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From Service Level to Maximum Considered Earthquake: Evaluating the Seismic Response of a Ten-Story Cold-Formed Steel-Framed Building

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Abstract

A high-fidelity nonlinear numerical model was developed in OpenSeesPy for the CFS10, a full-scale ten-story cold-formed steel (CFS) framed building tested at the University of California San Diego's Large High-Performance Outdoor Shake Table (LHPOST6), in June 2025, under 18 multi-directional ground motions of increasing intensity. The model captures material and geometric nonlinearities across structural and nonstructural components. Buckling of framing members is taken into consideration using material definitions that account for the interaction of yielding with local and distortional buckling. Shear wall sheathing hysteretic behavior, including pinching effects and strength and stiffness degradation, is captured through models calibrated against component-level data from the cold-formed steel-framed shear wall test database. Tie-rod anchorage systems, hold-downs, and the lateral resistance of nonstructural finishes are also accounted for. The model is validated against the CFS10 experimental measurements, with close agreement observed across global and local response metrics. Using the validated model, the ten-story building response is evaluated under the 1994 Northridge Canoga Park–Topanga Can ground motion record scaled to Service Level Earthquake (SLE), Design Earthquake (DE), and Risk-Targeted Maximum Considered Earthquake (MCE_R) intensities. Interstory drift ratios and story shear forces are compared across the three hazard levels, illustrating the progression of nonlinear behavior with increasing earthquake intensity in taller CFS-framed buildings.

1. Introduction

In June 2025 a full-scale ten-story CFS-framed building was subjected to a sequence of multi-directional shake table tests at the Large High-Performance Outdoor Shake Table (LHPOST6) at the NHERI@UC San Diego test facility [1]. The program, designated CFS10, provided an opportunity to observe the seismic behavior of both the structural system and an extensive suite of nonstructural components in a CFS building whose height exceeds the limits presently permitted by U.S. seismic provisions [2,3]. The specimen was driven by 18 multi-directional ground motions of progressively increasing intensity, yielding system-level data at full scale that supports the advancement of seismic design provisions and performance-based design procedures for CFS construction. The design of the specimen is documented in [4], and an overview of the test

program together with preliminary observations is given by Hutchinson et al. [5].

A high-fidelity numerical model of the specimen was built in OpenSeesPy [6,7] in advance of testing. Different structural member is represented in that model (studs, tracks, sheathing panels, hold-downs, and tie-rods) and its modeling assumptions were established from component-level test data. Predictions obtained with the model, and the stability assumptions underlying it, have been reported previously [13,14]; those studies presented pre-test estimates, since measured response was not yet available. In the present paper, the model is used (after validation against CFS10 test results) to examine how the ten-story building responds as the intensity of a single record, the 1994 Northridge Canoga Park–Topanga Can record, is raised from SLE through DE to MCE_R. Floor acceleration,

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interstory drift ratios, and story shear forces are examined at each level and compared across different hazard levels.

2. Structural Configuration of the CFS10 Test Specimen

Fig. 1(a,b) shows the specimen. Its design assumed a hypothetical high-seismicity site near Irvine, California, on Site Class C soil; the governing assumptions are reported in [8]. Framing, single-sided steel sheet shear walls, and gravity wall systems make up the structure, which reproduces one portion of a larger archetype building [9]. Lateral resistance along both principal axes comes from the steel-sheet sheathed shear walls. Detailing choices in those walls sit outside what AISI S400-20 [10] presently covers: HSS members serving as chord studs through the lower stories, power-actuated fasteners (PAFs) joining sheet to HSS chord, and sheet fastener layouts not covered in the S400 strength table entries. Chord stud tension is carried by continuous tie-rods along one principal axis; along the other, splice plates connect chord studs directly. Overall height reached 100 ft (30.5 m), 35 ft higher than the 19.8 m ceiling set by ASCE 7 [2,3].

3. Numerical Modeling Approach

OpenSeesPy provided the computational environment for the high-fidelity finite element (FE) model. Studs and tracks were represented by displacement-based beam-column line elements whose axial, in-plane flexural, and out-of-plane flexural responses were assigned independently using the "section aggregator" command. Inelastic response of these members is carried by elastic-perfectly plastic (EPP) materials calibrated to nominal capacities computed from AISI S100-16 [11], accounting for local and distortional buckling along with yielding. Diagonal truss pairs using a Pinching4 (P4) material represent the steel sheet sheathing. P4 assembles its backbone from four linear segments and reproduces pinching together with stiffness and strength decay under cycling, the behaviors treated in Section 9.8 of ASCE 41-23 [12]. Backbone and degradation parameters were calibrated using component-level cyclic tests from the cold-formed steel-framed shear wall test database [16,17].

Tension anchorage was represented explicitly: the continuous tie-rod assemblies were modeled over their full height, and the discrete hold-downs were assigned axial force-deformation relationships derived from component-level data. Nonstructural finishes that contribute in-plane stiffness were also included in the model using diagonal truss pairs.

The model, which is one of the largest OpenSees models of CFS-framed systems, has over 17,000 nodes and 21,000 elements. Fig. 1(c) renders the assembled geometry. Complete documentation of the model development and validation is carried in the dissertation of the lead author.

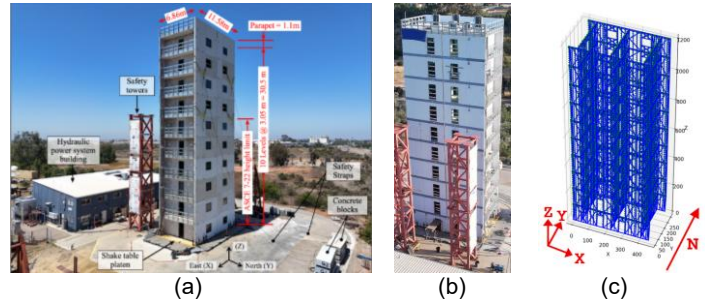


Figure 1: (a) Annotated northeast perspective of the CFS10 test structure [5], (b) southeast perspective of the CFS10 test structure, and (c) 3D visualization of the CFS10 high-fidelity model (southeast perspective) [14].

4. Seismic Response from Service-Level to MCE_R Intensity

One record, 1994 Northridge, Canoga Park - Topanga Can, was applied through the validated model at three intensities. Orientation was held fixed across all three runs: the 196° component was applied in the building X direction, the 106° component was applied in the Y direction. The three intensities are:

- Service Level Earthquake (SLE): peak ground acceleration (PGA) of 0.08g along axis X and 0.07g along axis Y;
- Design Earthquake (DE): PGA of 0.29g along axis X and 0.26g along axis Y;
- Risk-Targeted Maximum Considered Earthquake (MCE_R): PGA of 0.43g along axis X and 0.4g along axis Y.

Per ASCE 7-22, RotD100 scaling is applied to account for the influence of bi-directional motions. None of these three intensities formed part of the CFS10 test protocol; the responses below are predictions rather than reproductions. In this respect the validated model expands the response database beyond what the test sequence has delivered.

Figs. 3, 4, and 5 present the peak floor acceleration (PFA), peak interstory drift ratio (PIDR), and peak story shear force profiles computed at the three intensities; the discussion below is focused on the absolute peak of each quantity over the height of the building. At SLE those peaks are 0.28g and 0.34g for PFA along X and Y, 0.09% and 0.13% for PIDR, and 128.6 kN and 143.8 kN for story shear. These drift levels places the building firmly within its elastic range at this hazard level.

At DE, the absolute peaks rise to 0.91g and 1.15g for PFA, 0.49% and 0.44% for PIDR, and 457.6 kN and 418.3 kN for story shear. Measured against SLE, peak drift along X increases by a factor of 5.4 while peak story shear increases by 3.6; along Y the corresponding factors are 3.4 and 2.9.

At MCE_R , the absolute peaks reach 1.80g and 1.61g for PFA along X and Y, 0.91% and 0.74% for PIDR, and 605.8 kN and 554.4 kN for story shear. Relative to DE, drift grows by 1.86 along X and 1.68 along Y while story shear grows by only 1.32 and 1.33, and peak floor acceleration by 1.98 and 1.40. Setting shear forces under the MCE_R -scaled motion against the design base shear (design base shear is 162 kN in both directions) gives an overstrength of 3.73 in X direction and 3.42 in Y direction.

Measured against the fragilities for steel sheet sheathed CFS walls [15], the peak MCE_R IDR of 0.91% lies above the 0.56% median for Damage State 1, the end of elastic behavior, and well below the 1.60% median for Damage State 2, which marks peak wall capacity. The shear walls have therefore left their elastic range at MCE_R while remaining far from their strength limit, in a building whose height exceeds the ASCE 7 [2,3] ceiling.

5. Conclusions

A high-fidelity FE model of a ten-story CFS-framed building was developed in OpenSees, with representation of the structural and nonstructural components and explicit treatment of material deterioration, cyclic pinching, and the buckling modes that govern CFS members. The model was validated against measurements from the CFS10 shake table program and then used to trace the response of the building under a single record scaled from SLE through DE to MCE_R intensity. The governing peak interstory drift ratio rose from 0.13% at SLE to 0.49% at DE and 0.91% at MCE_R , peak floor acceleration from 0.34g to 1.15g and 1.80g, and peak story shear force from 143.8 kN to 457.6 kN and 605.8 kN. Even at MCE_R , and even though the building exceeds the 19.8 m code height restriction, the drift demand remains well below the drift levels at which steel sheet sheathed CFS-framed shear walls reach peak capacity.

The three intensity levels examined here are discrete points on a continuum. The same validated model is being carried forward into an incremental dynamic analysis, in which the record set is scaled progressively to collapse so that the collapse margin ratio of the ten-story building can be quantified within the FEMA P-695 framework and weighed against the height limit that presently governs CFS-framed construction.

6. Acknowledgments

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conclusions are those of the authors and do not necessarily reflect those of the sponsoring organizations.

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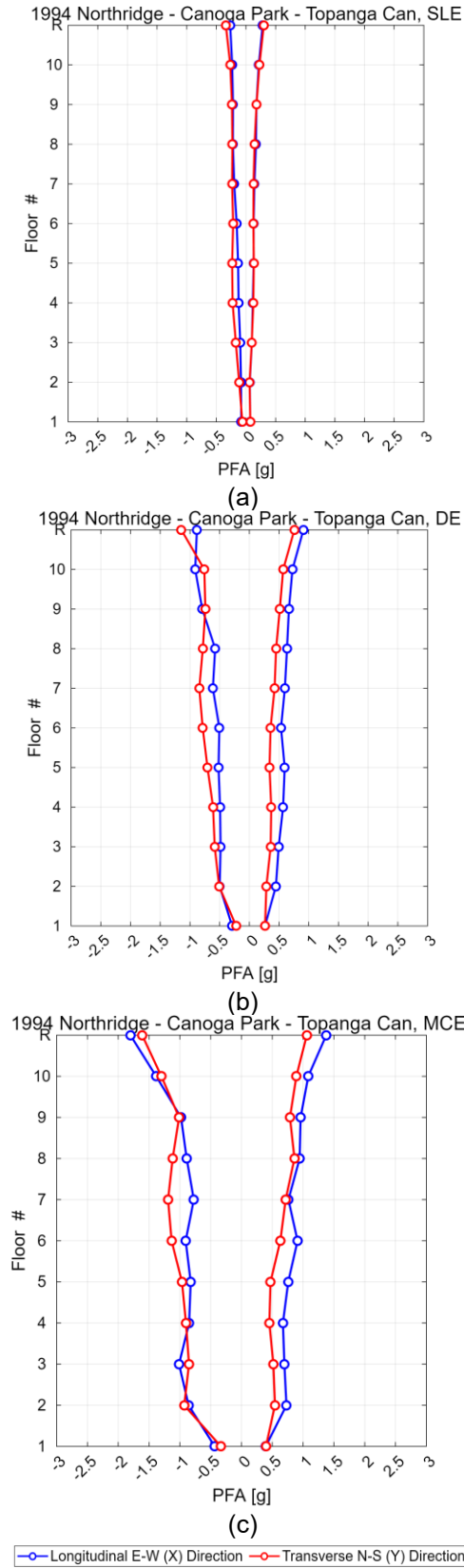


Figure 2: Peak floor acceleration profiles along both principal building axes at (a) SLE, (b) DE, and (c) MCER intensity, computed with the validated model for the 1994 Northridge (Canoga Park - Topanga Can) record.

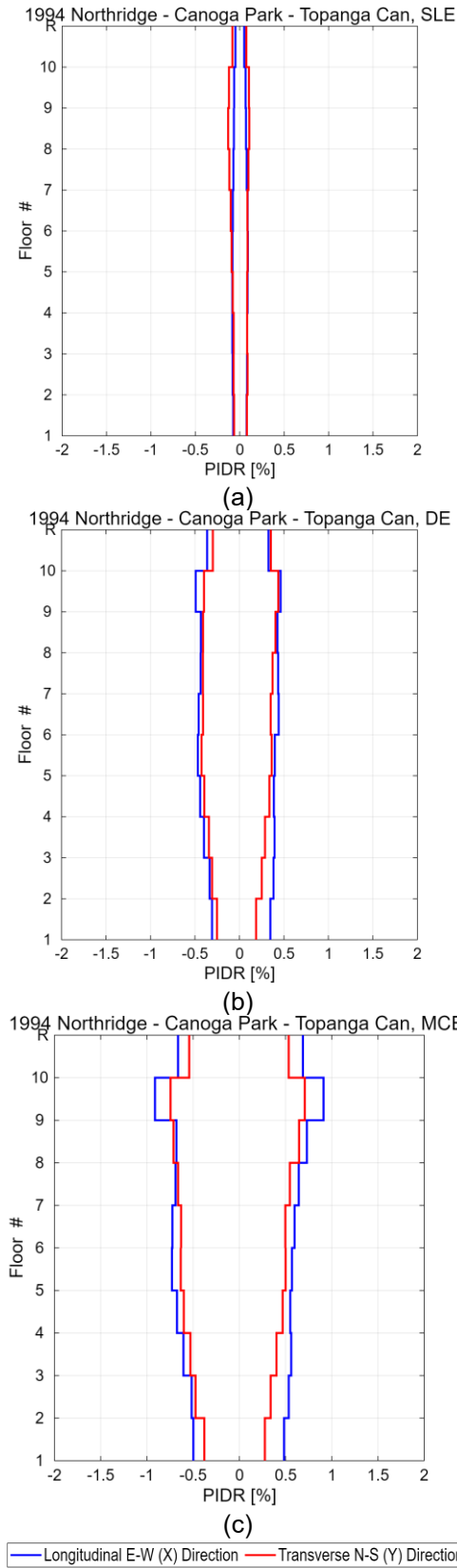


Figure 3: Peak interstory drift ratio profiles along both principal building axes at (a) SLE, (b) DE, and (c) MCER intensity, computed with the validated model for the 1994 Northridge (Canoga Park - Topanga Can) record.

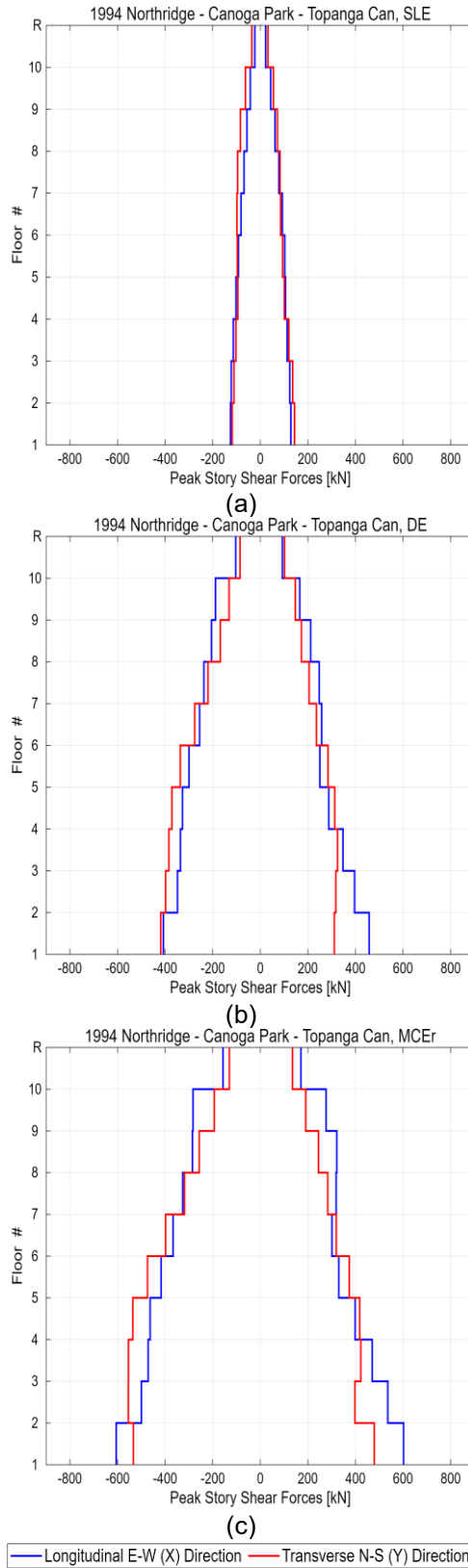


Figure 4: Peak story shear forces along both principal building axes at (a) SLE, (b) DE, and (c) MCER intensity, computed with the validated model for the 1994 Northridge (Canoga Park - Topanga Can) record.