Seismic Performance and Design of Linked Column Frame System (LCF)

M. Malakoutian & J.W. Berman
Department of Civil and Environmental Engineering, University of Washington, Seattle, WA, USA

P. Dusicka & A. Lopes
Department of Civil and Environmental Engineering, Portland state University, Portland, OR, USA

ABSTRACT:
The Linked Column Frame (LCF) is proposed as a steel seismic load resisting system consisting of two components: a primary lateral system made up of dual columns interconnected with link beams (denoted linked columns); and a secondary moment frame. A design procedure is proposed that ensures the links of the linked columns yield at a significantly lower story drift than the beams of the moment frame, enabling design of this system for three distinct performance states: linearly elastic; rapid return to occupancy, where only link damage occurs and quick link replacement is possible; and collapse prevention. Results of nonlinear response history analyses show that the LCF system has the capability to limit economic loss by reducing structural damage and allowing for rapid return to occupancy. Also, the LCF system and component behaviour s are being investigated experimentally through hybrid testing and an overview of the large-scale 2-story test program is presented.

Keywords: Rapid return to occupancy; Link column frame; Nonlinear response history analysis

1. INTRODUCTION

Modern design codes and structural systems have been successful in preventing collapse and loss of life in recent earthquakes. However, damage to conventional structural systems continues to result in significant economic losses. New systems are needed that integrate collapse prevention with post event ease of repair.

In this paper, a new seismic load resisting system, the linked column frame system (LCF), is being studied and shown in Figure 1. During the initial development of the system, non-moment transferring connections were introduced at all columns to foundation locations and in strategic beam to column locations (Dusicka and Iwai 2007). This system consists of easily replaceable link beams between two closely spaced columns and an adjacent flexible moment resisting frame. The LCF links behave similarly to links in eccentrically braced frames and dissipate energy while limiting the inelastic deformation and related damage to the structural members of the adjacent moment resisting frame.

In this study, the seismic performance of the LCF system is investigated through numerical simulation. Nonlinear response history analyses of several prototype LCF designs subjected to a range of ground motions representing different seismic hazard levels are discussed.

Additionally, experimental research is needed to evaluate the constructability of the LCF system and to validate analytical models. This ongoing research includes studies of the link, link-to-column connection details, system response, column behaviour and investigation of construction methods. A brief overview of the experimental program is provided here.

2. LINKED COLUMN FRAME SYSTEM

The proposed LCF system shown in Figure 1 consists of two components that work essentially in parallel to provide the desired seismic response. The primary lateral force resisting system, denoted the linked column (LC), is made up of two closely spaced columns connected with replaceable link beams. The secondary lateral force resisting system is a moment resisting frame (MF) that also acts as part of the gravity load system. The moment resisting frame is designed to be relatively flexible by utilizing beams with fully restrained connections at one end and simple connections at the other. The links are designed to act as yielding elements and provide a stable source of energy dissipation until large drifts are reached and plastic hinging occurs in the MF beam.

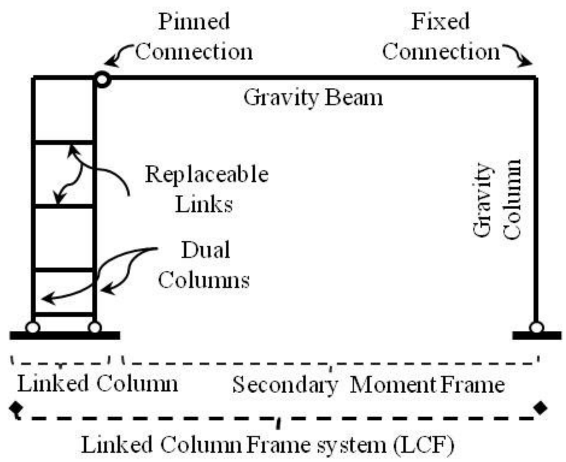


Figure 1. Plane Frame Elevation of a Building Bay with the Linked Column Frame System

The pushover response of an idealized LCF system and the contribution of its components, i.e., the moment frame and the linked column, are shown in Figure 2. As shown, the resulting lateral response of the LCF system shows three performance objectives, which include: Immediate occupancy (IO) following an earthquake with a 50% probability of exceedance in 50 years (50% in 50 year), requiring both the linked column and moment frame to remain elastic (Damage State 1 (DS1)); Rapid return to occupancy (RRO) following an earthquake with a 10% probability of exceedance in 50 years (10% in 50 year) where the moment frame remains completely elastic but plastic hinges develop in the links, which may necessitate their repair or replacement (Damage State 2 (DS2)); and Collapse prevention (CP) following an earthquake with a 2% probability of exceedance in 50 years (2% in 50 year) where significant yielding and plastification of the links and moment frame beams may occur (Damage State 3 (DS3)).

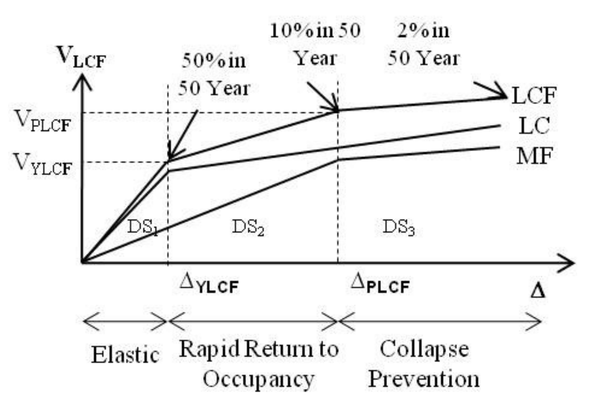


Figure 2. Idealized LCF and Component Pushover Curves

To obtain these three distinct performance objectives, a drift requirement is used such that $1.2 < \Delta_{YMF}/\Delta_{YLC} < 3$, where Δ_{YLC} is the roof displacement at which the links of the linked columns yield and Δ_{YMF} is the roof displacement at which the beams of the moment frame yield. The effectiveness of this simple displacement constraint that ensures link yielding occurs before moment frame yielding is evaluated below using the prototype designs and response history analysis. The Δ_{YLC} and Δ_{YMF} may be estimated via pushover analysis using common structural analysis software or by using separate plastic analyses of the linked columns and moment frames and their elastic stiffness. To ensure that the development of the first plastic hinge in the beams occurs after significant inelastic link deformation, the beams in the LCF system are pinned at the connections to the LC as illustrated in Figure 1.

3. OVERVIEW OF PROTOTYPE DESIGNS

Prototype LCF systems were designed for modified versions of the SAC buildings (Gupta and Krawinkler,1999). The buildings considered were 3-, 6-, and 9-stories tall with uniform 3.96 m (13 ft) story height. Story masses, plan dimensions, and dead and live loads were consistent with the SAC buildings. The LCF systems were designed for a site class D soil and adjusted maximum considered earthquake spectral response parameters at 0.2 sec. and 1 sec. In order to maintain the overall plan dimensions of the SAC buildings, beam lengths had to be decreased due to the introduction of the linked column. Two bays of LCs were used for the 3- and 6-story LCFs as shown in Figure 3. In total, nine different LCF systems were designed. Their overall designs are discussed here and their fundamental behaviour s are compared and contrasted following the analytical model development section below. Table 1 shows the primary characteristics of the different designs. The naming convention for the prototype variations in Table 1 starts with the number of stories. For the 3-story LCFs this is followed by a descriptor to show the relative strength of the LCs with respect to the MFs (SLC for strong LC, SMF for strong MF and SS for the same strength of LC and MF). For the 6-story LCFs, the number of stories is followed by a descriptor for the type of links that have been used (S for shear link, I for intermediate link and F for flexural link) followed by the LC column spacing in inches. For the 9-story LCFs, the number of stories is followed by the number of LCs in one frame line, which is either 2 or 3.

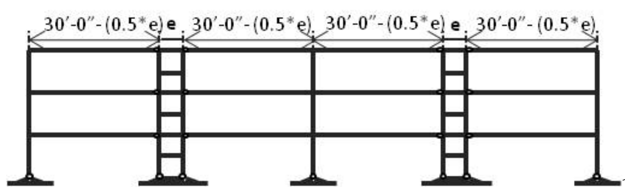


Figure 3. LCF Model Based on SAC Model

Table 1. Design Characteristics of LCF Buildings

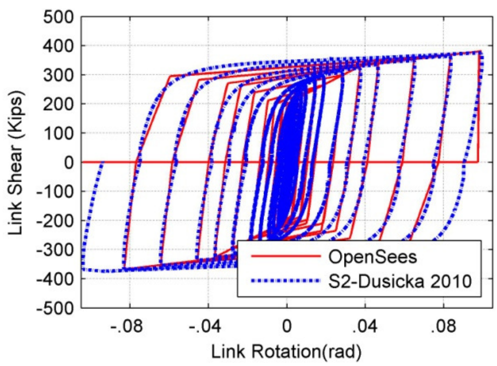
Name	No. of Stories	No. of LCF Bays	LCF Column Spacing (m)	Link classification
3-SLC	3	2	1.52	Shear
3-SMF	3	2	1.52	Flexure
3-SS	3	2	1.52	Shear, Intermediate, Flexure
6-S-80	6	2	2.03	Shear
6-I-100	6	2	2.55	Intermediate
6-I-120	6	2	3.05	Intermediate
6-F-120	6	2	3.05	Flexure
9-2b	9	2	3.05	Shear, Intermediate, Flexure
9-3b	9	3	3.05	Shear, Intermediate, Flexure

4. ANALYTICAL MODELS

The structural analysis software OpenSees (Mazzoni et al., 2009) was used to develop analytical models of the prototype LCF systems. The models are an assembly of beam-column elements with cyclic load-deformation behaviour s calibrated to represent the behaviour s of the links, beams and columns. The selected method for modeling the link behaviour uses a distributed plasticity beam-column element with a fiber cross-section that controls the axial and flexural response and is aggregated with an independent nonlinear shear force vs. shear deformation section. For axial and flexural response, the material stress-strain behaviour is specified and applied to the fibers. For the shear response, a shear stress-strain behaviour is specified and simply multiplied by the shear area, which for wide-flange sections is the web area (Malakoutian, 2011).

The hysteretic material model available in OpenSees was applied to the fiber cross-section and parameters were calibrated to experiments. Degradation of response was modelled using additional uniaxial material models combined with the hysteretic material model. Two different models for deterioration were chosen because of the difference in the observed degradation of shear and flexural links.

Figure 4 shows the cyclic shear force vs. total link rotation response of the link element with calibrated material properties along with the cyclic shear force vs. total link rotation response of selected link test specimens from Dusicka and Lewis (2010).



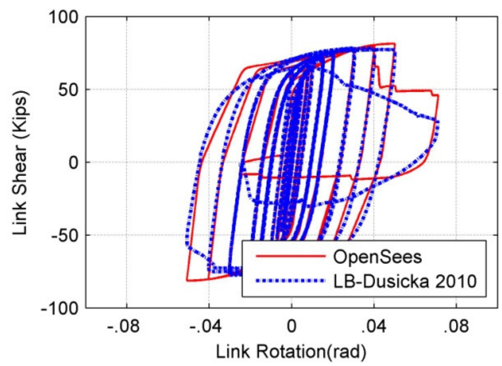


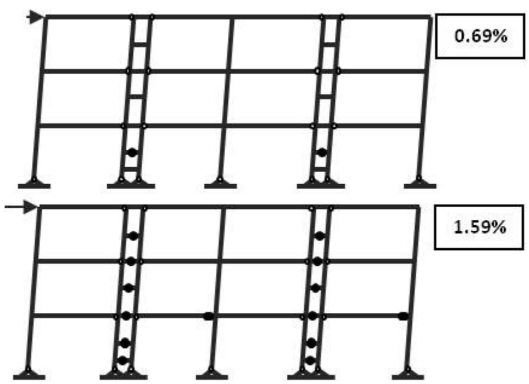
Figure 4. Comparison of Experimental Link Shear vs. Link Rotation Results with the Developed OpenSees Model. (a) Shear Link (S2, W12X96) (b) Flexural Link (LB, W12X22) (Dusicka and Lewis, 2010)

The beams and columns were modelled using force-based distributed plasticity beam-column elements with fiber cross-sections. The steel material for the columns and beams was modeled using the Steel02 Material available in OpenSees, which is a Giuffr-Menegotto-Pinto model. An approximate yield stress of 345 MPa (50 ksi) was used with 2% strain hardening of as these are typical values for the steel used for rolled wide flange shapes. The value of the parameter R_0, which controls the sharpness of the transition from the elastic to plastic branches in the Giuffr-Menegotto-Pinto model, was taken as 20.

Monotonic and cyclic pushover analyses were performed on the LCF models using a lateral load distribution determined by the code-prescribed equivalent lateral forces applied at the floor levels to explore the system behaviour . The progression of yielding and cyclic pushover response of the system for LCF-3SLC is shown in Figure 5. As shown, the first link plastic hinge forms at a roof drift of 0.69% and the first plastic hinge develops in a beam at a roof drift of 1.59%, where a link plastic hinge is when the plastic shear strength is achieved and a beam plastic hinge is when the plastic moment is achieved. At 5% drift there is extensive link and beam plastification. The large difference between the story drifts at which the links and beams develop plastic hinges enables the structural designer to specifically design for the two different performance objectives of rapid return to occupancy and collapse prevention.

5. FUNDAMENTAL CHARACTERISTICS OF THE PROTOTYPE LCF SYSTEMS

As described previously and shown in Table 1, several LCF systems were designed at 3-, 6- and 9-stories. For each system, three different pushover analyses were done: pushover analysis of the system with hinges at the beam ends; pushover analysis of the system with the hinges at the link ends; and pushover analysis of the complete system. These analyses enable approximation of the contributions of the LCs and MFs to the total response. The resulting characteristic system values are shown in Table 2 for all LCF designs.



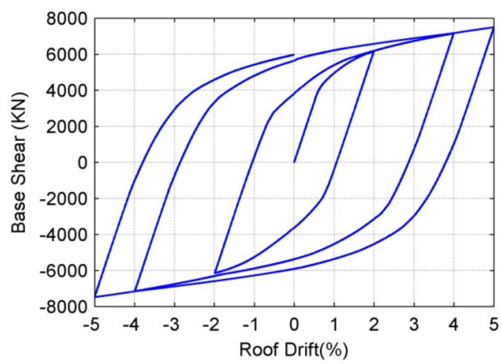


Figure 5. Pushover Response of LCF (a) Pattern of Plastic Hinge Formation (b) Cyclic Analysis

Three different 3-story LCF systems were designed for this study. These systems were designed such that each has a similar overall behaviour including the total strength, V_p, and the fundamental period. However, each 3-story system has different types of links (shear, flexural or intermediate), and therefore, different relative strengths of the LCs with respect to the MFs.

Four different 6-story LCF systems were investigated. They again were designed to have similar overall behaviour; however, these systems not only differ in link type, but also in the spacing of the linked columns and corresponding link lengths. Various link lengths were employed to examine the impact of overturning moment on LCF behaviour.

In the 9-story designs, two different layouts were chosen as discussed previously, one with two sets of linked columns (LCF 9-2b) and the other with three sets of linked columns (LCF 9-3b), however, the link lengths were the same.

Table 2. Fundamental Characteristics of 3-, 6- and 9-Story LCF

Name	$T_{(sec)}$	$K_{e\ (KN/m)}$	$\Delta_{YMF/}\ \Delta_{YLC}$	$V_{YMF/}\ V_{YLC}$
3-SLC	0.81	37541	1.84	0.52
3-SMF	0.85	37009	2.75	2.07
3-SS	0.84	37415	2.46	0.96
6-S-80	1.29	38259	1.61	0.63
6-I-100	1.24	38340	1.52	0.39
6-I-120	1.23	38283	1.71	0.26
6-F-120	1.23	38607	1.62	0.32
9-2b	1.44	32755	1.19	0.57
9-3b	1.61	36162	1.28	0.41

6. DYNAMIC RESPONSE RESULTS

After establishing the basic characteristics of the LCF systems considered, nonlinear response history analysis was performed on each. Three suites of 20 earthquake ground motion records were used in the nonlinear analyses. The ground motions were those developed in the SAC project for the Los Angeles site by Somerville et al. (1997) for soil type D. These three suites of ground motions represent three seismic hazard levels: 50% in 50 year, 10% in 50 year, and 2% in 50 year earthquakes scaled to target spectral acceleration values at four periods by Somerville et al. (1997). Figure 6 shows the median and 84th percentile values for the maximum story drift, link rotation and beam rotation obtained for the ground motions for each hazard level just for LCF-3SLC due to space limitation. The other eight buildings have similar trends.

As shown in Figure 6, the LCF is capable of meeting drift limits since the story drifts for each hazard level are within ranges that would be considered acceptable for most applications, i.e., less than 1% for the 50% in 50 year hazard, less than 2% for the 10% in 50 year hazard and less than 5% for the 2% in 50 year hazard.

The rotation of which plastic hinge forms, $\Upsilon_{p,}$ is shown by vertical solid lines for reference for link deformation plot in Figure 6. As shown, the links are predominantly elastic for the 50% in 50 year hazard and should not require repair. In the 10% in 50 year hazard, links in all stories have rotations larger than Υ_p and some may have damage that warrants link replacement. In the 2% in 50 year hazard level the links have larger inelastic demand and are even more likely to require replacement.

The beam rotation demands, θ, for the three hazard levels with θ_p, the approximate rotation at which a plastic hinge forms, are also shown in Figure 6. As shown, beam initial plastic hinge formation does not occur until the 2% in 50 year hazard level. This ensures that no repairs would be necessary following the design basis earthquake, which achieves the performance objectives and will help to minimize post event repair costs and downtime.

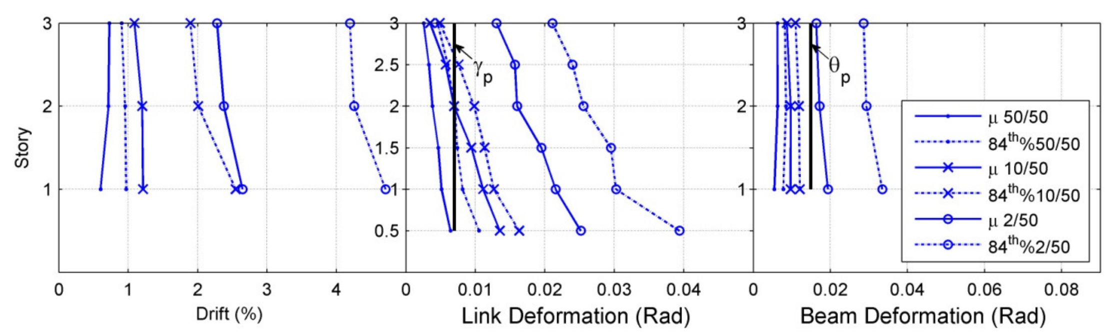


Figure 6. Median and 84th Percentile Story Drift Results for 3-, 6- and 9-Story LCF Buildings

7. LCF HYBRID SYSTEM TEST SETUP

Three LCFs have been designed to be tested for the purposes of evaluating LCF behaviour and further validation of numerical models. These three systems are as follows: LCF-B-ISL which has built-up sections for the links and has links only at inter-story locations; LCF-B-L which has built-up sections for the links and has links at both floor and inter-story locations; and LCF-W-L which has built-up sections for the links and has links at both floor and inter-story locations.

To date LCF research has focused on system level numerical model development, system level analysis and component tests (Dusicka and Lewis, 2010). The next step in understanding the behaviour of the system is to perform experimental testing of prototypical LCF systems. The proposed experimental test setup to validate the response at a system level is illustrated in Figure 7. The goals of the tests are to understand how the LCF system components interact together as a unit, to monitor the progression of damage in the replaceable links and ultimately validate the rapid return performance based design methodology. The experimental testing program will be conducted at the National Science Foundation NEES site at the University of California at Berkeley (NEES@Berkeley).

LCF systems will be investigated experimentally as full-scale 1-bay and 2-story structures. All steel is assumed to be 345 MPa nominal yield stress except for the links which are assumed to be 250 MPa. The typical bay width is 7.5 m, the typical story height is 3.0 m, and each linked column is spaced 1.0 m apart. Links are bolted to the columns to facilitate post-earthquake replacement. The LCF moment frame is expected to remain elastic under moderate seismic demands, while links are expected to yield and deform plastically. For large demands beam yielding is also expected.
The hybrid test scheme developed for LCF combines physical testing with model-based simulation to investigate the overall structure response. To perform the LCF hybrid simulation OpenSees will be used as finite element software to model and analyze the LCF. The Open-source Framework for Experimental Setup and Control, OpenFresco, will be used to connect the finite element analysis software with a control and data acquisition software.

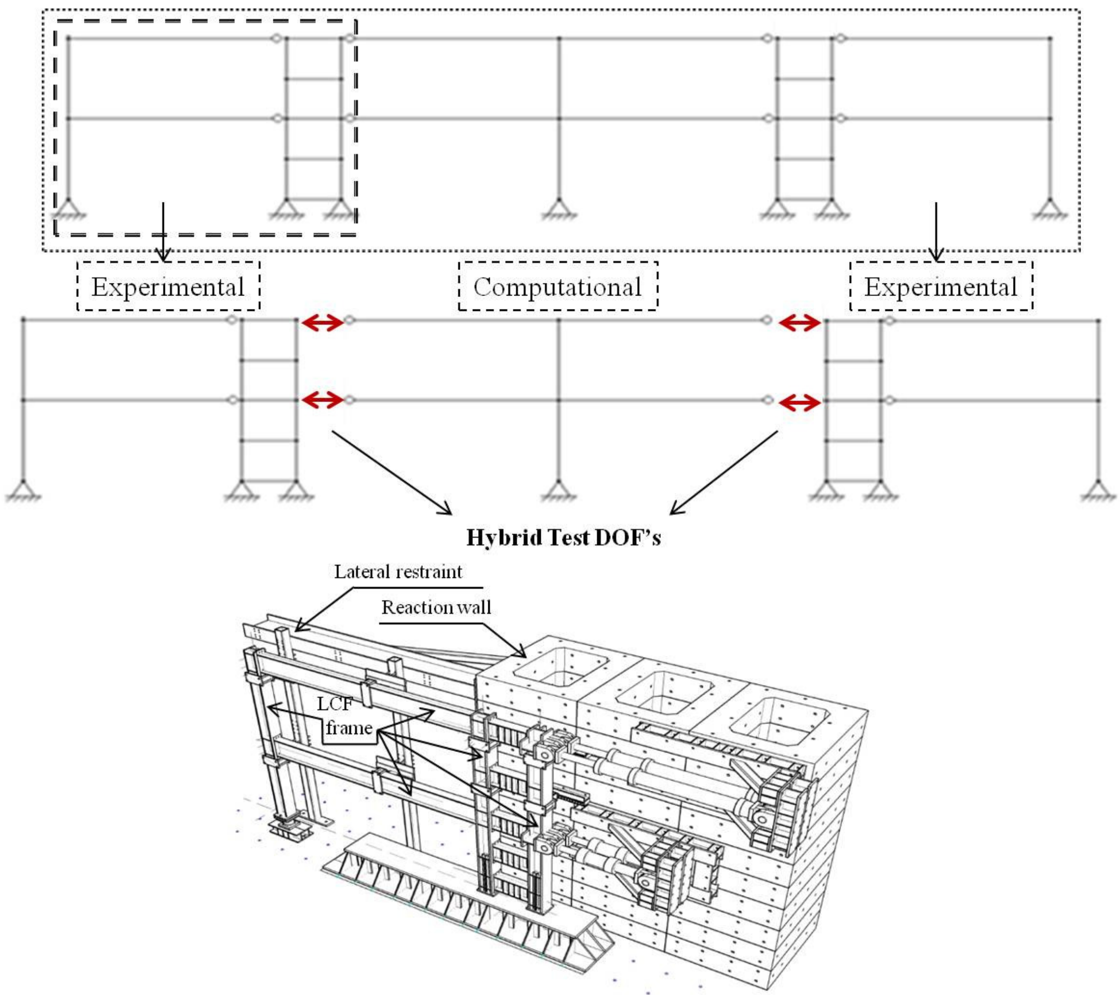


Figure 7. Proposed Experimental Setup for the LCF

8. CONCLUSIONS

A new lateral load resisting system denoted the linked column frame system (LCF) that can provide rapid return to occupancy following an earthquake event was discussed. The system incorporates replaceable links placed between closely spaced columns, combined with a secondary moment frame where the links are designed to develop plastic hinges well before the beams of the secondary moment frame.

The results of nonlinear response history analysis of a series of 3-, 6- and 9-story LCF show that all LCFs achieved the key design objectives, namely, that no repair is needed after a 50% in 50 year event as only minor link yielding was observed, that rapid return to occupancy by replacing the damaged links is achieved for a 10% in 50 hazard level as plastic hinging was successfully limited to the links and that collapse prevention for 2% in 50 year hazard level is achieved since the story drifts were generally less than 5%. Additionally, even though most LCF designs were drift controlled, all story drifts from response history analyses were less than 2% for the 10% in 50 year earthquakes, which approximated the design seismic demands. This performance for the design seismic demands is acceptable. System level experiments using NEES infrastructure and hybrid simulation are underway to verify these conclusions.

AKCNOWLEDGEMENT

This research was conducted at the University of Washington and Portland State University and was supported in part by the National Science Foundation under award number CMMI-0830414. However, any opinions, findings, conclusions, and recommendations presented in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

REFERENCES

Dusicka, P., Lewis, G. (2010). Investigation of Replaceable Sacrifficial Steel Links. *Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering*,1659, EERI.

Dusicka, P., Iwai, R. (2007). Development of Linked Column Frame System for Seismic Lateral Loads. *Proceedings of the SEI Structures Congress*, American Society of Civil Engineers, Vancouver, B.C.

FEMA P695 (2008), Quantification of Building Seismic Performance Factors, Technical Report P695, SAC JointVenture for the Federal Emergency Management Agency, Washington, D.C.

Gupta, A., Krawinkler, H. (1999). Seismic Demands for Performance Evaluation of Steel Moment Resisting Frame Structures, *Technical Report 132, John A. Blume Earthquake Engineering Center*, Stanford University, Stanford, CA.

Malakoutian, M. (2011). Seismic Response Evaluation of the Linked Column Frame System, Ph.D. partial fulfillment, Civil and Environmental Engineering Dept., University of Washington, Seattle, WA.

Mazzoni, S., McKenna, F., Scott, M. H., Fenves, G. L. (2009).Open System for Earthquake Engineering Simulation User Command-Language Manual- OpenSees Version 2.0, Pacific Earthquake Engineering Research Center,University of California, Berkeley, Berkeley, CA.

Somerville, P., Smith, N., Punyamurthula, S., Sun, J. (1997), Development of Ground Motion Time Histories for phase 2 of the FEMA/SAC Steel Project, Technical Report SAC/BD-97/04, SAC Background Document.