



## HYBRID SIMULATION OF AN EXTERIOR STEEL COLUMN IN A 20-STORY MOMENT RESISTING FRAME

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### ABSTRACT

The availability of reliable numerical models is essential to reduce the uncertainties present in the prediction of structural behavior. Experimental studies allow the calibration and development of numerical models capable of characterizing the realistic behavior of structural elements and components until the limit state of collapse is approached. Quasi-static testing is the most commonly used experimental technique, in which the structural element or component is subjected to a predefined loading or displacement history. However, these loading histories are not consistent with the demand time histories experienced by structural components during an earthquake event. An alternative method for performing experiments is hybrid simulation. In hybrid simulation a structure is represented by a structural component or substructure (physical specimen) in the laboratory, while the rest of the structure is modelled in a computer (numerical model). This approach facilitates response-history analysis of the coupled system (physical and numerical) exposed to a ground motion. In this study, two hybrid simulations were conducted in which a 1:8 scaled W36X652 exterior column that is part of a 20-story steel moment resisting frame was considered as the physical substructure. The three-actuator setup (small bearing machine) at NEES@ Buffalo laboratory was utilized. The most relevant lessons learned from this study are threefold. First, the results reinforced the need to further investigate the response of deep steel columns to provide data useful for model calibration and improved numerical collapse predictions of structural systems. Second, it was demonstrated that hybrid simulations up to collapse could be very sensitive to the properties assigned to the numerical portion of the structure. Third, it was shown that the proposed hybrid substructuring technique and displacement-control approach implemented in this study were successful in tracing the behavior of a tall steel structure until the onset of global instability was approached.

### INTRODUCTION

Hybrid simulation has been conducted since the 1970's (Takanashi, 1975). Throughout the years the implementation of this testing approach has been greatly facilitated by improvements associated with its accuracy and efficiency. Hybrid testing results in the interaction between a numerical (finite element) model and experimental specimens (physical substructures) during a test. In this approach, the components of the structural system that can be modeled with a higher degree of confidence are included in the numerical model. The physical substructures are comprised of those components of the structural system that need to be studied experimentally. An advantage of hybrid simulation is that the complete structural system does not need to be constructed and tested in the laboratory (e.g., shake

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table studies). Thus, hybrid simulation is more economical, safe, and provides more flexibility to test specimens of various scales (Schellenberg et al., 2009).

In this study slow hybrid simulation was performed on a 20-story moment resisting frame structure designed for Century City, California. The experimental substructure is a 1:8 scaled column specimen corresponding to a prototype W36X652 first-story exterior column. The mass, damping, and stiffness associated with the rest of the structure were part of the numerical model. One of the objectives is to provide much needed information on the behavior of deep steel column sections exposed to lateral drift, rotation at the tip, and variable axial load demands from elastic behavior to the onset of collapse. Two hybrid simulations were conducted for two levels of ground motion intensity. The second test was performed until the limit state of collapse was approached. In the past, experiments have been performed with similar experimental setups to predict the inelastic response of reinforced concrete columns (Yamada et al., 1990) and steel box sections (Yamada et al., 1992) without necessarily approaching the limit state of collapse. The information obtained in this paper is to be used in conjunction with quasi-static tests conducted by the authors as part of the NEESR project titled *Collapse Simulation of Multi-Story Buildings Through Hybrid Testing* to calibrate numerical models of deep steel columns that account for strength and stiffness degradation in the presence of axial loads (Zargar et al., 2014).

The need for these tests arises from the scarcity of experimental data on the behavior of deep steel columns. These data are important for the calibration of numerical models of column elements necessary for a more accurate and reliable prediction of structural behavior up to the limit state of collapse. Lignos and Krawinkler (2010) developed a database of more than 300 experiments on wide flange sections and calibrated deterioration parameters for the modified Ibarra-Medina-Krawinkler hysteretic model (IMK) (2005). However, the amount of experimental data on the response of wide flange column sections was limited. Furthermore, most of the available experiments were conducted considering that columns behaved in double curvature (Newell and Uang, 2006). Forcing the column to deflect in perfect double curvature does not provide a realistic representation of the rotation associated with the column/panel zone/beam interface, and hence, may not provide a reliable representation of changes in the moment gradient along the height of the element during a response history analysis.

The hybrid simulations in this study were performed at the NEES lab @Buffalo. The Open System of Earthquake Engineering Simulation Platform software (OpenSees, 2007), OpenSees, was used for the numerical modeling and OpenFresco was used as the interface between the finite element software and the control of physical actuators and data acquisition software. OpenFresco is an object-oriented software that was developed by Takashashi and Fenves (2006) and was further modified and extended by Schellenberg (2006). In this study, the implicit Newmark method with fixed number of iterations was used for the integration scheme during the hybrid simulations. Furthermore, a predictor corrector algorithm was used to provide synchronization between the integration approach and the actuator control. A discussion on the substructuring technique used for these tests, as well as the control strategy required to impose appropriate demands at the tip of the column are also presented.

It is demonstrated herein that (a) there is a need to obtain additional information on the seismic response of deep steel columns; (b) hybrid simulations of moment frames can be very sensitive to assumed properties in the numerical modeling of columns; and (c) the proposed hybrid simulation approach was able to reliably characterize the response of the exterior steel column until the onset of global instability was approached.

## **NUMERICAL MODEL**

The prototype structure for the hybrid simulations consists of a 20-story office building with perimeter moment resisting frames located in Century City, CA. A plan view of the 20-story structure is shown in Fig. 1. This structure was designed based on ASCE 7-10 (2010) and the Steel Seismic Provisions (AISC, 2010). The exterior columns are W36X652 sections, which is the prototype structural element used in all tests. A 1:8 scaled two-dimensional model of the moment-resisting frame structure (North-South (N-S) frame) was developed using OpenSees.

Note: All units are in ft.

The height of the 1<sup>st</sup> Story is 14.5 ft. from top of the base-plate to the axis of the first floor girder and the height of the other stories is 13 ft. between the axis of two adjacent floor girders.

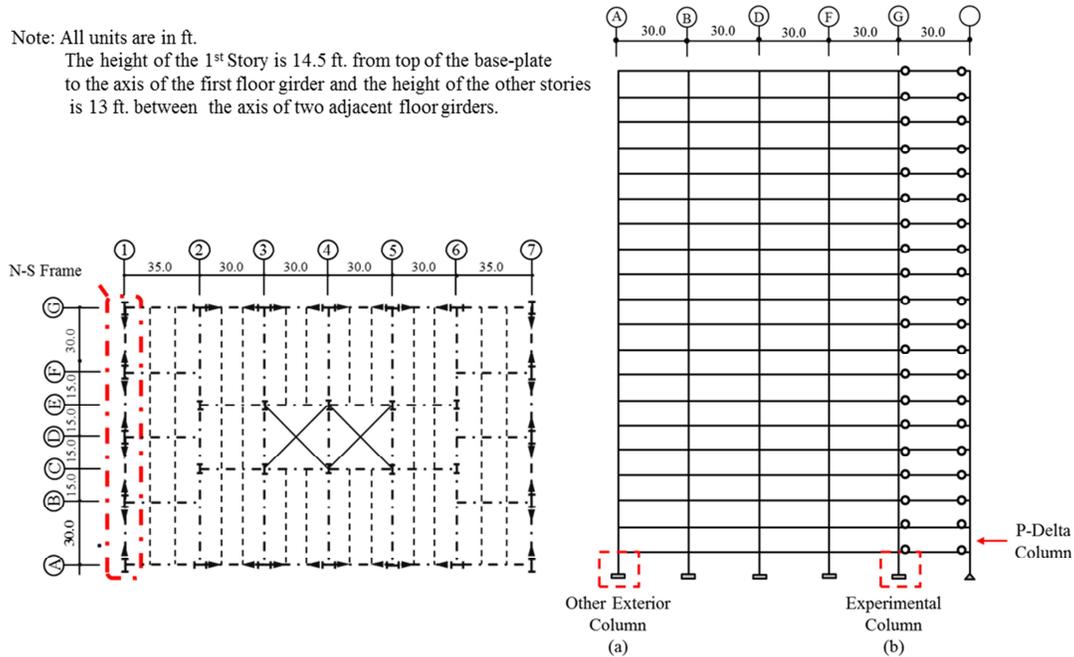


Figure 1. Typical floor plan of the 20-story building (left) and elevation view of prototype model of N-S frame (right)

The fundamental period of the scaled structural model was estimated as 1.04 s (2.93 s for the full scale prototype model). The model consisted of a combination of nonlinear rotational springs and elastic beam elements. The springs were placed at the top and bottom of the columns, as well as at the center of the reduced beam sections. The hysteretic behavior of the springs was modeled based on the modified Ibarra-Medina-Krawinkler deterioration model (Lignos and Krawinkler, 2010). The deterioration properties for the beam and column sections were calculated using the regression equations of Lignos (2010). Panel zones were modeled following the Gupta-Krawinkler approach (1999). In order to account for the P-Delta effect, a leaning column with floor gravity loads corresponding to half of the floor mass minus the tributary load of the N-S frame at each level was connected to the frame with rigid links (see Fig. 1-right). Rayleigh damping of 2% was assumed for the first and fifth period of the scaled frame. The approach proposed by Zareian and Medina (2010) was used to model damping. The bending moment strength of the column in the presence of axial loads was estimated based on the P-M interaction equations given in AISC-ANSI 360-05 (2005). To obtain the required axial load demand, the axial force of the column from a pushover analysis with a FEMA (SAC Joint Venture, 2000),  $k = 2$  (parabolic) lateral load pattern was obtained. The axial load demand was estimated from combining the factored gravity axial load in the column ( $P_{grav} = 1.05P_D + 0.25P_L$ ) with 50% of the maximum axial load ( $P_{E,max}$ ) experienced by the column due to the application of the lateral loads during the pushover analysis,  $P_r = P_{grav} + 0.5P_{E,max}$  (NIST, 2011). Nonlinear static and dynamic analyses were conducted with a set of 100 recorded horizontal ground motions (50 stations) to facilitate the selection of the ground motion used to perform the hybrid simulations. The Duzce, Turkey 1999 horizontal ground motion record (NGA no 1605 DZS 270, Duzce Station) was used to evaluate the behavior of the structure up to collapse. From here on, this ground motion record is referred to as the Duzce record.

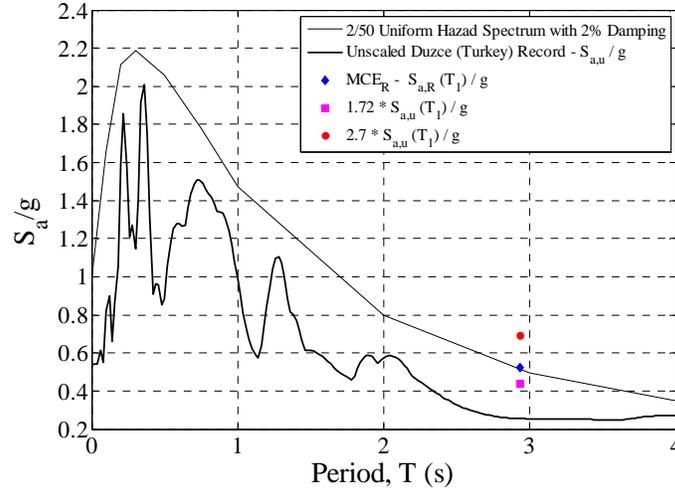


Figure 2. 2/50 uniform hazard spectrum (USGS, 2008) and response spectra for Duzce, Turkey 1999 horizontal ground motion (Duzce Station) with 2% damping

## PHYSICAL SPECIMEN

The capacity of the available actuators dictated the 1:8 scale used for the test specimen. The scaling process was focused on matching relevant parameters that control the inelastic behavior of wide flange steel elements (e.g.,  $t_f/t_w$ ,  $h/t_w$ ,  $b_f/2t_f$ ) within 10% of the target values, as shown in Table 1. The height of the experimental specimen was considered as the scaled height of the prototype column from the top of the base plate to the bottom of the panel zone region. A test specimen height equal to 19.4 in. was obtained, which corresponds to a prototype column height of 155.2 in. An average yield strength of 50 ksi and an ultimate strength of 68 ksi were estimated from six tensile coupon tests conducted with pieces of flange and web procured during fabrication of the specimens.

Table 1. Cross section specification of the prototype column and test specimen

Specifications	$b_f$ [in.]	$t_f$ [in.]	$b_f/2t_f$	$d$ [in.]	$h$ [in.]	$t_w$ [in.]	$h/t_w$	$t_f/t_w$	$I$ [in. <sup>4</sup> ]	$A$ [in. <sup>2</sup> ]	$Z$ [in. <sup>3</sup> ]
<b>W36X652</b>	17.6	3.54	2.49	41.1	32.1	1.97	16.3	1.8	50600	192	2910
<b>Test Specimen</b>	2.00	0.40	2.50	5.56	4.02	0.25	16.4	1.63	13.60	2.91	5.78

In hybrid simulation, the initial stiffness matrix should be calculated prior to the test. The system identification test was conducted and the 3x3 initial stiffness matrix of the experimental element was estimated as shown in Eq.1. The measured values were smaller than the theoretical stiffness values (Eq.2) due to the minor lateral and axial flexibilities associated with small rotations of the loading beam, as well as small deformations of the lateral yellow frames and the support pedestal (see Fig. 3).

$$k_{i,Experimental} = \begin{bmatrix} 3262 (k/in) & 0 & 0 \\ 0 & 400 (k/in) & -4948 (k.in/in) \\ 0 & -4948 (k.in/in) & 64000 (k.in) \end{bmatrix} \quad (1)$$

$$k_{i,Theoretical} = \begin{bmatrix} 4350 (k/in) & 0 & 0 \\ 0 & 648 (k/in) & -6288 (k.in/in) \\ 0 & -6288 (k.in/in) & 81320 (k.in) \end{bmatrix} \quad (2)$$

## EXPERIMENTAL SETUP AND SUBSTRUCTURING APPROACH

The three-actuator test setup shown in Fig. 3 was utilized for this experiment at the NEES@ Buffalo laboratory. This setup was used to control the two translational and rotational degrees of freedom (DOF) at the tip of the column (Fig. 4- Left). The three actuators were in displacement-control mode. The location of the control and data acquisition DOF of the experimental setup is illustrated in Fig. 4. The actuators were connected to a loading beam (orange beam in Fig. 3) in order to impose appropriate deformation demands at the tip of the column. The base of the horizontal actuator was connected to a reaction frame, while the base of the vertical actuators was connected to a support beam (red beam in Fig. 3). The test configuration provided restraints (yellow frames in Fig. 3) to minimize out-of-plane displacements at the tip of the column.

The ThreeActuatorJntOff Experimental Setup in OpenFresco was modified for the current study to relate and transform the displacement and forces between the tip of the actuators and the tip of the column. In the original code the transformation of displacements and forces occurs between the pin connection of the actuator swivels and the location on the centroidal axis of the loading beam directly above the tip of the column (Fig. 3). However, the tip of the prototype column is below the panel zone, which in the scaled specimen is located at the bottom face of the top base plate.

A coupled numerical model was generated to (a) test whether the hybrid substructuring technique was sound, (b) determine the appropriate ground motion scale factor to bring the structure to the onset of dynamic instability during the hybrid simulation, and (c) evaluate the required capacity (load cell and stroke) of each actuator. In this context, the term *coupled* refers to a model in which the physical specimen that forms part of the hybrid architecture is also modeled numerically using OpenSees. Thus, a virtual (purely numerical) hybrid simulation can be conducted with the numerical model of the rest of the structure (master) and the numerical model of the physical specimen (slave). An adapter element (Schellenberg et al., 2008) allowed coupling of the master and slave models through the connection of OpenFresco and OpenSees.



Figure 3. Experimental setup configuration.

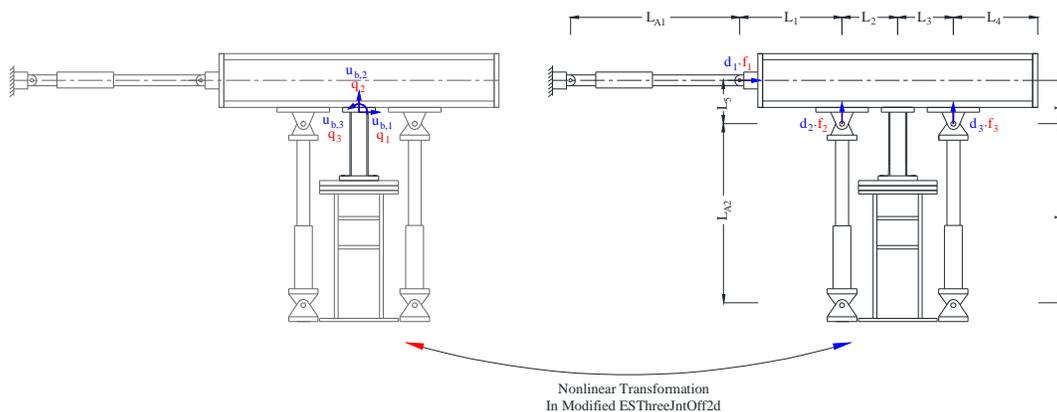


Figure 4. Transformation in the modified ThreeActuatorJntOff Experimental Setup (Schellenberg et al., 2009)

## HYBRID SIMULATION ARCHITECTURE

The architecture used in the hybrid simulations is shown in Fig. 5. A hybrid laboratory had to be designed and built as part of this study because a setup for hybrid test was not available where the three-actuator test setup (small-bearing testing machine) is situated at the NEES@ Buffalo laboratory. The OpenSees finite element software was used for the modeling and analysis of the numerical structure on a host computer. OpenFresco enables the communication between OpenSees and the hybrid test setup. The TCP/ICP sockets connect the numerical analysis machine (host) to the digital signal processor (target). The xPC experimental control object was employed to connect to the real-time xPC Target machine, which runs the event driven predictor-corrector model. Communication between the xPC Target machine and the MTS controller is provided by a National Instrument board. The MTS controller sends the command displacements to the actuators and returns back the measured forces and displacements in all actuators.

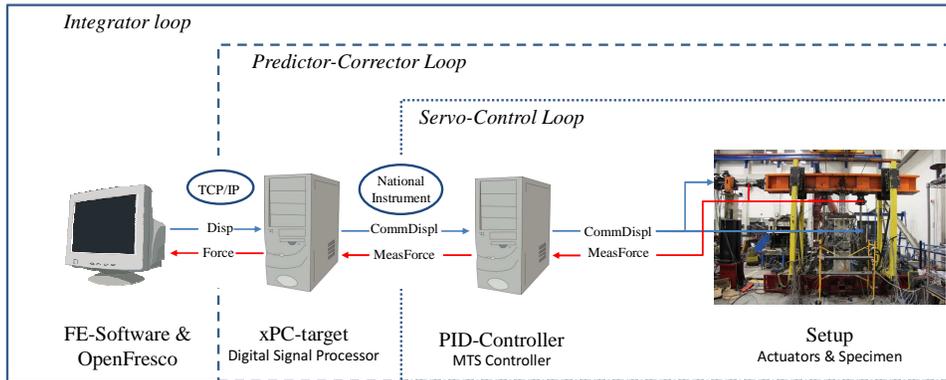


Figure 5. Architecture of the hybrid simulation

## ORIGINAL COLLAPSE TEST

Nonlinear time history analyses using the coupled model showed that the onset of dynamic instability was achieved when a scale factor of 1.72 was applied to the Duzce record. The scaled 2%-damped spectral acceleration at the first mode period of the structure was equal to 0.44g, which is less than the values corresponding to the 2%-damped 2/50 and risk-targeted maximum considered earthquake spectra at the site (both equal to 0.52g) (see Fig. 2). However, the hybrid test with this scaled ground motion had to be terminated before global collapse of the structure was attained as explained later in this section.

For this hybrid experiment, in order to numerically estimate and predict the response of the structure up to collapse, a coupled model (CM-56-56) was generated with a reduced moment capacity of  $0.56 M_p$  for both exterior columns (Fig. 6 (i)).

The first-story drift ratio time history shows a good agreement between the hybrid test and the coupled model (CM-56-56) in the elastic range (time less than 2.5 s) as shown in Fig. 7. However, once the first-story exterior column (column (b) in Fig. 1-right) of the CM-56-56 model yields at its base, larger first-story drift ratios are obtained with respect to the ones from the hybrid simulation (at approximately 2.75 s). The moment at the base-first story drift ratio response demonstrates that the physical specimen has a higher strength (in the order of 70%) as compared to the strength specified for the first-story column nonlinear springs (see Fig. 8). The strain-hardening slope (the slope between yield rotation and rotation at maximum moment) is also steeper than the one assigned to the nonlinear spring of the numerical model. These increases are deemed to be caused by significant spread of inelasticity observed throughout the height of the test specimen (75% of the height), which is not appropriately captured by the concentrated plasticity approach used for the numerical model. This

spread of inelasticity is due to the moment gradient imposed on the column from the combined effect of lateral drift and the applied rotation at its tip.

A reduced moment strength of  $0.56 M_p$  was considered for exterior columns (columns (a) and (b) in Fig. 1-right) due to the presence of axial load based on the approach described in the *Numerical Model* section of this paper. However, the maximum moment obtained experimentally was approximately equal to  $0.95 M_p$  (Fig. 8). Once again, this is primarily due to the fact that  $M_p$  is estimated based on cross-sectional properties without taking into account the spread of inelasticity in the element. The bending moment strength of the first-story exterior column nonlinear springs was updated accordingly in the coupled model and a virtual hybrid simulation was conducted (this model is referred to from here on as the updated coupled model (CM-56-95) (see Fig. 6 (ii)). The first-story drift time history of this simulation is presented in Fig. 7, which shows a good agreement with the performed hybrid test. This result highlights the importance of appropriate modeling of columns in the inelastic range.

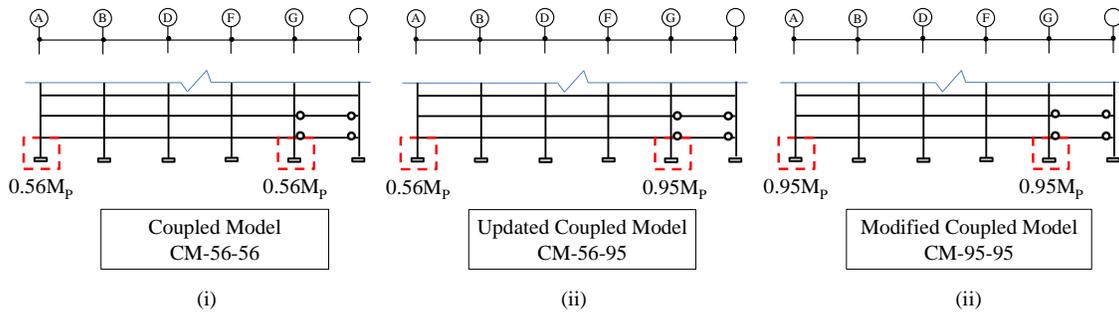


Figure 6. Specification of the coupled models

The base shear hysteresis in Fig. 9 illustrates the contribution of global P-Delta to the response of the structure for both the hybrid simulation and the updated coupled model (CM-56-95). In the inelastic range, as the first-story drift ratio increases, the base shear tends to decrease. An evaluation of the distribution of drift ratios along the height of the structure is shown in Fig. 10. Results from the hybrid test and the updated coupled model (CM-56-95) are depicted. In the elastic range (time equal to 1.5 s) and at a first-story drift ratio close to 0.04 (time equal to 2.65 s), the story drift ratios are consistent. However, at larger levels of inelastic behavior (e.g., at time equal to 4 s) differences in the order of 10% are observed in the lower stories.

The Newmark's method with a fixed number of iterations, which was implemented in the hybrid simulations, does not check for convergence at the end of each step. Therefore, the unbalanced forces had to be monitored separately. The simulation time step was varied during the test as shown in Table 2 to reduce the norm value of the unbalanced force vector during the simulation. The time step sizes had to be reduced as much as possible by taking into consideration values of incremental displacements that were appropriate for the range of the resolution of the actuators.

Table 2. Variation in the time steps for original collapse test

Interval	Ground Motion Duration (s)	Number of Iterations	Integration Time Steps (s)
1	$150 \cdot 0.005 \cdot (1/8)^{0.5}$	4	$0.02 \cdot (1/8)^{0.5}$
2	$600 \cdot 0.005 \cdot (1/8)^{0.5}$	4	$0.01 \cdot (1/8)^{0.5}$
3	$500 \cdot 0.005 \cdot (1/8)^{0.5}$	6	$0.0025 \cdot (1/8)^{0.5}$
4	$3250 \cdot 0.005 \cdot (1/8)^{0.5}$	6	$0.001 \cdot (1/8)^{0.5}$

The primary failure mode for the specimen was lateral torsional buckling. The onset of lateral torsional buckling took place right at the point of maximum bending moment at the base in the hysteretic response of the physical specimen.

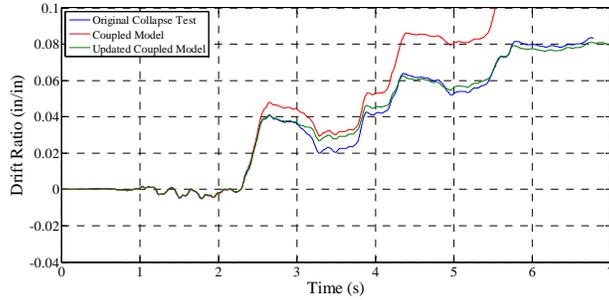


Figure 7. First-story drift ratio time history of original collapse test and coupled models

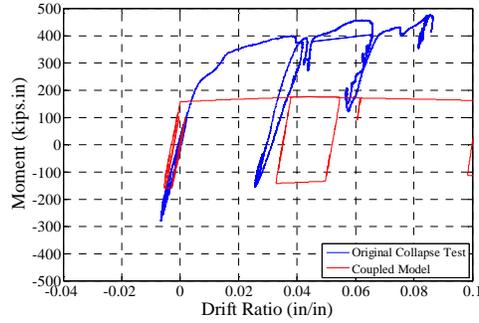


Figure 8. Moment- drift ratio relationship at the base of the column of original collapse test and coupled model

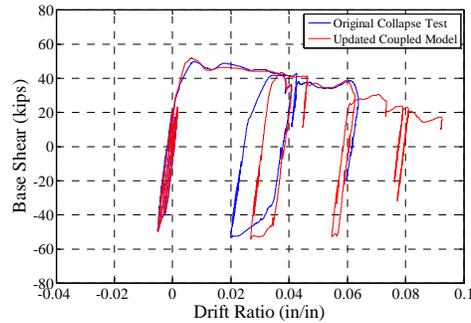


Figure 9. Base shear vs. first-story drift ratio of original collapse test and updated coupled model

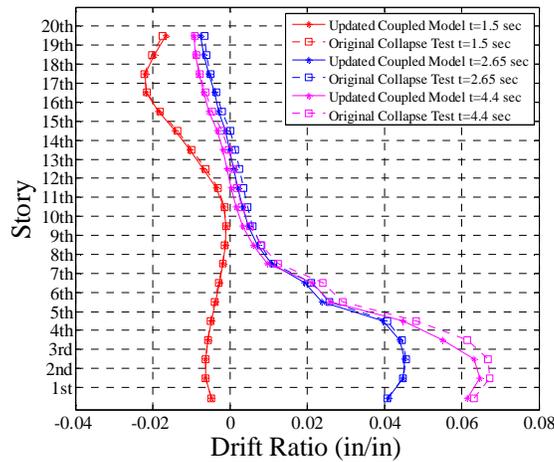


Figure 10. Drift ratio profiles for original collapse test and updated coupled numerical model

## MODIFIED COLLAPSE TEST

In the second hybrid simulation, two modifications were made with respect to the previous test. First, the bending moment capacity of the numerical model of the first-story exterior column (column (a) in Fig. 1- right) was modified from  $0.56 M_p$  to  $0.95 M_p$  based on the results from the previous hybrid simulation. Second, the Duzce record with a larger scale factor (2.7) was utilized (see Fig. 2). A larger scale factor was needed in order to approach the limit state of collapse when the aforementioned increase in bending moment strength was implemented. The time step sizes used in the various portions of this analysis are shown in Table 3. The scaled 2%-damped spectral acceleration at the first mode period of the structure was now equal to  $0.69g$ , which is now greater than the values corresponding to the 2%- damped 2/50 and risk-targeted maximum considered earthquake spectra at the site (both equal to  $0.52g$ ) (see Fig. 2).

In this case, the results from the hybrid simulation were much closer to the predicted numerical results. For this experiment, the reduced moment capacity of  $0.95 M_p$  was considered for both exterior columns (modified coupled model (CM-95-95), Fig. 6 (iii)). For instance, the time history of the first-story drift ratio shows reasonable agreement throughout the history up to collapse (Fig. 11). The moment at the base-drift ratio diagram in Fig. 12 demonstrates that the evaluated bending moment strength of first-story exterior columns assigned to the numerical model was more consistent with the strength exhibited by the physical specimen. However, the strain-hardening slope was still underestimated by the numerical model. This discrepancy is most likely due to the spread of inelasticity through columns height. Furthermore, the column specimen experienced a steeper negative slope after the point of maximum moment (i.e., post-capping slope). This observation is of paramount importance for the calibration of column hysteretic models given that very limited data on the behavior of deep steel columns are available in the literature to evaluate the magnitude of this post-capping slope. This pronounced deterioration in strength and stiffness is attributed to lateral torsional buckling of the column specimen at drift ratio levels greater than 0.04. The authors are currently performing a more detailed evaluation of the reasons behind such an abrupt decrease in strength after the maximum moment is achieved.

Second-order, P-Delta effects are also dominant in this hybrid simulation as shown in Fig. 13. It can also be observed that the overall prediction of base shear responses provided by the modified coupled model (CM-95-95) is close to that of the hybrid experiment. An alternative way of evaluating this P-Delta effect is provided by the story drift ratio profiles shown in Fig. 14. It can be seen that the story drift ratios of the hybrid experiment match the drift ratio profiles from the modified coupled model (CM-95-95) in the elastic and slightly inelastic range (time equal to 2.65 s). However, once the level of inelastic behavior and the effect of P-Delta become more significant (time equal to 3.9 s), P-Delta effects result in an amplification of drift ratio demands in the bottom stories – the drift ratios obtained from the hybrid simulation are 17% larger. This difference is attributed primarily to the relative sudden drop in bending moment capacity of the test specimen after the maximum moment is attained.

Table 3. Variation in the time steps for modified collapse test

Range	Ground Motion Duration (s)	Number of Iterations	Integration Time Steps (s)
1	$600 * 0.005 * (1/8)^{0.5}$	4	$0.02 * (1/8)^{0.5}$
2	$1800 * 0.005 * (1/8)^{0.5}$	6	$0.001 * (1/8)^{0.5}$
3	$3000 * 0.005 * (1/8)^{0.5}$	6	$0.0008 * (1/8)^{0.5}$

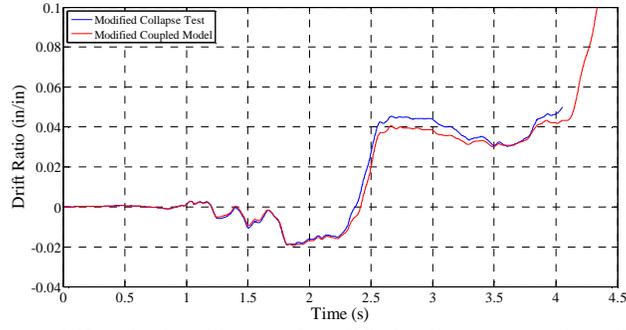


Figure 11. First-story drift ratio time history of modified collapse test and modified coupled model

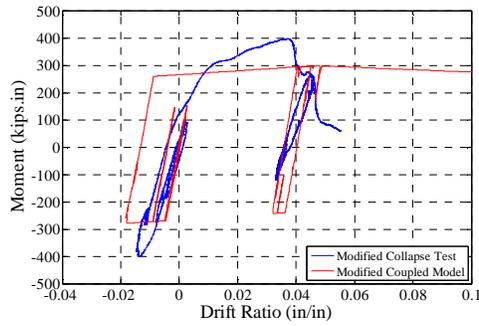


Figure 12. Moment- drift ratio relationship at the base of the column of modified collapse test and modified coupled model

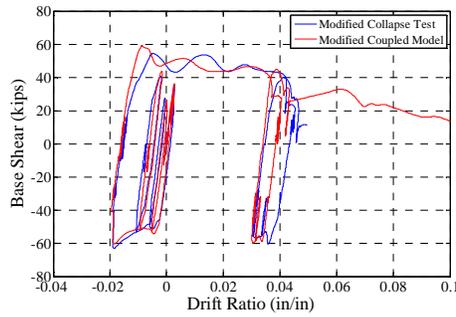


Figure 13. Base shear vs. first-story drift ratio of modified collapse test and the modified coupled model

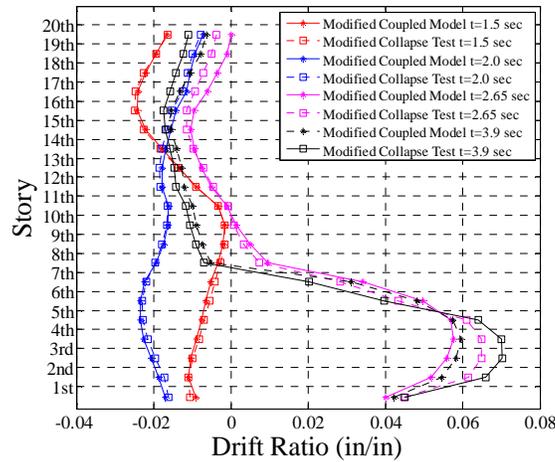


Figure 14. Drift ratio profiles for modified collapse test and the modified coupled model

## CONCLUSIONS

Two different hybrid tests were performed on a 20-story moment resisting frame with a 1:8 scaled model of an exterior deep steel column as the physical element. Lateral displacements, vertical displacements, and rotations at the tip of the column were controlled and the response of the column was evaluated from elastic behavior to the onset of global collapse. This implies that variable shear force, axial load, and bending moment demands were imposed at the tip of the column, which allowed for a more accurate simulation of changes in the moment gradient of an exterior column that is part of moment-resisting frame. These results are deemed to be valuable for an improved calibration of numerical hysteretic models of deep steel columns that are exposed to significant strength and stiffness degradation in the presence of variable axial load demands.

The primary failure mode of the column specimens was lateral torsional buckling. This resulted in severe strength and stiffness degradation after the maximum bending moment was achieved in the modified collapse test. The hybrid simulations demonstrated that spread of inelasticity along 75% of the height of the column provides in this case a maximum bending moment strength 41% larger than the one predicted from *a priori* knowledge based on tests conducted in the past. This discrepancy reinforces the need to further investigate the response of physical deep steel columns to provide data useful for model calibration and improved numerical collapse predictions of structural systems. In addition, these results highlight the importance of a newly developed numerical updating approach for hybrid simulation in which the properties of the numerical model can be updated during the experiment based on the knowledge obtained from the response of the physical specimen (Negrete et al., 2014).

In general, it was shown that the hybrid substructuring technique and displacement-control approach implemented as part of the hybrid architecture of these tests were successful in tracing the behavior of a tall steel structure until the onset of global instability was approached.

It is important to note that these hybrid simulations referred to two-dimensional models; a single column prototype; and a single ground motion scaled to two different intensity levels. More general conclusions of deep steel column behavior relevant for numerical model calibration will necessitate additional experiments with columns of various sizes and scales that are exposed to biaxial bending moment demands, as well as ground motions with various intensities, durations, and frequency contents. The results from these tests should be interpreted within the conditions and assumptions used to conduct them. The quasi-static tests conducted with these columns (Zargar et al., 2014) as well as the hybrid tests presented herein are just initial steps geared toward characterizing the behavior of deep steel column sections more accurately.

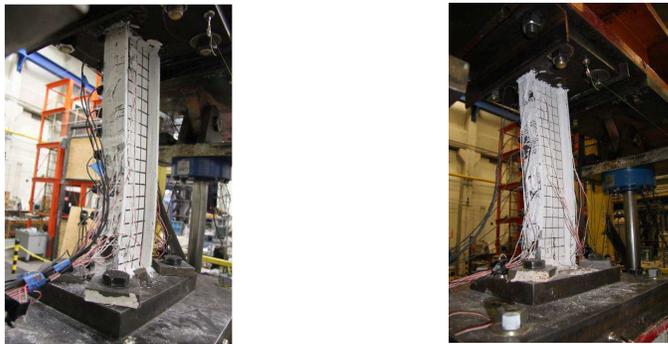


Figure 15. Lateral torsional buckling of the south flange (left-Original Collapse Test) east view (right- Modified Collapse Test).

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