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HYBRID TESTING OF A STEEL MOMENT RESISTING FRAME RETROFITTED WITH HIGH PERFORMANCE FIBER REINFORCED CONCRETE INFILL PANELS

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ABSTRACT

This paper discusses about the application of a new infill panel system made of high-performance fiber reinforced cement composites as a retrofit system for existing steel moment-resisting frames. The infill system is used to protect the structural system under an extreme seismic event and should secondarily be able to provide protection to non-structural systems. The effectiveness of the proposed seismic retrofit system is evaluated based on state-of-the-art hybrid simulation method of a 2-story steel moment resisting frame designed in the 1980s. A coupled simulation technique is used to analyze the retrofitted structure using the OpenFresco framework for hybrid testing. The effectiveness of the proposed HPFRCC infill panel system as a retrofit for steel moment-resisting frames is demonstrated through two series of tests with ground motions at various levels that represent intensities of interest of engineering profession.

Introduction

After recent earthquake events (Northridge 1994, Kobe 1995) the need for seismic retrofit of steel frame structures became evident particularly for frames with fracture critical connections. At the same time critical facilities (hospitals, emergency response centers, schools, etc) must remain in full service during any seismic event as well as during repair activities after a seismic event, i.e., residual story drifts should be minimized as much as possible with a use of a retrofit system.

Over the years many different retrofit techniques were developed for existing steel moment resisting frames including fuse elements (Leelataviqat et al. 1998), self-centering systems (Christopoulos et al. 2002) buckling restraint braces (Wada et al. 1998, Lopez et al. 2002, Uang and Kiggins, 2003) and viscous dampers (Soong and Spencer, 2002).

Recently, infill systems made of innovative materials have been proposed for retrofit of existing steel structures. Jung et al. (2006) proposed three prefabricated infill panels made of polymer matrix composite (PMC) material that is easy to be used in construction and can provide substantial structural enhancement. Kesner and Billington, (2005) demonstrated that high-performance Fiber reinforced Cementitious Composites (HPFRCC) can be used to increase

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strength and lateral stiffness of existing structures. The ease of replacement for infill panels is considered to be a potential advantage over alternative retrofit techniques.

This paper discusses the effectiveness of a recently developed infill panel retrofit system described in detail in Olsen and Billington, (2009) made with ductile high performance fiber reinforced concrete (HPFRC) for existing steel moment frames through the design and evaluation of an experimental series of a 2-story steel moment frame designed in 1980s. The hybrid simulation testing technique is used for this series of tests. Since experiments are under way at the NEES facility of the University of California, Berkeley (UCB) and experimental results are not available yet, results from structural finite element coupling through the experimental software framework, OpenFresco (Takahashi and Fenves, 2006; Schellenberg et al., 2007; Schellenberg, 2008) are presented here in.

Description of Prototype and Test Frame

The prototype structure that is used for seismic retrofitting with HPFRC panels is a two story 3-bay office building with perimeter steel moment resisting frames designed based on 1980s seismic provisions. The building, which is shown in Figure 1 in plan view and elevation, does not meet the retrofit objectives based on ASCE 41, FEMA 351, guidelines. For this reason, the east west (EW) moment resisting frame of the building is retrofitted with HPFRC infill panels that are treated as a degrading system per ASCE 41 provisions. The building in the EW direction has a predominant period of 0.75sec.

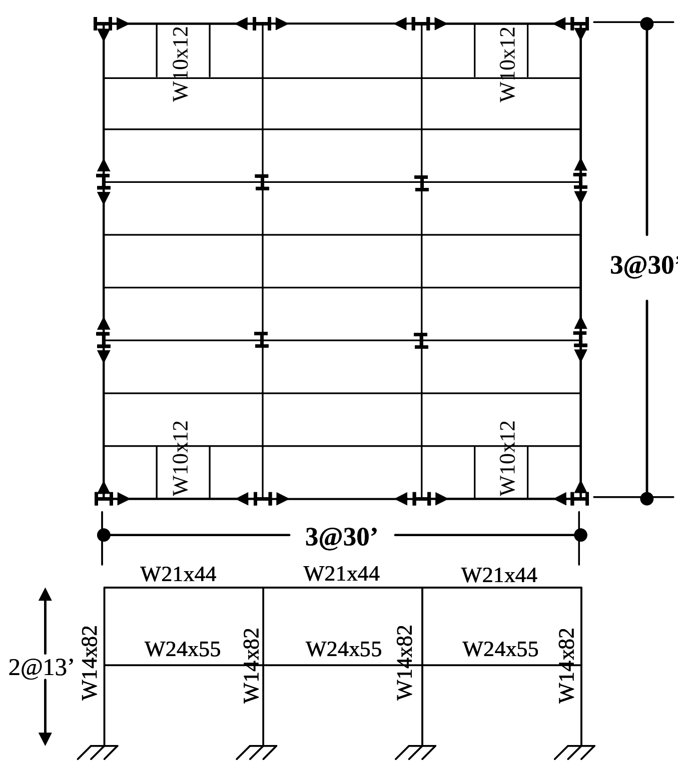


Figure 1. Plan view and elevation of the 2-story prototype structure

The steel moment frame is retrofitted with five double panels per story installed in the first bay as shown schematically in Figure 2. A 2/3 scale model of the retrofitted EW steel moment resisting frame was designed and fabricated for testing at the NEES facility at UCB. The beams of the prototype SMRF were made of A36 steel material but since W sections are available in A992Gr. 50 steel material the difference in yield strength was considered when the prototype frame was scaled for strength. In order not to change the collapse mechanism of the scaled frame versus the prototype structure it was decided to keep the column to beam strength ratios per joint the same both for scaled and prototype frame. Testing involves hybrid simulation with the physical subassembly to be the one bay sub-frame that includes the HPFRC panels .and the numerical subassembly to be the other part of the moment resisting frame modeled in the Open System for Earthquake Engineering Simulation (OpenSees, 2009) platform. Figure 2 shows a schematic representation of the hybrid simulation scheme together with the experimental and numerical and physical subassemblies. The two horizontal translational degrees of freedom at each one of the floor levels of the experimental subassembly are controlled during the hybrid test.

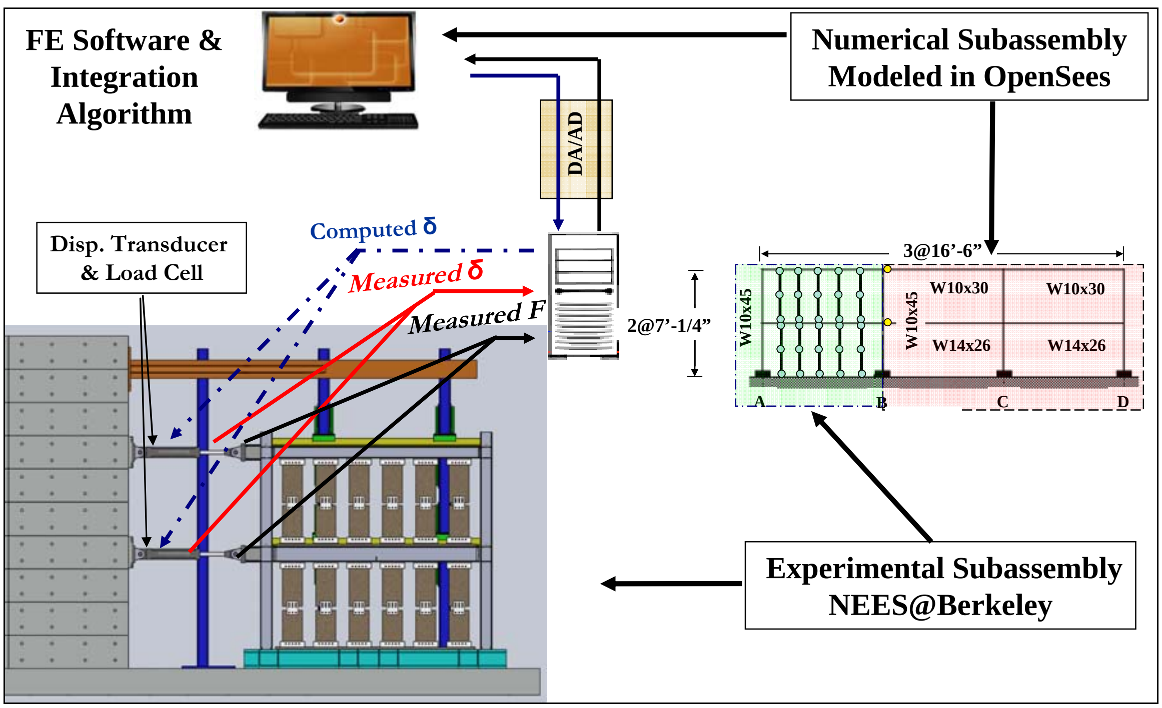


Figure 2. Schematic representation of hybrid simulation with the retrofitted 2-story frame

Test Frame and Experimental substructure

The 2/3 scale frame of the prototype SMRF is designed with a W10x45 exterior and interior columns and W14x26 and W10x30 first and second floor beams. These sizes are based on similitude laws for strength and stiffness based on Moncarz and Krawinkler, (1981). The predominant period of the retrofitted scale frame is 0.39sec. The $b_f/2t_f$ and h_f/t_w ratios of the selected scaled sections are almost the same with the ones of the prototype SMRF, i.e. the deterioration parameters of the components of the scaled frame represent reasonably well the ones of the prototype frame. The geometry and basic dimensions of the physical subassembly is shown in Figure 3a. Five double HPFRC infill panels are installed per story. The fabricated panels are shown in Figure 3b. Connection details of the panels are described in detail in Olsen and Billington, (2008) and Lignos et al. (2009). Even if the prototype SMRF was designed based on Pre-Northridge seismic provisions the four steel moment connections in the physical subassembly are designed as standard welded unreinforced flange – bolted web connections (see Figure 3c) per FEMA-350 provisions to avoid any control instability of actuators during testing because of fracture. Fracture is simulated in the numerical portion of the hybrid model though recognizing the possibility of having fracture(s) at design level earthquake events.

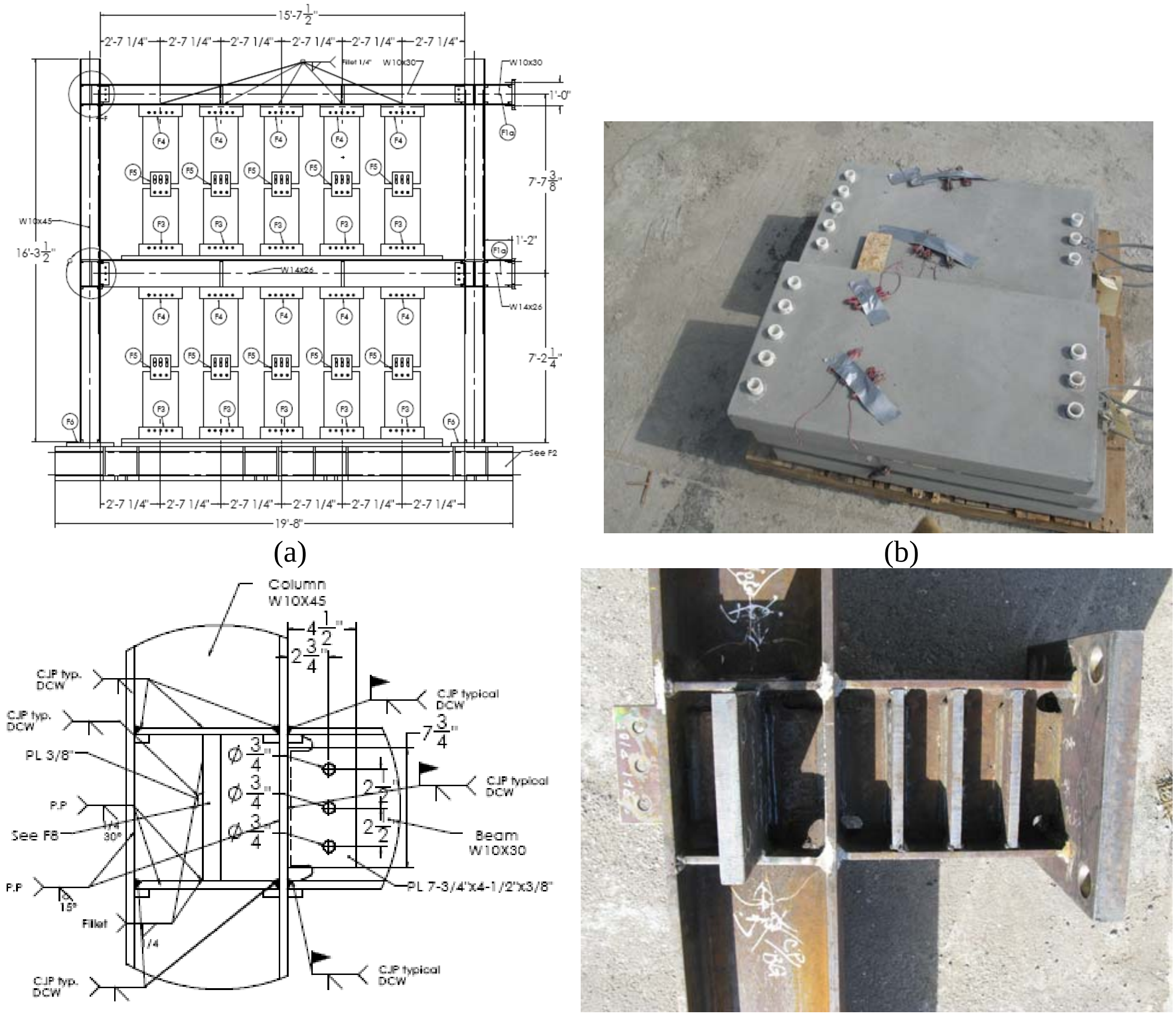
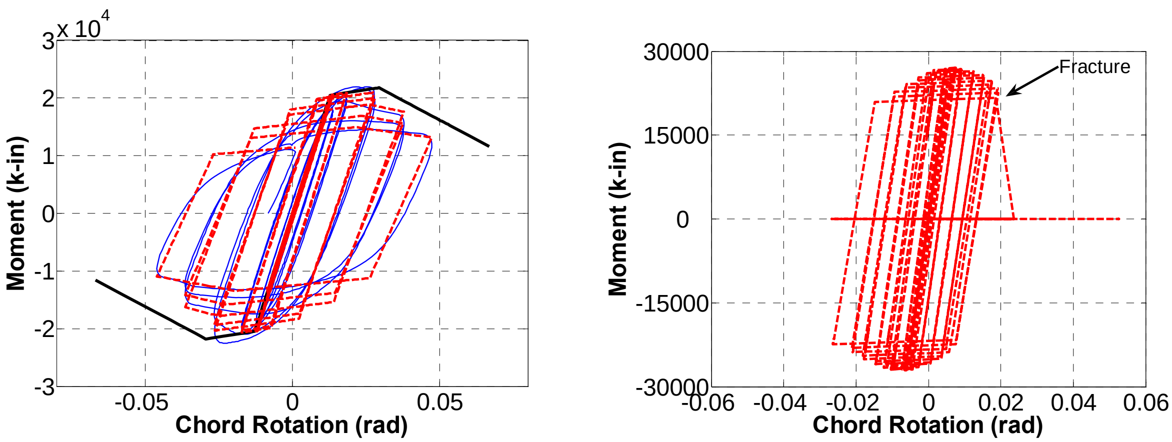


Figure 3. Experimental subassembly; (a) basic geometry and dimensions; (b) concrete panels after fabrication; (c) typical connection detail of the experimental subassembly; (d) actuator adaptor connection

During hybrid testing the two horizontal translational degrees of freedom are the control quantities and the physical subassembly is connected with a link that is shown in Figure 3d designed to behave elastically. Two 220kips dynamic actuators impose the computed displacements and also measure the force and displacement quantities from the physical subassembly (see schematic representation in Figure 2).

Component Modeling

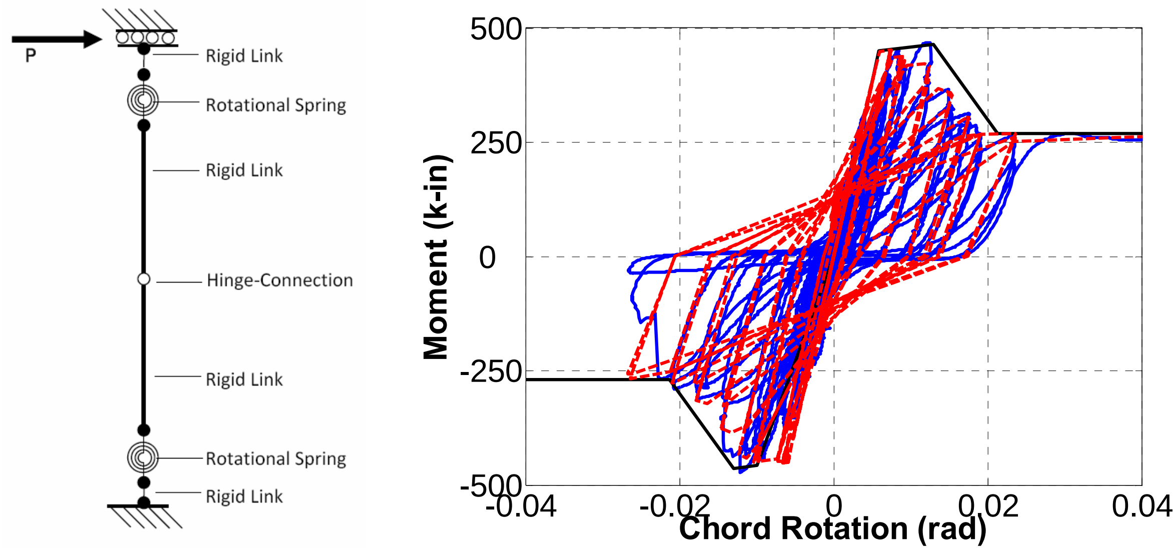
The 2-dimensional test frame is modeled in OpenSees with elastic beam column elements that have concentrated plasticity springs at their ends. The hysteretic response of these springs is bilinear. The springs simulate component deterioration based on the modified Ibarra Krawinkler model (Lignos and Krawinkler, 2009). Deterioration parameters of the components are determined from relationships for deterioration modeling proposed by Lignos and Krawinkler, (2009). These relationships have been derived from a recently developed steel database of steel components for deterioration modeling (Lignos and Krawinkler, 2007, 2009). A calibration example using the modified IK model is illustrated in Figure 4a. The modified IK deterioration model is able to simulate brittle fracture with an ultimate deformation parameter θ_u that is set to be 2% for one end of the first floor exterior beam, recognizing the possibility of having a brittle fracture at an early inelastic cycle as reported in FEMA-351. An illustration of the brittle failure of the connection is shown in Figure 4b.



(a) (b)

Figure 4. Illustrations of modified Ibarra Krawinkler deterioration model; (a) calibration example of a beam that fails in a ductile manner (data from Taejin et al. 2000); (b) brittle failure

The analytical model developed to capture the hysteretic response of the HPFRC infill panels is shown in Figure 5a. Two rigid links are connected together with a hinge connection in the middle that allows vertical movement of one panel with respect to the other. Each infill panel at its one end has a concentrated plasticity spring that utilizes the modified Ibarra-Krawinkler deterioration model with peak-oriented hysteretic behavior. Experimental data provided by Hanson and Billington (2009) are used to calibrate the infill panel model. The calibrated moment rotation diagram of the HPFRC panel is shown in Figure 5b. P-Delta effects are simulated numerically with a leaning column that does not contribute to the lateral stiffness of the building. Two percent Rayleigh damping is assigned to the bare steel moment frame. For the retrofitted SMRF 3% Rayleigh damping is assumed in order to consider the effect of concrete panels on viscous damping of the building.



(a) (b)

Figure 5. (a) Analytical model for HPFRC infill panel, and (b) typical calibration of infill panel model with experimental data (data from Hanson and Billington, 2009)

Testing Phases

Validation of the proposed high HPFRC infill panel system for retrofitting of existing SMRF involves experimental testing with the scale frame discussed earlier. Two testing phases are scheduled for this reason. In both phases, the same SMRF is used but the HPFRC infill panels are replaced between the two phases. In both phases the ground motion records are scaled appropriately to represent levels of intensity that are of particular interest for the engineering profession. The two testing phases are summarized in Tables 1 and 2, respectively.

Table 1. Experimental program for testing phase I

Level of Intensity	**Notation**	**Gr. Motion Intensity**	**Earthquake Record**
Service Level	SLE	30%	Petrolia (Cape Mendocino, 1992)
Design Level	DLE-I	70%	Petrolia (Cape Mendocino, 1992)
Design Level	DLE-II	100%	Canoga Park (Northridge, 1994)

Table 2. Experimental program for testing phase II

Level of Intensity	**Notation**	**Gr. Motion Intensity**	**Earthquake Record**
Service Level	SLE	30%	Petrolia (Cape Mendocino, 1992)
Maximum Considered Level	MCE	105%	Petrolia (Cape Mendocino, 1992)
Collapse Level	CLE	100%	JR Takatori (Kobe, 1995)

Phase I is concerned with seismic performance of the retrofitted 2-story frame during two subsequent design level earthquakes (DLE). Phase II involves experimental testing with a maximum considered earthquake (MCE) based on the scaled component of the Petrolia record from the Cape Mendocino earthquake in 1992. After the end of this event the frame is subjected to the unscaled component of the JR Takatori record from the 1995 Kobe earthquake in Japan.

Coupled Simulation using OpenFresco

Results of the experimental program discussed in this paper will be available in January 2010. However simulation results for the two testing phases discussed in the previous section are available through a new simulation method that couples two or more displacement-based structural finite element analysis programs together through a generic adapter element approach, which is implemented in the Open-source Framework for Experimental Setup and Control (OpenFresco)(Takahashi and Fenves, 2006; Schellenberg et al., 2007). The numerical subassembly of the 2-story SMRF shown in Figure 6 is analyzed in OpenSees and is connected with a generic Super-Element within OpenSees that represents the physical subassembly tested in the laboratory (Master Program). The physical subassembly itself is modeled in OpenSees but as a separate input file (Slave Program). The master program imposes boundary conditions on all the subassemblies. The 2-node adapter element connects to the interface nodes of the physical subassembly in the slave program and is responsible for imposing trial displacements on such subassembly. Details about the couple simulation theory, adapter elements and implementation details can be found in Schellenberg et al. (2007, 2008).

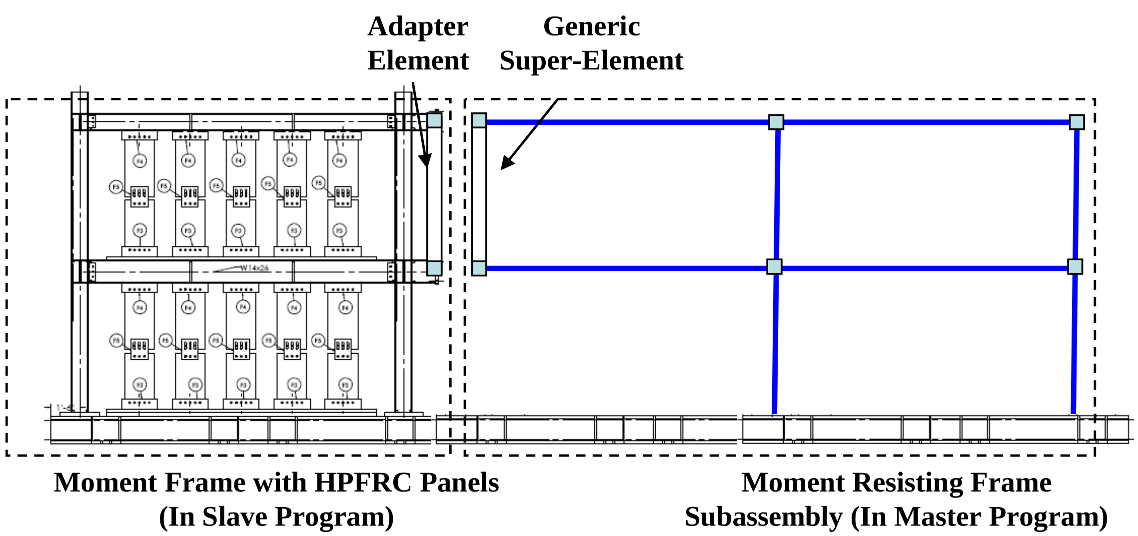


Figure 6. Coupled simulation of 2-story steel moment frame with HPFRC infill panels

Assessment of Coupled Simulation Results

During DLE-I and DLE-II motions of Phase I the retrofitted frame does not exceed a maximum story drift ratio (SDR) of about 1.5% compared to 2.5% of the bare frame ground motions. Figure 7 illustrates the drift histories of the bare versus retrofitted frame for DLE-I level of intensity. This indicates that during the DLE events no fracture occurs in any of the steel moment connections. At the end of both DLE-I and II levels of Phase I the residual drift ratios of the retrofitted frame are almost zero compared to about 0.6% of the bare frame at both stories.

After replacing the HPFRC infill panels with new ones (Phase II) and subjecting the retrofitted frame to the MCE level ground motion (105% of the unscaled Petrolia record) peak story drift ratios of the retrofitted frame stay below 2.5% at the first story. Due to fracture of the connection at the end location of the first floor beam of the bare frame, SDR increase of the bare frame are about 30% larger compared to the retrofitted one indicating that the proposed retrofit system is effective for retrofitting existing steel moment frames with fracture critical beam column connections. Similarly, residual story drifts of the retrofitted frame are reduced at about 50% compared to the ones from the bare frame (see Figure 8).

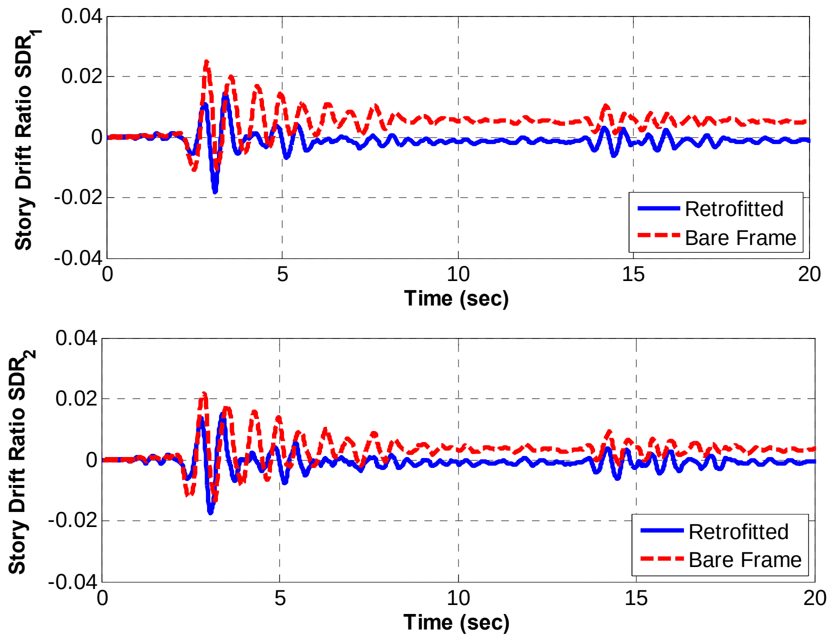


Figure 7. Comparison of story drift ratio histories between bare and retrofitted 2-story frame for during design level earthquake I of testing Phase I

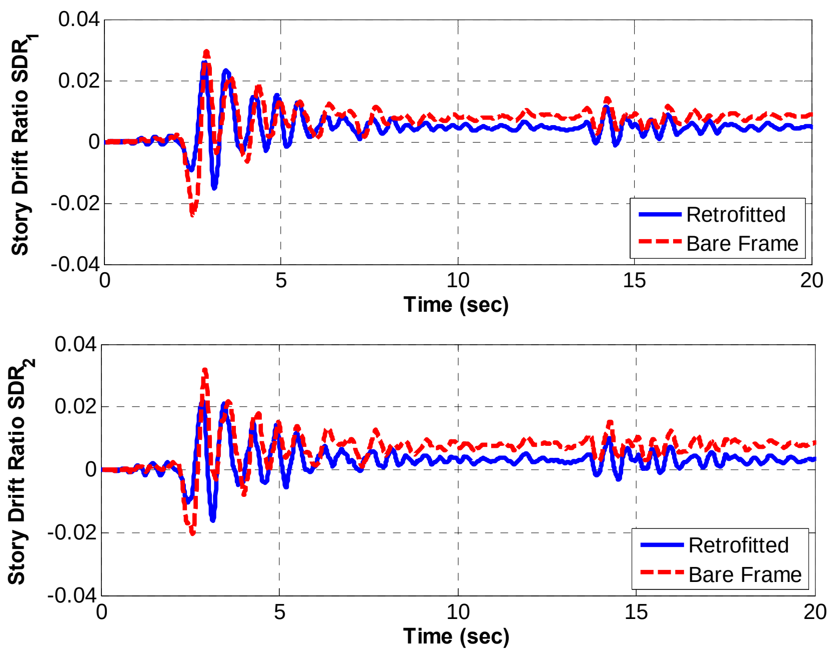


Figure 8. Comparison of story drift ratio histories between bare and retrofitted 2-story frame for during maximum considered level earthquake of testing Phase II

Summary and Conclusions

This paper is concerned with the seismic performance evaluation of a 2-story steel moment frame designed based on pre-Northridge seismic provisions and is retrofitted with high performance fiber reinforced concrete infill panels. The 2-story steel moment frame is currently erected for experimental testing using the hybrid simulation technique at the NEES facility at University of California, Berkeley. At this point the seismic evaluation of the retrofitted steel moment frame is carried out with a coupled simulation method that couples two or more displacement-based structural finite element analysis programs together through a generic adapter element approach. Based on the "virtual" hybrid simulation for the two scheduled testing phases:

- Peak story drift ratios of the retrofitted frame are reduced by about 30% compared to the peak story drift ratios of the bare frame when they are both subjected to a design and maximum considered event. The implication is that brittle fracture of beam to column moment connections does not occur during design level events or is delayed during maximum considered earthquake events.
- Residual deformations of the retrofitted frame are almost zero for a design level event and are reduced by about 50% during a maximum considered event.

Hybrid simulation testing of the retrofitted 2-story steel moment frame is under way at the NEES experimental facility at Berkeley in order to confirm and improve pre-test analytical simulations and validate if the proposed HPFRC infill panel retrofit system can be used for retrofit of existing steel moment frames.

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