# REAL-TIME HYBRID SIMULATION FOR INFORMING SEISMIC RETROFITTING OF OLDER STRUCTURES

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## Abstract

Hybrid Simulation (HS) is one of the widely-used experimental techniques to evaluate seismic performance of structures, where components with well-understood behavior are simulated analytically, and hard-to-model components are simulated physically tested in the laboratory. The advantage of using selected elements of a structure as a physical model in HS testing could be a feasible and cost-effective for informing and selecting the best seismic retrofit strategies. This study focuses on the use of real-time HS (RTHS) for retrofitting decisions on using diagonal braces for existing older steel moment-resisting frames (MRFs) with limited ductility. Diagonal braces, e.g. conventional HSS steel braces, could be used as retrofit solutions for existing framed structures since braced frames exhibit higher lateral stiffness and lower inter-story drifts under seismic events. The objective of this study is to demonstrate the feasibility of using RTHS for optimizing retrofit decisions of older steel buildings, i.e. size and spatial location of braces. To be specific, existing MRFs buildings with expected strength degradation are selected to be the computational model and small-scale braces are tested physically using a compact RTHS setup at the University of Nevada, Reno (UNR). Due to the lack of strength and limited ductility in the older buildings, detailed computational models should be provided for analytical substructures to properly capture any nonlinear or inelastic behavior that could results from the additional demands due to retrofit. In this study, fiber-section models with highly nonlinear material models are used to properly model the behavior of MRFs. Given that RTHS is targeted to establish the methodology for future wider spectrum of applications, e.g. use of rate-dependent seismic protective devices, the performance of the nonlinear models and analysis convergence issues are assessed. Several retrofitting configurations are tested under different earthquake excitations by placing the braces into different stories, and the building responses are examined to find the best possible retrofitting solution.

*Keywords: real-time hybrid simulation, nonlinear models, retrofitting, steel frames*

## 1. Introduction

Hybrid Simulation (HS) is an advanced testing technique in structural engineering and widely-used to evaluate the performance of structures under dynamic loading such as earthquake excitation. In an HS test setup, analytical and experimental substructures are interacting with each other online in a controlled closed loop. The advantage of this interaction is to combine the analytical substructures which could be properly modelled with accurate predictable structural behavior with experimental substructures that are hard to model analytically with complete accuracy. This introduces cost-effective simulations with more accurate structural performance results. Several studies have been conducted over four decades to improve the capabilities and applicability of HS, including numerical integration methods [e.g. 1–5], substructuring techniques [e.g. 6–8], and experimental error mitigation and delay compensation [e.g. 9–11]. Moreover, a large volume of such studies focused on advancing real-time HS (RTHS), where the real time nature makes the methodology applicable for testing rate-dependent components (e.g. dampers) or dynamic substructuring. Due to these advancements, this experimental technique has been used as a reliable, accurate, efficient, and cost-effective real-time dynamic testing method for years [12].

While the studies on the development of RTHS still continue for large and complex structures, numerous applications could take advantage of the method for structural engineering. One possible application could be using RTHS to select the best retrofitting decision for existing structures. In an HS setting, existing structure is modeled as the analytical substructure while the retrofitting element could be considered as the experimental substructure. This study presents a series of RTHS tests to demonstrate the feasibility of using RTHS for selecting and informing optimal seismic retrofit strategies for existing older steel moment resisting frames (MRFs). Therefore, steel MRFs are retrofitted with conventional braces that form the experimental substructure of the HS system. The RTHS tests are conducted using a compact HS setup that is recently developed at the University of Nevada, Reno (UNR), and the system components are briefly explained in the paper. It is noted that since the experimental setup is small-scale, the objective of the study is to demonstrate and to provide a methodology using RTHS for retrofitting design process rather than determining the ultimate retrofit solutions for the given MRFs.

It was witnessed after the 1994 Northridge earthquake that MRFs are vulnerable to seismic excitations. Since then, investigating the performance of the MRFs, their connections, and seismic retrofitting have been an attractive topic for researchers [e.g., 13]. Either conventional steel braces, i.e., hollow structural sections (HSS) braces, or BRBs are commonly used as retrofit components for existing framed structures since braced frames exhibit higher lateral stiffness under seismic events, and the interstory drifts are considerably limited [14]. In this study, two different show cases with RTHS are presented for steel MRF retrofitting, where conventional braces are used as an experimental substructure.

Older buildings have limited strength and ductility, and retrofitting could produce additional seismic demands on other building components in an event of an earthquake. Thus, detailed computational models are necessary to capture the nonlinear behavior of the structure accurately. For this study, the analytical models are prepared in OpenSees [15] due to the wide spectrum of modeling and analysis options. To model MRFs analytically, two types of modeling assumptions can be followed, namely concentrated plasticity approach and distributed plasticity approach. In the first case study, the distributed plasticity approach is adapted where the plasticity is distributed through the member in specific sections. Since the distributed plasticity with fiber sections is already a complicated modeling approach, bilinear material nonlinearity is considered for steel material. A brief computational study is first presented here to study the RTHS applicability of various integration methods. Next, a nine-story MRF building is selected, and three retrofitting options are tested where the location of the brace changed. For the second case study, a two-story MRF is tested by placing two different braces into two different stories. For this study, the MRFs are modeled with distributed plasticity models. However, the nonlinearity is only distributed over the finite length hinge zone with strength degradation and post-peak negative stiffness. Overall, the findings on these two case studies are presented to show the applicability of the use of HS/RTHS for seismic retrofitting decisions.

## 2. System Components and Capabilities

The compact HS setup at UNR is designed and assembled to tackle new computational challenges for HS and RTHS, braced-frames demonstrations, and educational purposes [16]. The HS setup is capable of running both real-time and pseudo-dynamic (slow) experiments. Moreover, both Simulink or OpenSees/OpenFresco platforms could be used for modeling and execution of HS.

The system components of the HS setup are shown in Fig. 1 (a) which include a load frame with a dynamic actuator run by an isolated hydraulic pump. The load frame constitutes the experimental setup for the HS system. The maximum load capacity of the dynamic actuator of the system is 7 kips (31.14 kN), where the stroke is ±25.4 mm. Moreover, the peak velocity of the actuator at no load is 338.84 mm/sec, where the isolated hydraulic power supply system has 8.71 lt/min (2.3 gpm) pumps, and the reservoir capacity of oil volume is 56.78 lt (15 gallons). The second component is MTS STS controller (STS 493 Hardware Controller) with 4-channels with 2048 Hz clock speed which dictates the rate at which actuator is controlled. Currently, only one of the channels is connected to the actuator but there is the capability of controlling 4 actuators. Also, the system has an STS Host PC to identify and control the basic controller properties with a graphical user interface for the MTS 493 controller [17]. The third component is real-time high performance Simulink machine (Speedgoat xPC Target), which provides a high-performance host-target prototyping environment that enables the researcher to connect the Simulink and Stateflow models to physical systems. This environment is where the data is sent to and received from the controller. Other components include a windows machine (HostPC) where MATLAB, OpenSees, and the HS middleware OpenFresco [18] are located, and a SCRAMNetGT ring connected to all components to provide shared memory locations for the real-time communication of the system.

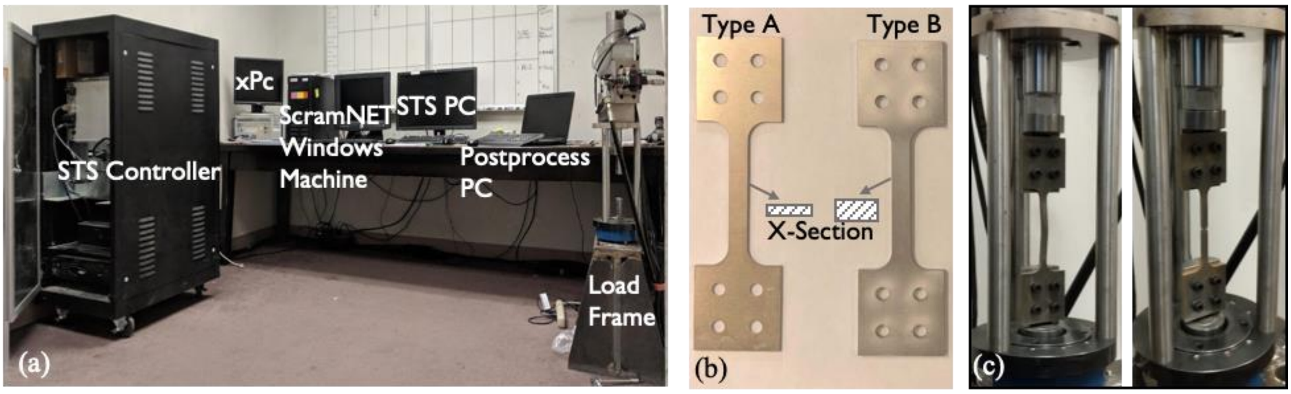


Fig. 1 – (a) Different components of the compact HS test setup at UNR, (b) type A and type B specimens used in this study, (c) typical buckling and fracture for one of the tested braces from the HS experiment

For this study, analytical substructures are modeled in OpenSees to take advantage of the various comprehensive modeling and analysis capabilities and options as opposed to developing the codes in Simulink. The specimens considered for this study, i.e. typical small-brace brace, and an illustration of buckling and fracture of the HS experimental substructure is shown in Fig. 1 (b) and (c), respectively. OpenFresco middleware coordinates the data transfer between the analytical and the experimental substructure. Moreover, the rate of the test (slow or real-time) is also controlled by the predictor-corrector algorithm that is defined within OpenFresco [11]. By defining the same integration time step with the simulation time step, the test runs in real time. To conduct a slow HS, the simulation time step should be defined larger than the integration time step. On the other hand, the system is capable of doing the slowing



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down of the simulation with controlling the actuator movement in constant velocity. Furthermore, the actuator delay is compensated with the Adaptive Time Series (ATS) compensator introduced by Chae [19].

## 3. Case Study 1: Retrofit for 9 Story Steel MRF

For the first case study, analytical models are developed and a series of nonlinear analyses are conducted to assess computational time for different buildings. Then one of these buildings is selected, and three RTHS tests are conducted where the braces are located at different stories of the structure.

### 3.1 Analytical Models and Computational Study

One-bay steel MRFs starting from 3 to 21-stories are modeled in OpenSees for the analytical study. Lateral resistance of the MRFs is satisfied through the bending of the columns, beams, and moment resisting beam-column connections. In order to model MRF members for this case study, distributed plasticity with fiber section elements are used (Fig. 2a). The nonlinearity is distributed through the specific sections of the element with fiber cross-sections (Fig. 2b). The uniaxial material is selected to be "Hardening material" that is defined in OpenSees (Fig. 2c), which has linear kinematic and isotropic hardening. For this study, only kinematic hardening is considered by defining kinematic hardening modulus ($H_{kin}$), where isotropic hardening ($H_{iso}$) assumed to be zero (Fig. 2c). The expected yield stress for the steel material is defined to be 36 ksi. The uniaxial material is assigned to the fibers of the cross-section. The strains across the cross-section are integrated to calculate the flexural and axial stiffness and the stress.

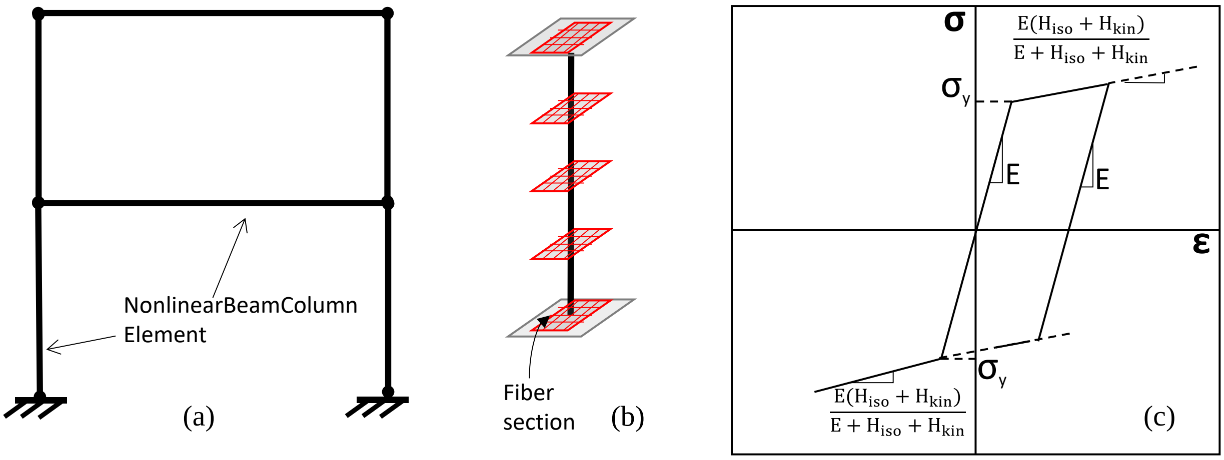


Fig. 2 – (a) Schematic of OpenSees model with distributed plasticity, (b) idealized element model for distributed plasticity with fiber sections, and (c) hardening material model

The bay width and heights are 12 ft for all buildings with W-sections used for columns and beams and changed in every three stories for taller buildings (see Fig. 3a). Columns are fixed at the base, and beams have fixed connections to columns. Lumped masses are assigned to the nodes of each floor in the lateral direction, which is 1 kip-sec$^2$/in. Negligible vertical and rotational masses are assigned in order to eliminate the errors and the stability problems that could be related to the explicit integration algorithms to solve the equation of motion. The structures are assumed to have 2% mass and stiffness proportional damping ratio, which is assigned to the first two modes of the structures.

The first three analytical model representations and the steel sections of the columns and beams that are used in the model are shown in Fig. 3(a). As known for RTHS, all of the calculations and data transfer between the analytical substructure and experimental substructure should be performed within a time step, which is typically less than 10 msec. First, an analytical study is conducted to define the computational time for each analysis time step and see the applicability of the RTHS with large analytical models with extensive nonlinear behaviors. Explicit KR-α method is used due to having unconditional stability and controllable



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numerical damping for linear elastic and stiffness softening type nonlinear systems. The integration time step is 0.01 sec. The computational time for each analysis time step for each building is obtained. It can be seen from the results presented in Fig. 3(b), with these modeling assumptions, it is possible to conduct RTHS for a structure with more than 120 degrees-of-freedom.

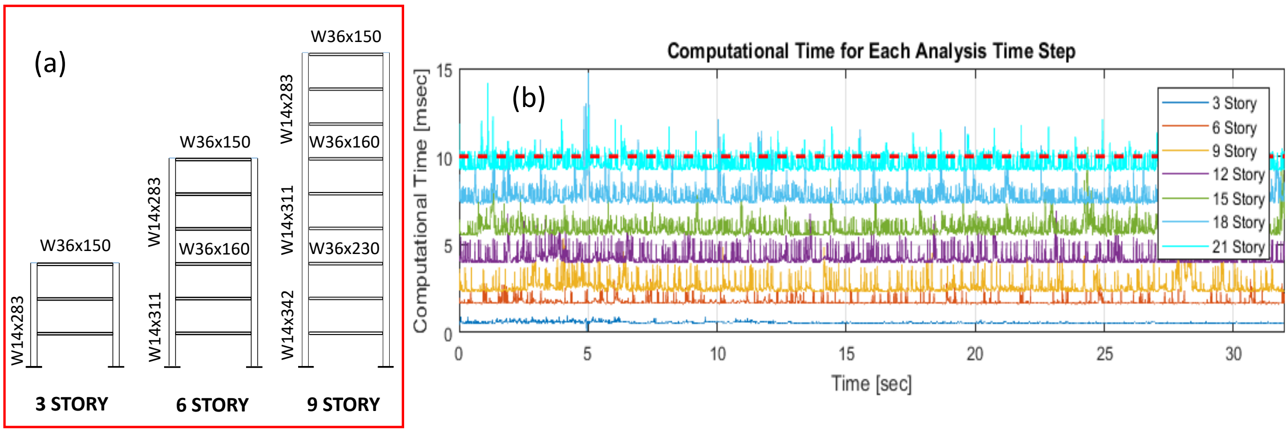


Fig. 3 – (a) Sample of analytical models, and (b) computational time for each analysis time step

### 3.2 RTHS Application and Results

Among the analytical models that are considered for the computational time study, nine-story MRF is selected to do retrofitting. As a retrofitting element, a conventional small-scale brace is considered as the experimental substructure of the HS system. The material of the small-scale dog-bone shaped specimen is aluminum ($F_y$=35ksi), which has a cross-section of 1/8”x1/2” and can be seen in Fig. 1(b) above as Type A. Similitude laws are considered where the scale factor is 1/25. The brace has pinned connections where it works as an axially loaded element. In this case study, the brace is placed in a different story, 1st, 4th, and 7th stories, one at a time in separate tests to show how the location of the retrofitting element would affect both global and local member behaviors. The brace locations are schematically presented in Fig. 4.

RTHS tests are conducted under the El-Centro ground motion with increasing scale until the brace ruptures. All tests start with 100% original scale ground motion and increased with 10% increments. However, a sample of the results of the RTHS tests are presented here for only original scale ground motion. Inter-story drifts, story displacements, selected member responses from the analytical substructure, i.e. fourth story column and first story beam, and experimental substructure force-displacement hysteresis are shown in Fig. 4 for the three different retrofit cases (each set of results boxed together using a different color for convenience).

One way of finalizing a retrofitting decision is to check the inter-story drift ratios for the retrofitted buildings. However, it is also essential to check how retrofitting affects the other structural members since retrofitting can result in higher demands on the existing structural members. It can be seen from the test results that placing the retrofitting element at the first story is limiting the interstory drift ratio to 2%. It is observed that it exceeds 2% when it is moved to the upper stories. Once the column responses are compared for each case, the column’s nonlinearity is increased when it is placed to first story instead of the other locations. On the other hand, beam nonlinearity is increased for the case where the brace is put at the seventh story when compared to the case that is located at the first story. In general, it can be seen that retrofitting could adversely affect the existing building if not properly designed. Moreover, brace force-displacement responses are obtained for all cases. The brace remained linear elastic for the case where the brace is placed at the first story. However, the nonlinearity of the brace can be observed when it is considered in either fourth or seventh story. According to these results, the best retrofitting solution for this case study is to place the brace to the first story.



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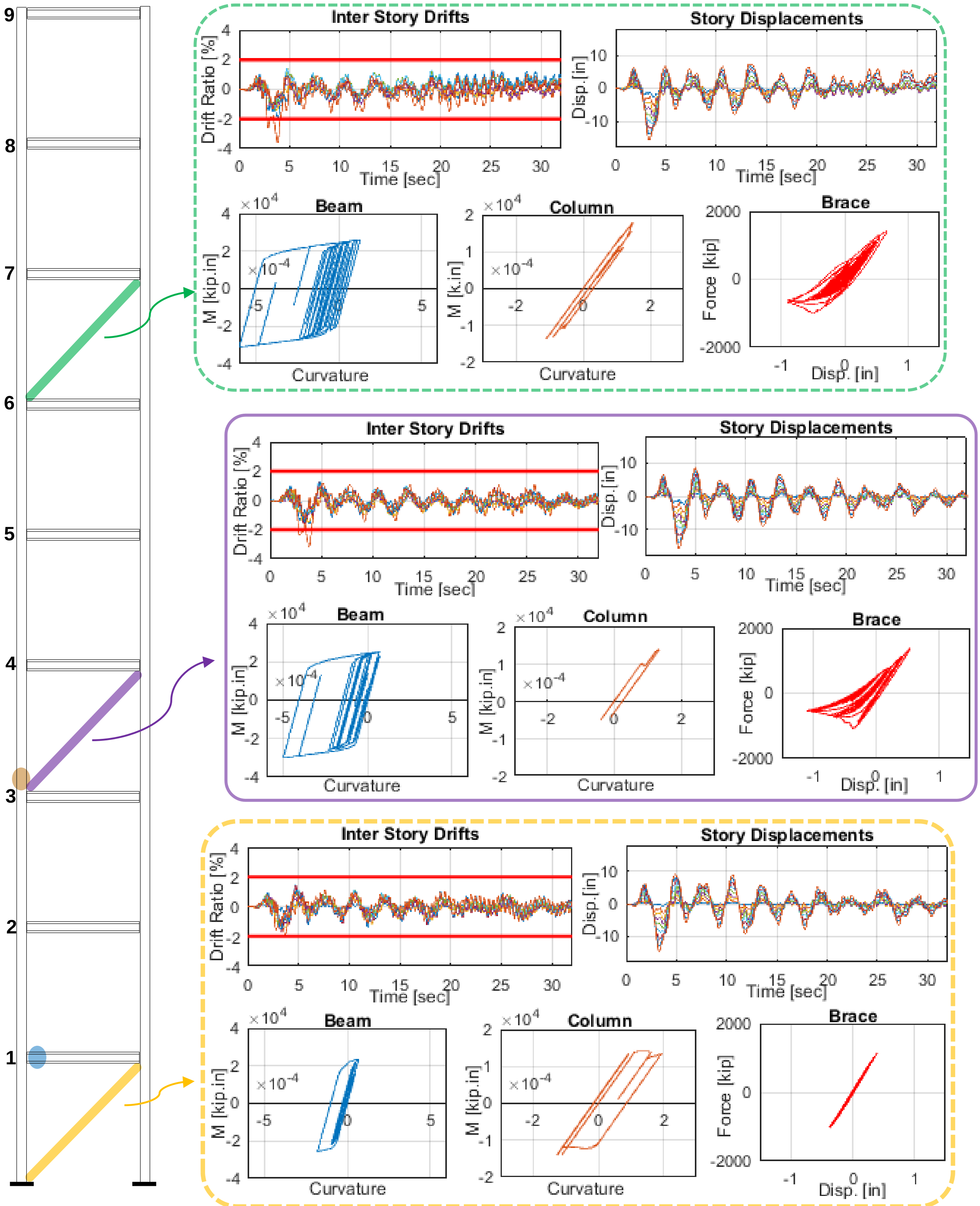


Fig. 4 – (a) 9-story MRF building with different retrofitting configurations. Selected RTHS results when the retrofitting element placed at (b) 7th story, (c) 4th story, (d) 1st story

## 4. Case Study 2: Retrofit of 2 Story Steel MRF

For this case study, a different analytical modeling assumption which has more realistic nonlinear behavior is considered. A total of four RTHS tests are conducted to see the effect of the retrofitting on a two-story MRF where not only the different locations but also different brace designs are considered.

### 4.1 Analytical Models

This study uses a two-story one-bay MRF as an analytical substructure that is modeled in OpenSees by considering distributed plasticity assumption as well. However, the element model for distributed plasticity, in this case assumes the plastic zone is located at the ends of the element with a finite length. The "beam with hinges" element that is defined in OpenSees is used to define this type of plasticity (Fig. 5a). In this element model, the interior is usually elastic, where the nonlinearity is distributed among the plastic hinge lengths and located at the ends of the elements (Fig. 5b). The element has six integration points, and the integration method is selected to be Gauss-Radau. The modified Ibarra-Medina-Krawinkler (IMK) deterioration model is considered for the nonlinear modeling of the cross-sections (Fig. 5c). This model is selected due to having an extensive database of steel W-beams and considers both stiffness and strength degradations. The yield stress of the steel material is defined to be 50 ksi.

For the two-story one-bay frame, bay width and the first story height is defined to be 15 feet where the second story height is 12 feet. For the columns and the beams, W24x131 and W27x94 steel sections are selected, respectively. For this model, the columns are also fixed with the fixed beam-column connections. The first and second floor masses are 1.25 kip-sec$^2$/in and 1.22 kip-sec$^2$/in, respectively. This model also has 2% mass and stiffness proportional damping ratio and assigned to the first two modes of the structure.

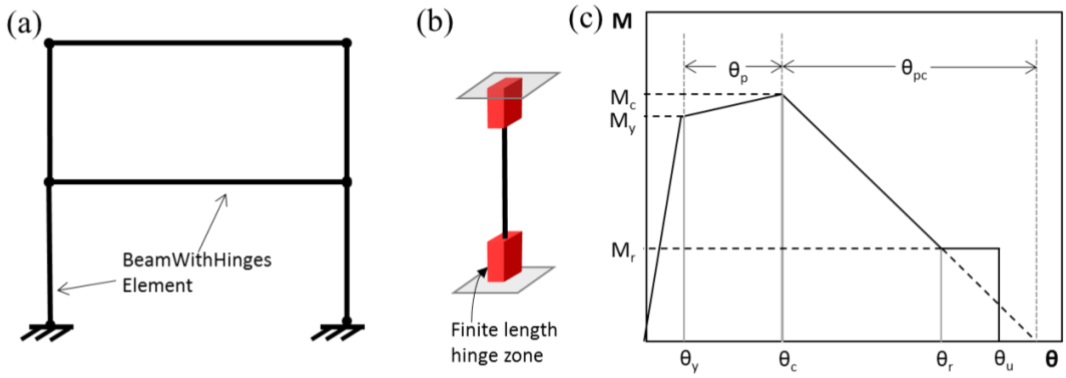


Fig. 5 – (a) Schematic of MRF model, (b) idealized element model for distributed plasticity with finite length hinge zone, and (c) modified IMK deterioration model [20]

### 4.2 RTHS Application and Results

Two types of small-scale dog-bone shaped aluminum specimens are used for this application. In total, four HS tests are considered with different configurations, and the test matrix is shown in Table 1. Type A has 35 ksi yield stress which has a cross-section of 1/8"x1/2", where Type B yields at 25 ksi stress and has 1/4"x1/2" cross-section (Fig. 1b). For the RTHS tests, the considered brace elements are scaled down with a factor of 15, and similitude laws are applied as well. One of the ground motion records of the 1994 Northridge earthquake is used to conduct the RTHS tests. The tests are starting with 100% of the original scale, and the scales are increased with 25% increments until brace failure. Explicit integration algorithms are not capable of the models that include heavily nonlinear structural response, including both stiffness and strength degradation. Therefore, implicit HHT-α with incremental limits that are modified for HS is used to conduct the RTHS tests. The integration time step is selected to be 0.001 sec.

The results from the RTHS tests with different brace sizes and configurations are presented in terms of global behavior and member responses. All tests are stopped when the brace failed. Fig. 6 shows the interstory drift ratio histories for the first and second stories of the four retrofitted cases. It can be seen from



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the responses that changing the brace location and size result in different drift responses of the structure. Configuration I is where the failure occurred at the earlier ground motion scale. For each story, the drift ratios are in an acceptable range. When the larger brace is placed at the first story as in Configuration II, interstory drift ratios are significantly reduced, and the building survived for a larger earthquake scale. For Configurations III and IV, the second story drifts are significantly reduced where stronger brace survived for larger scale earthquakes.

The global behavior is also represented by the base shear versus the drift ratio, which is shown in Fig. 7. The overall structural response through collapse can be obtained for all cases. The drift ratios for Configuration I, III, and IV are found to be 2%, where for Configuration II, it is exceeded 5%. Moreover, the strength capacity of Configuration II is increased. Analytical member behaviors are not shown here for brevity. However, it is essential to consider such local responses in future HS/RTHS testing of actual retrofitting members to see the effects of the retrofit on the rest of the structural components.

Lastly, the force-displacement response of the experimental element behaviors of the four different retrofit cases are presented. As can be seen from Fig. 8, different types of hysteretic behavior are obtained for each case. Moreover, the buckling and the fracture behavior of the braces can be observed for each brace since it is dependent on the size, location, and the nonlinear response of the columns and beams that the brace is connected.

Table 1 – RTHS Test Matrix

| Test Configuration | I | II | III | IV |
|---|---|---|---|---|
| Specimen | Type A | Type B | Type A | Type B |
| Brace Location in Frame | 1st Story | 1st Story | 2nd Story | 2nd Story |

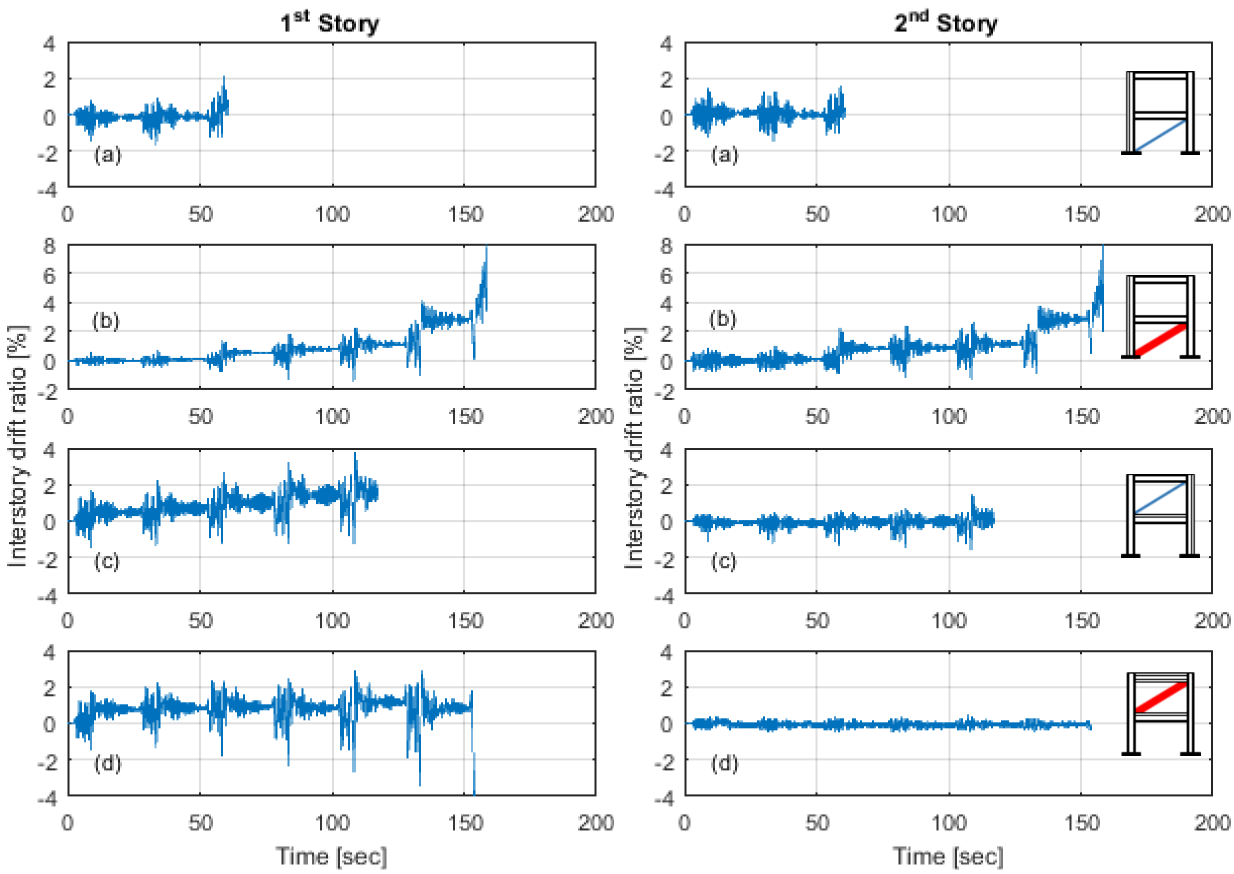


Fig. 6 – Interstory drift ratio history for 1st and 2nd stories of the four retrofit cases: (a) Configuration I (b) Configuration II (c) Configuration III and (d) Configuration IV



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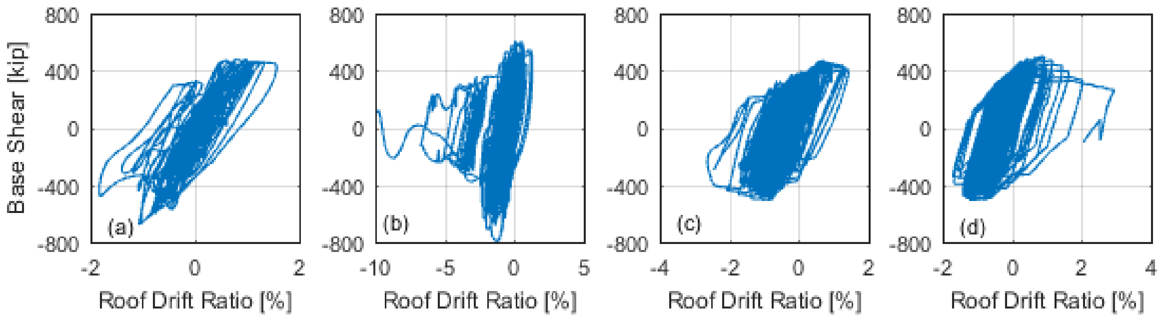


Fig. 7 – Base shear-roof drift ratio for: (a) Configuration I (b) Configuration II (c) Configuration III (d) Configuration IV

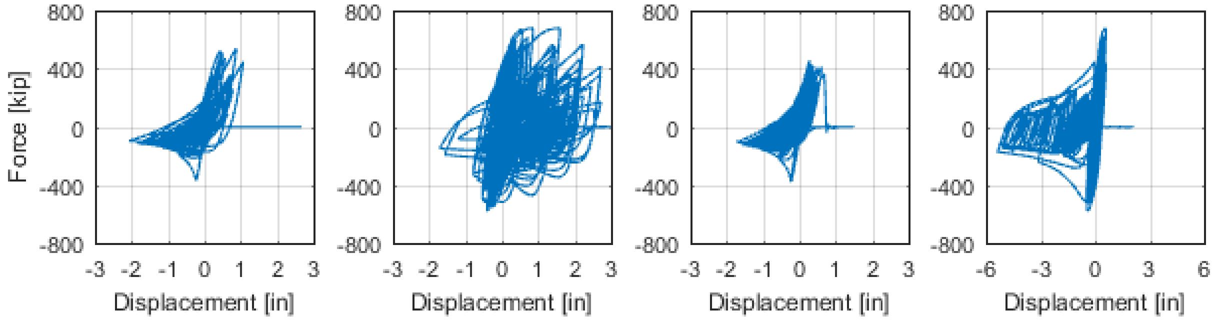


Fig. 8 – Force-displacement relationships for brace as obtained from the four different RTHS tests for: (a) Configuration#1, (b) Configuration#2, (c) Configuration #3, and (d) Configuration#4

## 5. Summary and Concluding Remarks

The objective of this presented study is to extend the use and show the advantages of RTHS for informing and selecting the best retrofitting decision of existing structures. In an HS setting, the existing structure can be analytical substructure, and the retrofitting element can be represented by the experimental elements. By doing this, several retrofitting configurations can be tested with realistic analytical models, which can lead to more economical and reliable decisions for retrofit design.

Two case studies are presented where MRFs are retrofitted with conventional braces. The existing structural response should be represented accurately since the retrofitting can inform additional demands to the structural elements. Moreover, due to having limited strength and ductility, the existing structural members can experience higher nonlinear responses. Thus, detailed computational models are needed to model MRFs. In general, two types of modeling assumptions are considered for MRFs, namely concentrated plasticity and distributed plasticity. For the first case study, a brief computational time study is conducted first with different number of story MRFs with distributed plasticity models where fiber cross-sections are distributed among the cross-section of the element. The material behavior is assumed to be bilinear for steel. Then, the nine-story one-bay MRF is selected to be retrofitted, and three RTHS tests are conducted. By placing the same type of small-scale specimen to different floors, global and local structural behaviors are obtained. For the second case study, the MRF is modeled with the distributed plasticity assumption as well. However, the nonlinearity is only distributed through a finite length zone, which is the plastic hinge length, whereas the inner part of the element is considered to be elastic. On the other hand, the modified IMK deterioration model is considered to model the material which is calibrated with a large database of experimental study for W-sections. This modeling considers stiffness and strength deterioration, which makes the analytical model more complex for real-time applications. Due to this, two-story one-bay MRF is considered to be retrofitted with different brace size and locations. A total number of four RTHS are conducted, and the global and local structural responses are presented.



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These two case study results are showing that the HS can be a very reliable, feasible, and cost-effective experimental method to inform and select the optimum retrofitting designs. The effect of retrofitting element for different location and sizes are demonstrated with different modeling assumptions. The focus here is to show how retrofitting configuration could lead to different member behavior and global responses, rather than showing a finalized design for the selected MRFs. In conclusion, nonlinear models for retrofitting decision are needed, and the advantages of RTHS can be extended with such engineering applications.

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