HYBRID SIMULATION OF STRUCTURAL COLLAPSE

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ABSTRACT:

The ability to model structural collapse is essential for improving our understanding of the performance of structures during very rare earthquake events. However, experimental investigations of collapse are complex, expensive and potentially dangerous. This paper demonstrates a novel extension of the hybrid simulation testing method to model structural collapse. The principal advantage of this extension is that, contrary to shaking table testing, no large physical masses and equipment protection systems are needed for the experimental part of a hybrid simulation, making the collapse experiment less expensive and significantly safer.

The Open System for Earthquake Engineering Simulation (OpenSees) structural modeling software and the Open Framework for Experimental Setup and Control (OpenFresco) hybrid simulation software are used to consistently model geometric nonlinearities in the analytical portion of the hybrid model, including the geometric transformation involving the degrees-of-freedom of the experimental subassemblies. A hybrid simulation of the seismic response of a one-story portal frame with two ductile columns is carried out until collapse to demonstrate the newly proposed implementation. The columns, modeled using physical specimens, were not axially loaded: instead, second-order analytical geometric transformations were used to affect the actions of the axial load. A direct comparison of the results of two hybrid simulations, with and without accounting for gravity load effects, shows that the hybrid model with P-Delta effects develops negative post-peak stiffness, incrementally increasing lateral displacements until collapse.

KEYWORDS: Hybrid Simulation, Structural Collapse, P-Delta Effects, OpenFresco

1. INTRODUCTION

Experiments are an essential aspect of understanding the effects of earthquakes on the built environment, and for developing and evaluating design and analysis procedures for use in earthquake engineering. One very important research area in the field of earthquake engineering, is concerned with the ability to model structural collapse in order to improve our understanding of the performance of structures during very rare earthquake events. However, experimental investigations of collapse are in general complex, expensive and potentially dangerous. At present, the following three well-established experimental methods are generally employed:

1) In the quasi-static testing method, actuators impose a predefined history of loads and displacements on the test specimen being investigated. By imposing the same load or displacement history on a series of specimens, the effect of systematic changes in material properties, boundary conditions, loading rates, and other factors can be readily identified. While such tests are relatively easy and economical to perform, the demands imposed on the test specimens are not directly related to the observed damage. This raises questions about whether the specimens were under- or over-tested, which in turn makes it extremely difficult to draw accurate conclusions about the behavior of a structure near or at collapse.

2) Shaking table tests are a second form of tests, and they are able to produce conditions that closely resemble those that would exist during a particular earthquake. These tests provide important data on the dynamic response to specific ground motions considering the inertial and energy dissipation characteristics of the structure tested and the consequences on response of geometric nonlinearities and localized yielding and damage. However, for such tests, a complete structural system is required, and the specimen needs to be constructed including complex active or passive gravity load setups. Furthermore, preventive measures need to be taken to protect expensive test equipment from specimen impact during collapse. This makes investigations of collapse on shaking tables very complex, expensive and potentially dangerous. In addition, due to simulator platform and/or control system limitations reduced scale or highly simplified specimens are commonly necessitated, which call into question the realism of many shaking table tests.

3) The third method is hybrid simulation, where the simulation is performed based on a step-by-step numerical solution of the governing equations of motion for a model that is formulated considering both numerical and physical portions of a structure. As implemented for a typical quasi-static, displacement-based hybrid simulation, the mass and viscous damping characteristics of the specimen are numerically modeled, and the incremental displacement response of the structure to a specified ground motion is computed at each step based on the current state of the physically and numerically modeled subassemblies of the structure. This makes it possible to investigate geometric nonlinearities, three-dimensional effects, multiple support excitation and soil-structure interactions by incorporating them into the analytical portion of the hybrid model, while the physically represented portions of the overall hybrid model can be tested in one or more laboratories using computer-controlled actuators. Hence, the hybrid simulation method gives the researcher the ability to test large-scale specimens. In addition, only the critical, collapse-sensitive, elements of the structure need to be physically tested, allowing for a substantial increase in the number of different collapse tests afforded by the same budget. Furthermore, since dynamic aspects of the simulation are handled numerically, such tests can be conducted quasi-statically using standard computer-controlled actuators. As such, hybrid simulation may be viewed as an advanced form of actuator-based testing, where the loading history is determined during the course of an experiment for a given system subjected to a specific ground motion.

To demonstrate how geometric nonlinearities can be accounted for in the numerical part of the model, a hybrid simulation, wherein a portal frame is tested by consistently accounting for second-order effects due to gravity loads, is carried out using the Open System for Earthquake Engineering Simulation (OpenSees) (McKenna, 1997; Fenves et al., 2007) and the Open Framework for Experimental Setup and Control (OpenFresco) (Takahashi and Fenves, 2006; Schellenberg et al., 2007) software packages. After a short introduction to nonlinear geometric effects, the theory and implementation details of such second-order effects, necessary to carry out the structural collapse simulations, are explained next. Finally, the portal frame example, which was utilized to validate the new approach, is presented.

2. SECOND-ORDER EFFECTS

Because the second-order effects are caused by gravity loads, it is important to understand how such loads affect the response of a structure. In general, gravity loads influence the response of a structure on three levels that can be categorized as shown in Table 2.1. To model these nonlinear geometric effects in a finite element software (such as OpenSees), the element equilibrium equations need to be satisfied in the deformed configuration. Furthermore, because of large displacements, the compatibility relations between element deformations and global element end-displacements become nonlinear. For frame elements it is then possible to separate these coordinate transformations from the actual implementation of the element itself, as long as the element force-deformation relations are defined in a basic coordinate system without rigid body modes. Hence, different geometric theories can be implemented for the same element without modifying the element code. In OpenSees the *CrdTransf* software class provides this exact functionality by transforming the response quantities of any frame element from the global degrees-of-freedom to the simply-supported basic element degrees-of-freedom and back. The *CrdTransf* class is an abstract base class that has concrete subclasses for linear, p-delta and corotational geometric transformations. This abstraction and encapsulation of the frame element coordinate transformations facilitates the switching among different linear and non-linear coordinate transformations in the OpenSees software framework without affecting the implementations of the frame elements themselves.

Table 2.1: Effects of gravity loads on structural response.

Level	Effect
Section	Gravity loads cause axial forces that interact with bending moments and shears at the cross section, commonly known as P-M and P-V interaction.
Element	Gravity loads affect the stability of structural elements and critical regions, causing softening and local and/or lateral-torsional buckling of elements, commonly known as the small P-Delta (P-δ) effects.
Structure	Gravity loads influence the stability of a structure, causing additional overturning moments due to large displacements. These effects are commonly known as the large P-Delta (P-Δ) effects.

Because the gravity loads on the simple one-story, one-bay portal frame shown in Figure 4 are to be entirely treated in the analytical portion of the hybrid model, with no axial loads imposed on the experimental parts of the structure (column cross-sections and elements), only the large P-Delta effects are accounted for in the demonstration and validation example in section 4.

3. THEORY AND IMPLEMENTATION

The 2D experimental beam-column element formulation in OpenFresco (which is used for the portal frame example) is based on the three collocated degrees-of-freedom of the cantilever basic system (see Figure 1c). The necessary transformations of displacements, velocities, accelerations and forces are implemented in the concrete EEBeamColumn2d class.

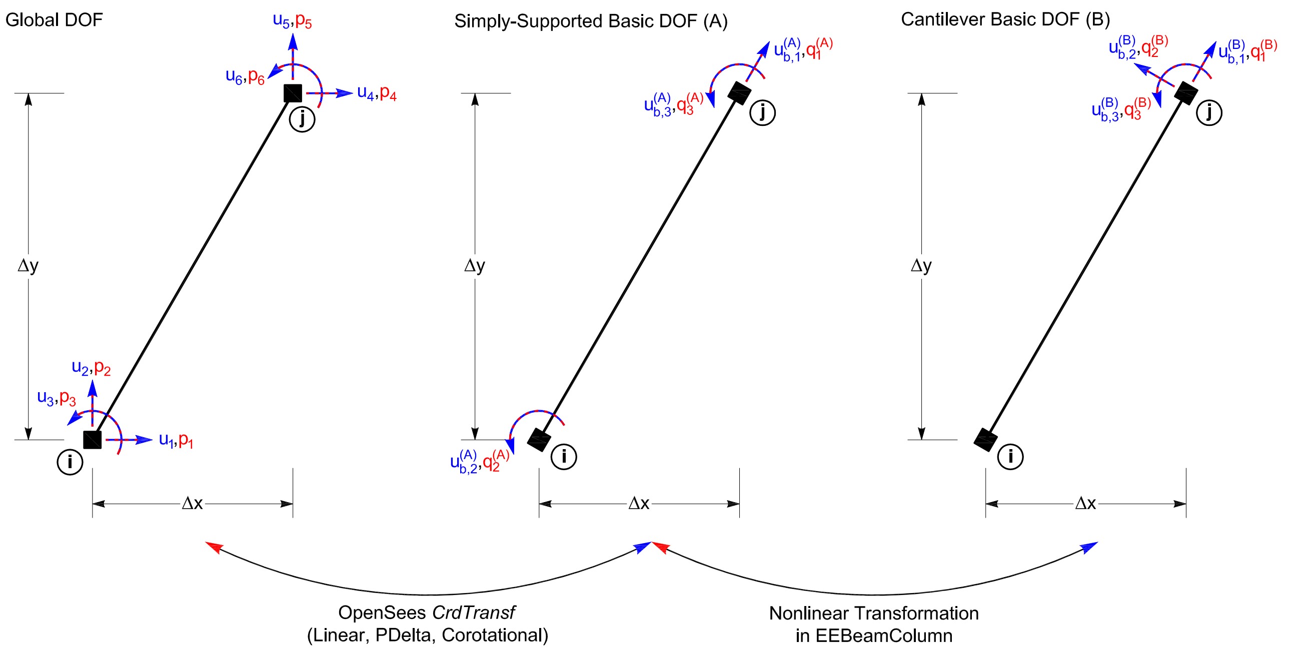

Figure 1: Experimental beam-column element (EEBeamColumn2d).

Since this experimental element is a frame member similar to an analytical beam-column element, the previously described OpenSees *CrdTransf* class is employed to transform the response quantities from the global degrees-of-freedom to the simply-supported basic element degrees-of-freedom and back. However, because the simply-supported basic system is not well suited for experimental testing (due to the two rotational degrees-of-freedom), the response quantities are further transformed from the simply-supported to the cantilever basic system and back. These additional nonlinear transformations are directly implemented in the EEBeamColumn2d class and described in detail next.

Of the three coordinate transformations (linear, p-delta and corotational) that are available in OpenSees, only the large displacement, moderate deformations corotational transformation (de Souza, 2000; Filippou and Fenves, 2004) is explained here. In a first step (which is not shown) the six end displacements of the beam-column element are transformed from the global to the local coordinate system by applying a rotational matrix. Afterwards, the element trial displacements u_l in the local coordinate system are transformed to the element deformations u_b in the basic coordinate system using the following equation.

$$\begin{aligned} u_{b,1}^{(A)} &= L_n - L \\ u_{b,2}^{(A)} &= u_{l,3} - \alpha \\ u_{b,3}^{(A)} &= u_{l,6} - \alpha \end{aligned} \qquad with \qquad \begin{aligned} L_n &= \sqrt{\left(L + u_{l,4} - u_{l,1}\right)^2 + \left(u_{l,5} - u_{l,2}\right)^2} \\ \alpha &= \arctan\left(\frac{u_{l,5} - u_{l,2}}{L + u_{l,4} - u_{l,1}}\right) \end{aligned} \tag{3.1}$$

where L_n is the length of the element chord in its deformed configuration and α is the rotation of the chord from the undeformed to the deformed element configuration.

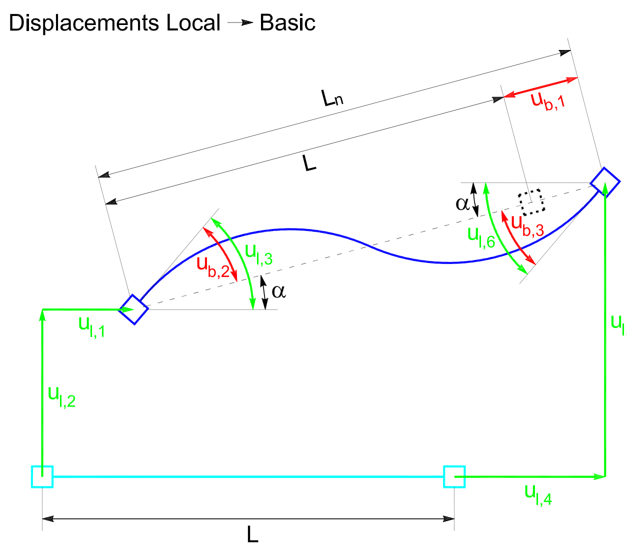

Figure 2: Corotational transformation from local to basic system A.

The transformations of the resisting forces and the element stiffness matrix from the basic system A to the local coordinate system take the following form.

$$\begin{aligned} \mathbf{p}_l &= \mathbf{a}^T \mathbf{q}^{(A)} \\ \mathbf{k}_l &= \frac{\partial \mathbf{p}_l}{\partial \mathbf{u}_l} = \frac{\partial}{\partial \mathbf{u}_l}\left(\mathbf{a}^T \mathbf{q}^{(A)}\right) = \mathbf{a}^T \mathbf{k}_b^{(A)} \mathbf{a} + \frac{\partial \mathbf{a}^T}{\partial \mathbf{u}_l} \mathbf{q}^{(A)} \end{aligned} \tag{3.2}$$

where the compatibility matrix, $\mathbf{a}$, and the elastic element stiffness matrix, $\mathbf{k}_b^{(A)}$, in the basic coordinate system A are given in Equation (3.3) below. $\mathbf{u}_l$ and $\mathbf{p}_l$ are the element displacements and forces in the local coordinate system, $\mathbf{u}_b^{(A)}$ and $\mathbf{q}^{(A)}$ are the element deformations and forces in the basic coordinate system A, α is the chord rotation and L_n is the length of the element chord in its deformed configuration. Furthermore, the first part of the local stiffness matrix in (3.2b) represents the material stiffness and the second part represents the geometric stiffness.

$$\mathbf{a}=\begin{bmatrix} -\cos(\alpha) & -\sin(\alpha) & 0 & \cos(\alpha) & \sin(\alpha) & 0 \\ -\frac{\sin(\alpha)}{L_n} & \frac{\cos(\alpha)}{L_n} & 1 & \frac{\sin(\alpha)}{L_n} & -\frac{\cos(\alpha)}{L_n} & 0 \\ -\frac{\sin(\alpha)}{L_n} & \frac{\cos(\alpha)}{L_n} & 0 & \frac{\sin(\alpha)}{L_n} & -\frac{\cos(\alpha)}{L_n} & 1 \end{bmatrix},\ \mathbf{k}_{\mathbf{b}}^{(A)}=\begin{bmatrix} \frac{EA}{L} & 0 & 0 \\ 0 & \frac{4EI}{L} & \frac{2EI}{L} \\ 0 & \frac{2EI}{L} & \frac{4EI}{L} \end{bmatrix} \tag{3.3}$$

The nonlinear transformations of the trial displacements from basic system A (simply-supported) to basic system B (cantilever) are shown in Figure 3 and given by the following expression.

$$\begin{aligned} u_{b,1}^{(B)} &= L_n \cos\left(u_{b,2}^{(A)}\right) - L \\ u_{b,2}^{(B)} &= -L_n \sin\left(u_{b,2}^{(A)}\right) \qquad \textit{with} \quad L_n = \left(L + u_{b,1}^{(A)}\right) \\ u_{b,3}^{(B)} &= -u_{b,2}^{(A)} + u_{b,3}^{(A)} \end{aligned} \tag{3.4}$$

where L_n is the length of the element in its deformed configuration as before. The nonlinear transformations for the trial velocities and trial accelerations can be determined by taking derivatives of Equations (3.4) with respect to time. The expressions can easily be evaluated with any computer algebra system (CAS) such as Mathematica or Maple, but get quite involved and are therefore not shown here.

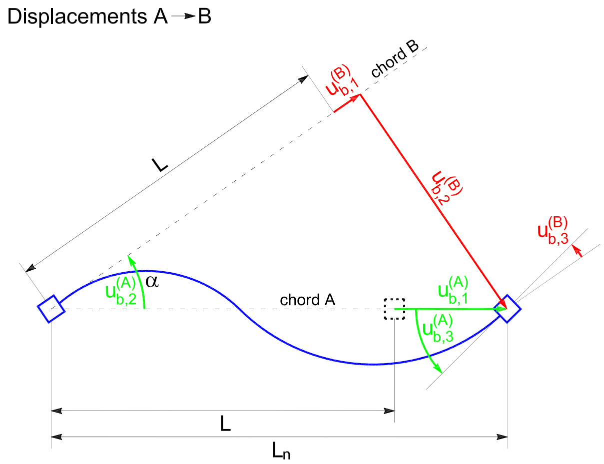

Figure 3: Nonlinear transformation from basic system A to B.

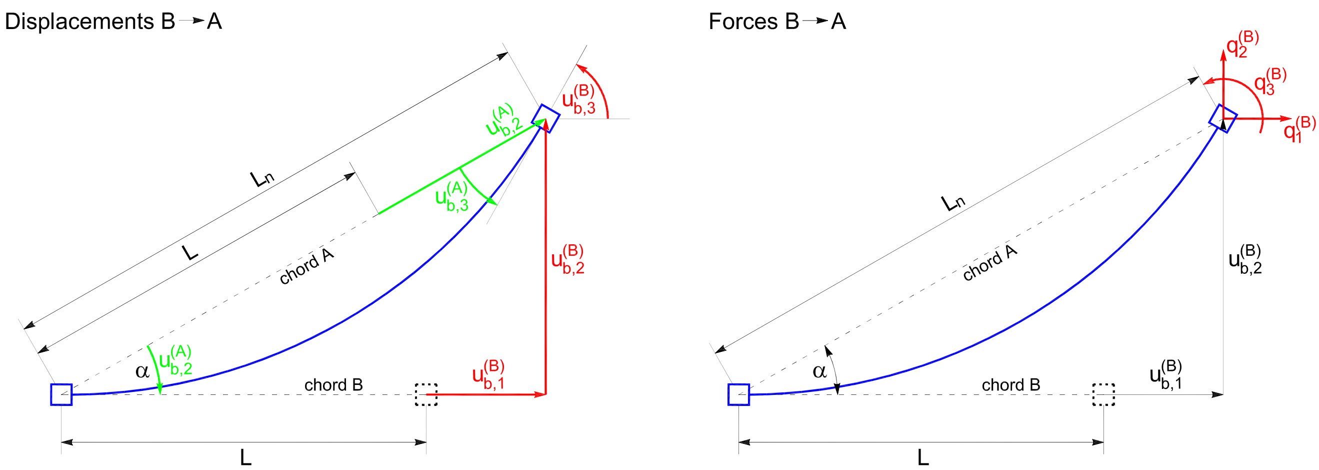

Figure 4: Nonlinear transformations from basic system B to A.

Finally, the measured forces are transformed from basic system B (cantilever) to basic system A (simply-supported) as shown in Figure 4 and given by the following expression.

$$
\begin{aligned}
q_1^{(A)} &= \cos(\alpha)\, q_1^{(B)} + \sin(\alpha)\, q_2^{(B)} \\
q_2^{(A)} &= u_{b,2}^{(B)}\, q_1^{(B)} - \left(L + u_{b,1}^{(B)}\right) q_2^{(B)} - q_3^{(B)} \qquad with \quad \alpha = \arctan\left(\frac{u_{b,2}^{(B)}}{L + u_{b,1}^{(B)}}\right) \\
q_3^{(A)} &= q_3^{(B)}
\end{aligned}
\tag{3.5}
$$

where α is the chord rotation and is given in terms of the trial displacements instead of the measured displacements to reduce the effect of experimental errors. This concludes the derivation of the 2D experimental beam-column element and provides the basis for the hybrid simulation of structural collapse in the next section.

4. PORTAL FRAME EXAMPLE

This section demonstrates how the novel experimental beam-column elements can be used to perform a hybrid simulation until collapse and validates the accuracy of such approach. In the illustrative example presented, the OpenSees software framework was employed as the computational driver, solving the equations of motion, and the OpenFresco middleware was utilized to represent the experimental elements and connect OpenSees to the laboratory. As shown in Figure 5a, the hybrid model of the steel single-story, single-bay portal frame consisted of a numerically modeled elastic beam (W6x12) and two physically tested S4x7.7 beam-columns. The connecting beam, the viscous energy dissipation and mass properties as well as the gravity and earthquake loads were modeled analytically in OpenSees. The EEBeamColumn2d experimental element (see Figure 1) was used to represent the two physical columns. For convenience in the laboratory, the two beam-column specimens were inverted with the pin end at the top and the fixed end at the bottom. Special clevises incorporating replaceable steel coupons were used to produce stable, repeatable hysteresis-loops in the plastic hinges at the bottom ends of the columns. It is important to emphasize that because the basic coordinate system moved as the elements deformed, the two experimental one-actuator setups (Figure 5b) utilized in these tests also behaved as if they had been attached to and moved with the elements.

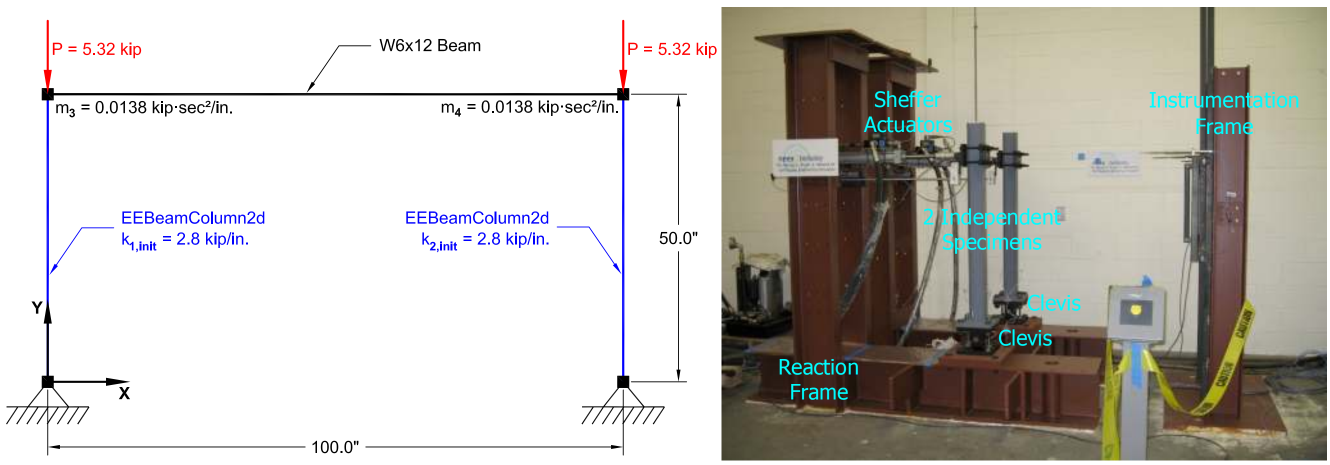

Figure 5: Hybrid model and experimental setup in *nees@berkeley* laboratory.

Considering the axial and flexural deformations in the elements, the portal frame model had a total of 8 degrees-of-freedom (4 translational and 4 rotational). However, it was assumed that only the two horizontal degrees-of-freedom had mass (m = 0.0138 kip·sec^2/in.), which made the 8x8 mass matrix singular with rank equal to two. To achieve an unconditionally stable direct integration method for the hybrid simulation, the Newmark method with 10 sub-steps and the Newton-Raphson algorithm were employed. The fundamental period of the structure was 0.49 seconds and the viscous energy dissipation was modeled with 5% mass proportional damping. The magnitude of the gravity loads (P = 5.32 kip) was set at 50% of the critical side-sway buckling load of the physical columns (including the clevises) assuming that the beam was flexurally rigid and the base supports for the columns were pinned. The hybrid portal frame model was subjected to the SACNF01 near-fault ground motion of the SAC database, scaled to a peak ground acceleration of 0.906g. The ground motion was originally recorded during the 1978 Tabas earthquake. The integration time step was chosen to be $\Delta t_{int} = 0.01$ seconds. On the other hand, the simulation time step was chosen to be $\Delta t_{sim} = 2^{-5} = 0.03125$ seconds and in combination with the 10 sub-steps per time step, the hybrid simulation was performed at 31.25 times slower than real-time and each test lasted 13 minutes.

To investigate the effect of gravity loads on the response of the portal frame, two pairs of simulations were performed. One simulation was performed without any gravity loads applied, and then gravity loads were added to the analytical portion of the hybrid model for the second test. Both tests use the corotational geometric coordinate transformation. The story-drift time histories as well as the story-shear vs. story-drift hysteresis-loops are compared in Figure 6. Even though the frame drifts and the residual displacements are substantial for the first test without gravity loads, the hybrid model does not collapse. In the second hybrid test, the presence of the gravity loads is clearly evident from the negative post-yield tangent stiffness exhibited by the combined hybrid model.

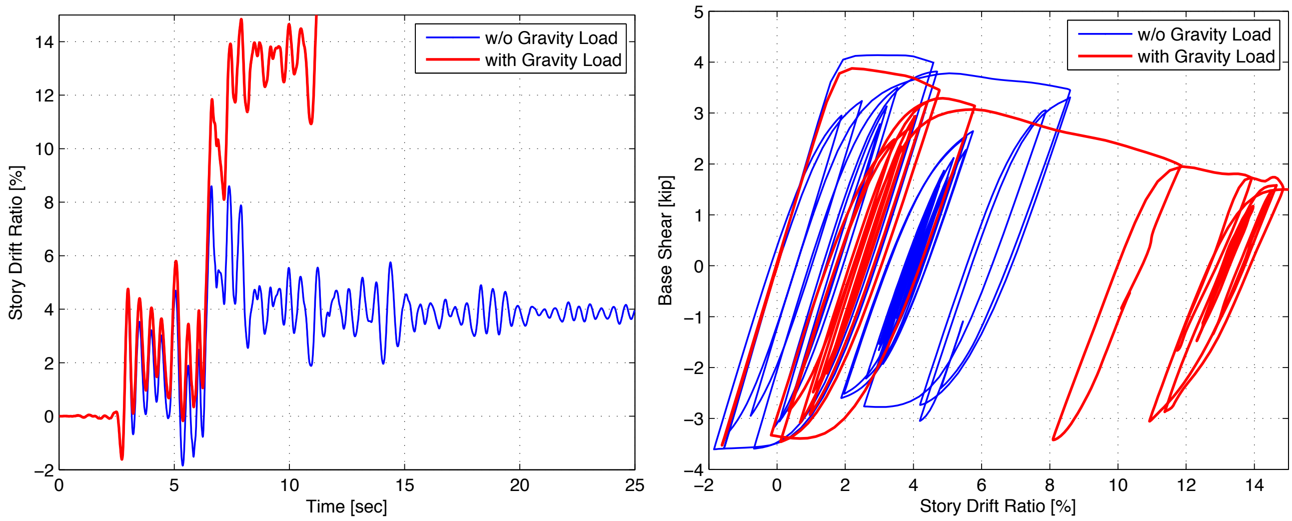

Figure 6: Comparison of story-drift time-histories and total story hysteresis-loops.

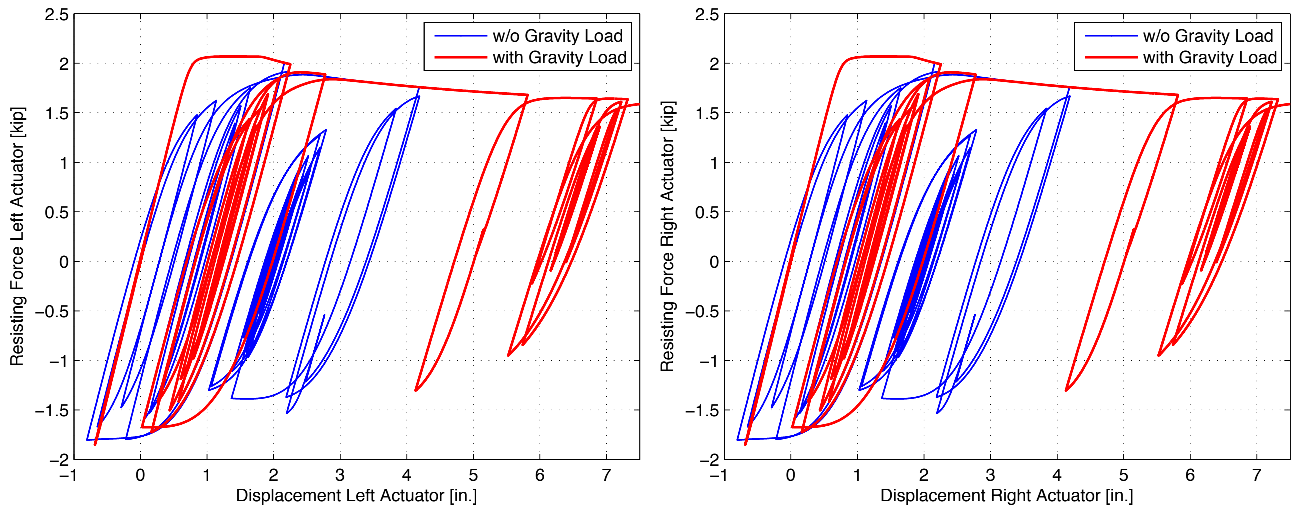

Figure 7: Comparison of actuator force-displacement hysteresis-loops.

As can be seen from Figure 6a, story drifts increase rapidly at 6.5 seconds into the test which can be considered as the initiation of frame collapse. With each additional cycle, the portal frame shifts further over until the hybrid simulation is finally terminated when the actuators reach their stroke limit of 7.5 inches, which corresponds to an inter-story drift ratio of 15%. The second simulation demonstrates that the hybrid model can be tested all the way to collapse of the portal frame, which is difficult or dangerous to accomplish in shaking table tests.

Figure 7 compares hysteresis loops for the two experimental beam-column elements. The actuator-force vs. actuator-displacement relationships are plotted in the basic cantilever coordinate system B as they are acquired from the laboratory, before the gravity load effects are applied by the nonlinear geometric transformations. Thus, no negative post-peak stiffnesses are observed in the actual test. The differences between the hysteresis-loops are entirely due to the consistently applied displacements. Furthermore, it can be seen that for large displacements of the cantilever column tips (respectively large rotations of the clevises) the hysteresis-loops exhibit some degradation. This degrading behavior is due to the buckling of the replaceable steel coupons as the clevis goes through large rotations.

5. CONCLUSIONS

The OpenFresco middleware, with the novel experimental beam-column element, provides a useful and effective set of modules for performing hybrid simulations of structural collapse. The ability to correctly account for the second-order effects in hybrid models is crucial for simulating collapse of structures under gravity loads. Hybrid simulation of collapse behavior offers three significant advantages over conventional testing methods: 1) the gravity loads and the resulting geometric nonlinearities are represented in the analytical portion of the hybrid model in the computer, eliminating the need for complex active or passive gravity load setups; 2) there is no need to protect expensive test equipment from specimen impact during collapse because the actuator control system will limit the movements of the test specimens; and 3) only the critical, collapse-sensitive, elements of the structure need to be physically tested allowing for a substantial increase in the number of different collapse tests afforded by the same budget. Finally, a hybrid simulation of the seismic response of a one-story portal frame with two ductile columns was carried out until collapse to demonstrate and validate the newly proposed implementation.

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