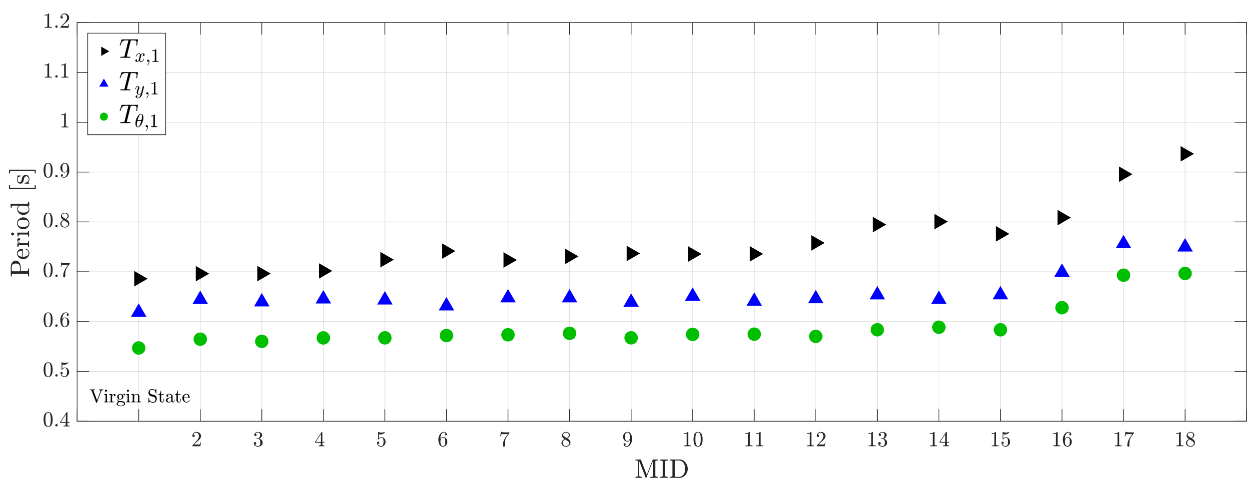

Fig. 4 Building modal period evolution based on FE model eigenvalue analyses results

Experimental System Identification (SI) Results

Using WN test data, the building period was estimated using the Stochastic Subspace Identification (SSID) technique [12]. In this analysis, 40 channels of acceleration time series data recorded at the estimated center of mass and the northwest corner of the building at each floor, and four channels of acceleration data on the shake table platen were used. In the identification of the period corresponding to the flexural-dominant (X-direction) and torsional modes, the data from WN tests running in the X-direction were used. Similarly, to determine the period of the flexural-dominant (Y-direction) mode, the data from the WN tests running in the Y-direction were used. Fig 5 summarizes the building's periods corresponding to the first three vibration

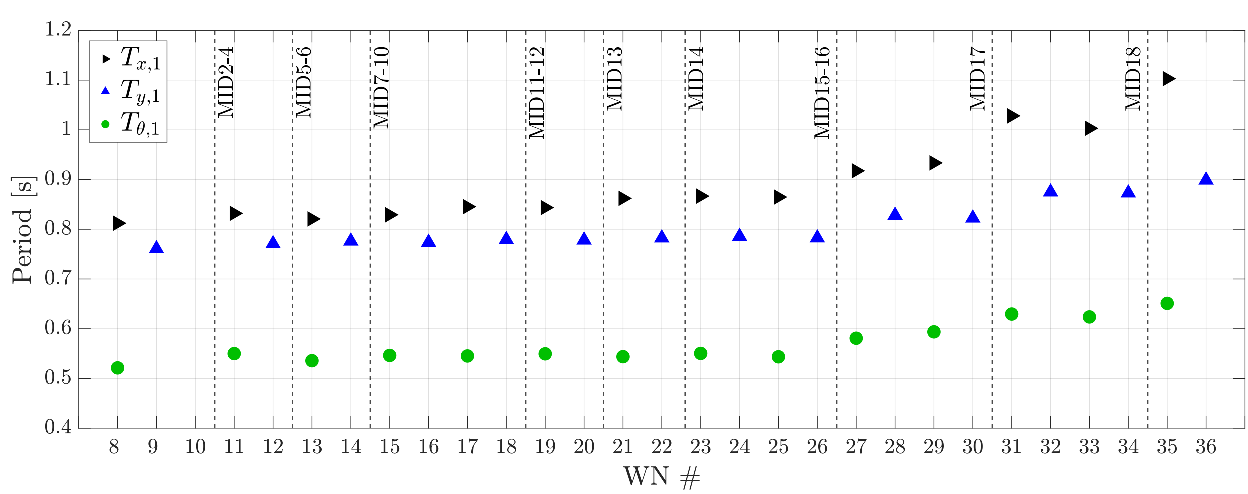

Fig. 5 Building modal period evolution based on experimental SI results

modes throughout the "stairs unlocked" configuration test phase. Interested readers may find useful the preliminary period evolution results throughout the entire test sequence documented in [3] and the detailed discussion on the impact of stair locking on the vibration period of the building in [6]. These analyses, similar to the FE model, capture the generally increasing amplitude trend in the primary modes, with larger amplitude increases following MCE_R excitations of MID17 and MID18.

FE Model Eigenvalue Analyses and Experimental SI Results Comparison

Figs. 6 and 7 compare the evolution of building periods as predicted by the sequential FE model eigenvalue analyses and experimentally determined SI results, shown as absolute values (Fig 6) and percentage change (Fig 7). It can be observed that the FE model consistently predicts smaller periods than the SI-based building periods for both the longitudinal (X) and transverse (Y) modes, reflecting stiffer lateral stiffness assumptions in the FE model, whereas the FE model consistently produces larger periods than the SI-based results for the torsional (θ) modes. The deviations between the FE-model-based results and SI-based results are much smaller for the torsional mode than the translational modes.

Expressing the period evolution as percentage change results in improved comparison between the FE-model-based and SI-based results. Generally, the FE-model-based results produce larger percentage change than the SI-based results, although the difference remains within 10%. In the longitudinal (X) direction, the percentage change after MID4 (using WN11 for

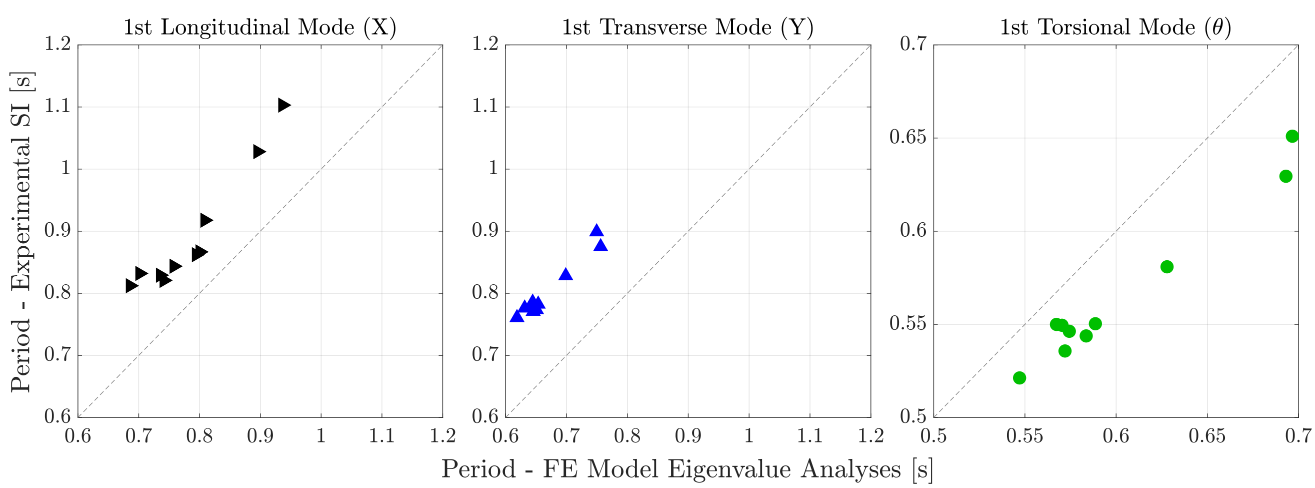

Fig. 6 Period evolution comparison between experimental SI-based and FE-model-based results

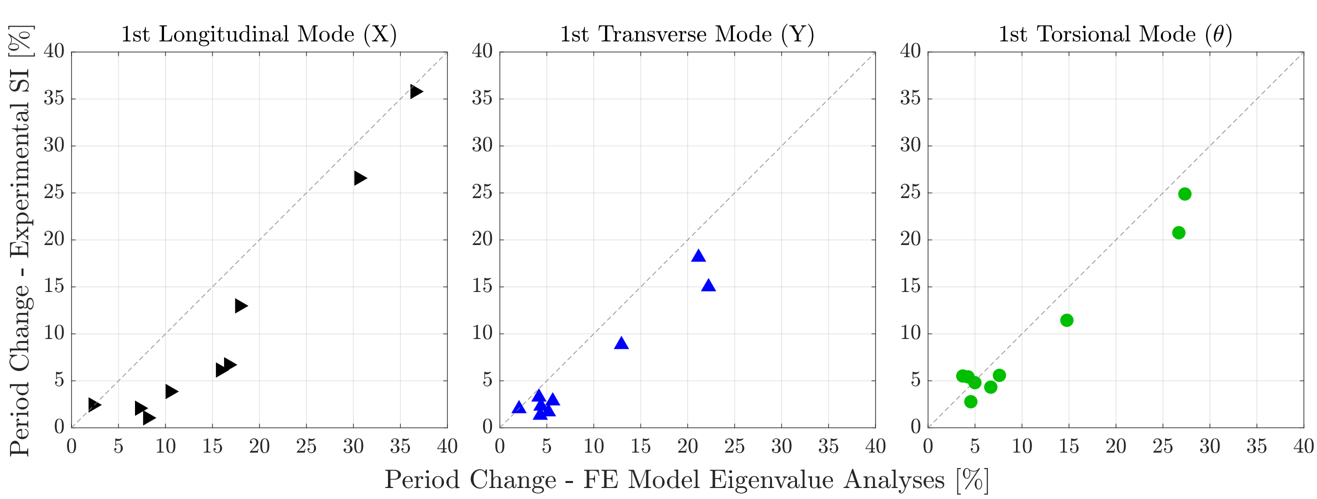

Fig. 7 Percent change in period, compared with virgin state, as estimated using experimental data (SI-based, WN) and numerical data (FE-model-based, eigenvalue analyses)

SI) and accumulated percentage change after MID18 (using WN35 for SI) were almost the same, indicating a very robust performance of the FE model in terms of capturing accumulated damage. In the transverse (Y) direction, the percentage change after MID6 (using WN14) was almost identical, while the difference between the FE-model-based results and SI-based results diverges as the motion intensities increase.

Concluding remarks

This paper examines the evolution of modal characteristics of the CFS10 building as damage accumulates under seismic excitations with increasing intensity. A preliminary comparison is presented between the experimentally extracted modal characteristics, obtained from the utilization of the extensive instrumentation of the test specimen, and predictions from a phenomenological FE model. Such comparison indicates that while the current FE model yields larger lateral stiffness assumptions, which signals the need for further study of uncertainties in material properties definitions in the initial development of the FE model as well as considering as-built conditions of the building, the FE model can reasonably capture the accumulated damage in terms of the relative increase in the fundamental building periods.

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