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DEVELOPMENT OF TESTING PROCOTOL FOR LINKS IN ECCENTRICALLY BRACED FRAMES

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SUMMARY

An analytical study was conducted to develop a new testing loading protocol for short links in eccentrically braced frames (EBFs). The current loading protocol for links in the AISC Seismic Provisions is a modified version of the moment frame protocol, without a rational basis. Three EBFs were designed according to current U.S. seismic provisions and non-linear time-history analyses were performed using an ensemble of Los Angeles ground motions scaled to match the design spectral acceleration of each frame. Cumulative and maximum rotation demands obtained from the analyses provided the basis for the new protocol. The analysis results indicate that the link loading sequence currently in the AISC Seismic Provisions is too conservative for short shear links. It has 1.5 times the cumulative rotation demand and a much higher percentage of large cycles than the analysis results indicate is necessary. The proposed protocol more accurately represents design earthquake demands.

INTRODUCTION

Steel eccentrically braced frames, EBFs, have been a popular alternative to moment frames and concentric braced frames since their introduction to practice in the early 1980's. The successful performance of

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EBFs under seismic loading depends on stable inelastic rotation of active links while other frame components remain essentially elastic.

Current U.S. design provisions permit short links ($eV_p/M_p < 1.6$, where e = link length, M_p = plastic moment capacity, and V_p = plastic shear capacity) to be designed for inelastic rotations of 0.08 rad [1]. This deformation capacity is mainly based on results from A36 short links tested at the University of California, Berkeley (UCB) in the 1980's [2, 3, 4, 5, 6].

Recently, a number of experimental short links tested at the University of Texas, Austin (UTA) did not reach the design inelastic rotation, 0.08 rad, prior to failure [7]. The most common failure mode of the UTA short links was horizontal web fractures which propagated from the termination of stiffener-to-web fillet welds. An analytical investigation of the failures found that close stiffener spacing explained the mode of failure, while the loading protocol was likely responsible for links failing to reach 0.08 rad inelastic rotation [8]. The protocol in the AISC Seismic Provisions was used in the UTA tests.

A number of different cyclic loading protocols were used in the UCB link studies in the 1980's. Malley [3] noted that link rotation capacities were dependent on the loading protocol used in testing, but standardized protocols for experimental testing were not developed until after the 1994 Northridge, California earthquake. Since Northridge, rational loading protocols have been established for steel moment frames [9] and woodframe structures [10]. The loading protocol for EBF link testing in the AISC Seismic Provisions [1] is a modification of the moment frame protocol and not based on any rational analysis.

Figure 1 compares the number of inelastic cycles that are necessary to reach 0.08 rad inelastic rotation under various protocols that have been used in short link testing (assuming elastic rotation ≈ 0.0075 rad). The AISC protocol requires more inelastic cycles and cumulative rotation than previous protocols, which may explain why the UTA links tested with the AISC protocol did not achieve the large rotations reported in earlier UCB tests. The objective of this study was to investigate the cumulative rotation demands on short links in EBFs under design earthquake loading and develop a cyclic loading protocol that reflects those demands.

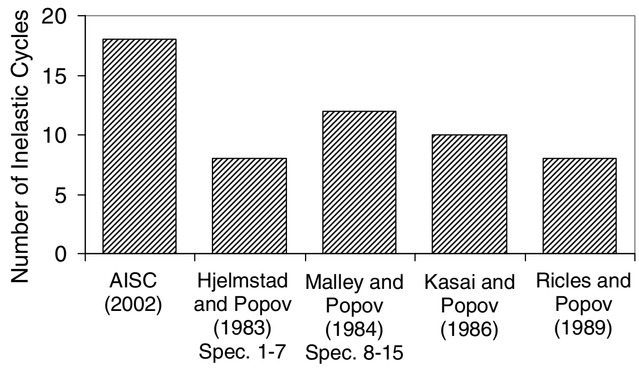


Figure 1 Number of inelastic cycles to reach 0.08 rad inelastic rotation with various link loading protocols

PROTOTYPE STRUCTURES

Two prototype eccentrically braced buildings, one 3 stories and one 10 stories, were designed according to the IBC [11] and AISC Seismic Provisions [1]. The buildings were similar in dimension and gravity loading to buildings used for moment frame analysis in the SAC project [12]. Figure 2 shows the plan views of the two buildings with the bay dimensions, column orientation, and EBF braced bays indicated.

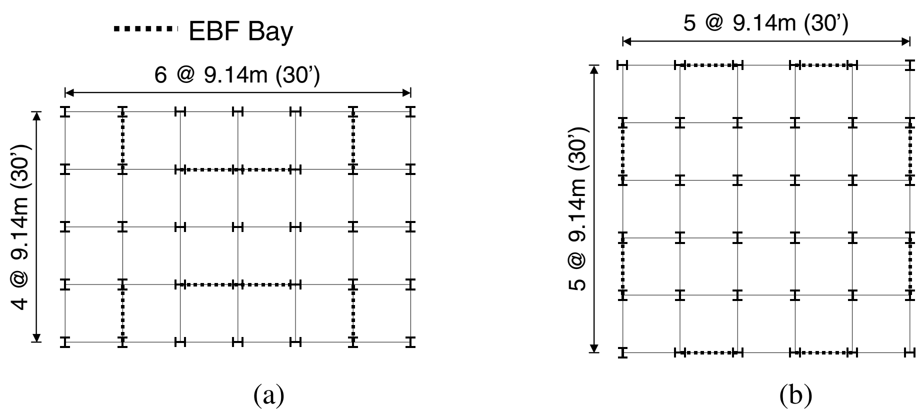
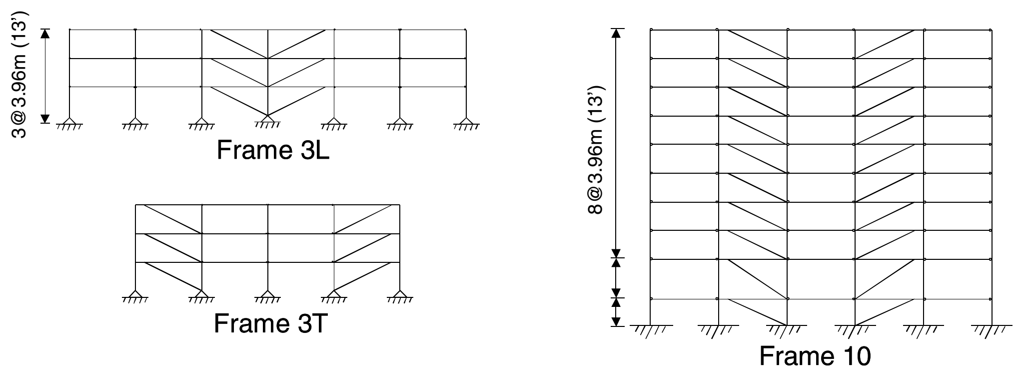


Figure 2 Plan views of prototype EBF buildings: (a) 3-story; (b) 10-story

The 3-story building had different EBFs for longitudinal (Frame 3L) and transverse loading (Frame 3T). The 10-story building had the same EBF in both directions (Frame 10). Figure 3 shows the elevations of these frames. Table 1 indicates member sections. The links in the EBFs of each frame were sized to be shear links and have similar demand/capacities ratios to encourage distributed link yielding as recommended by Popov [13].



(a) (b)

Figure 3 Elevations of prototype EBF frames: (a) 3-story; (b) 10-story

Table 1 Member Sections and Link Properties: (a) Frame 3L, (b) Frame 3T, (c) Frame 10

(a)

	Member Sections			
Story	Braces	Columns with link connection	Columns without link connection	Beams/links
1	W14×176	W14×132	W14×61	W18×86
2	HSS14×14×5/8	W14×132	W14×61	W14×82
3	HSS12×12×5/8	W14×132	W14×61	W10×68

(b)

	Member Sections			
Story	Braces	Columns with link connection	Columns without link connection	Beams/links
1	HSS16×16×5/8	W14×90	W14×61	W16×77
2	HSS14×14×5/8	W14×90	W14×61	W14×74
3	HSS12×12×1/2	W14×90	W14×61	W10×45

(c)

	Member Sections			
Story	Braces	Columns with link connection	Columns without link connection	Beams/links
1	HSS14×14×5/8	W14×311	W14×257	W14×74
2	HSS16×16×5/8	W14×311	W14×257	W18×106
3	HSS14×14×5/8	W14×193	W14×159	W16×77
4	HSS14×14×5/8	W14×193	W14×159	W14×82
5	HSS14×14×5/8	W14×132	W14×109	W14×82
6	HSS14×14×5/8	W14×132	W14×109	W14×74
7	HSS14×14×1/2	W14×90	W14×68	W14×68
8	HSS14×14×1/2	W14×90	W14×68	W10×68
9	HSS14×14×1/2	W14×61	W14×43	W12×45
10	HSS12×12×1/2	W14×61	W14×43	W10×45

MODELING

Models for the three frames (3L, 3T, and 10) were developed and analyzed with DRAIN-2DX [14]. Beams, braces, and columns were modeled with a beam-column element consisting of an elastic beam, two rigid-plastic hinges, and rigid end zones. Shear links were modeled using a technique similar that proposed by Ramadan [15] but with some modification [16].

A gravity load combination of 1.2D+0.5L [11] was applied to the structures in combination with the earthquake time history. The gravity loads for the half of the structure associated with each frame, but not acting directly on it, were applied to a P-delta column. Nonproportional damping, with no viscous damping in the links [17] was used with damping coefficients based on 2 percent damping in the first mode and at a period of 0.2 seconds for each frame.

TIME HISTORY ANALYSES

Earthquake Records

Twenty LMSR (large magnitude small distance) Los Angeles ground motions [18] were used for the dynamic analyses. Scale factors were calculated to make the spectral acceleration of each record, with 2 percent damping, equal to a design spectral acceleration of each frame. Figure 4 shows the 2 percent damping design spectra which was used, obtained by adjusting the 1997 UBC [19], Soil Type D spectra, which is for 5 percent damping, using the scaling procedure in FEMA 356 [20].

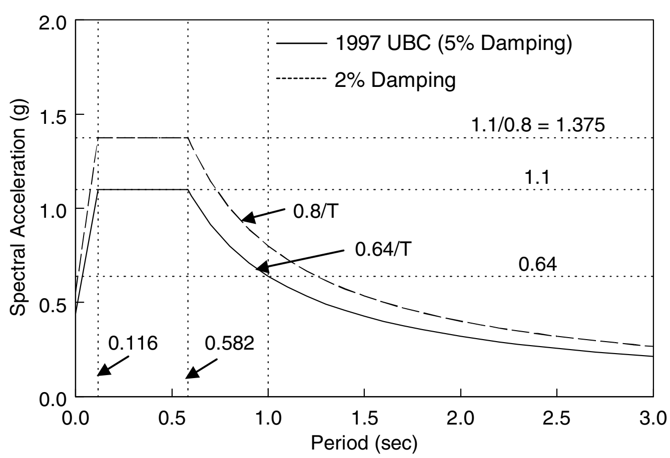
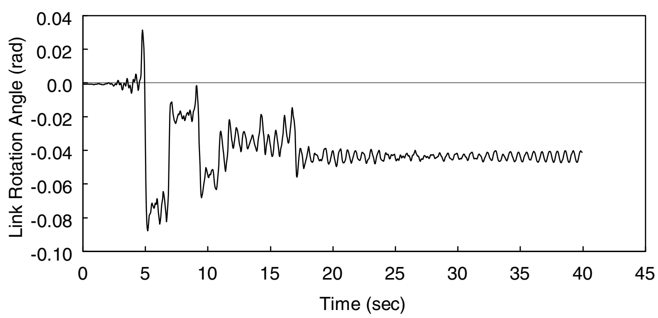


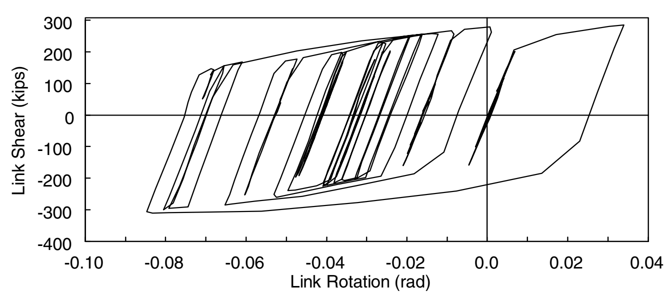
Figure 4 Design spectra for 2 percent damping

Data Reduction

Figure 5(a) shows a typical link rotation time history from a first story link of Frame 3T under one ground motion. Figure 5(b) shows the link shear versus rotation hysteresis for the same link. The link rotation time histories needed to be converted into series of cycles before they could be used for loading protocol development. The simplified rainflow cycle counting method, used by Krawinkler [9] in the SAC Moment Frame Protocol study, was utilized. When mean effects are not considered, the rainflow cycle counting process results in a number of symmetric cycles defined by their range (change in deformation from peak to peak). Figure 5(c) shows cycles calculated from the rotation time history in Figure 5(a). All of the link rotation time history data was reduced in this way so that for each link in each frame under each seismic event there was an associated sequence of symmetric cycles ordered with decreasing rotation range.



(a)



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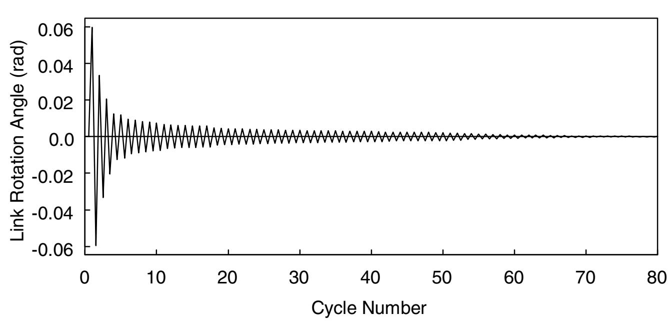


(c)

Figure 5 Representative link data: (a) rotation time history; (b) shear versus rotation hysteresis; (c) ordered cycles from rainflow counting procedure

RESPONSE PARAMETERS

The development of the short link protocol followed the same methodology used in developing the basic SAC moment frame loading history [9]. For the SAC moment frame loading protocol, the basic deformation parameter was the interstory drift angle, θ. For shear link loading protocols, however, the link rotation angle, γ, has typically been used as the deformation parameter.

In terms of γ, appropriate values of the following demand parameters needed to be determined from the time history analyses results:

- N = *Number of Cycles* (cycles with rotation range > 0.0075 rad). Rotations greater than half the yield rotation are considered damaging. The range of 0.0075 rad was selected because a link rotation of 0.00375 rad (half of that range) corresponds to an estimate of half the yield rotation. Cycles with range less than 0.0075 rad are not considered damaging.
- $\Delta\gamma_i$ = Rotation Range of Cycle *i*. Cycles are arranged in descending order so that cycle 1 has the largest range, cycle 2 as the second largest, and so forth.
- $\Sigma\Delta\gamma_i$ = *Sum of Cycle Ranges* (cycles with rotation range larger than 0.0075 rad). This is the measure of cumulative rotation demand.
- N_p = *Number of Inelastic Cycles*. Link yield rotation varies somewhat depending on section geometry and link length, but is generally close to 0.0075 rad for short shear links. Inelastic cycles can be roughly defined as those with $\Delta\gamma_i$ greater than 0.015 rad (2×0.0075).
- $\Delta\gamma_{max}$ = *Maximum Rotation Range*.
- γ_{max} = *Maximum Rotation*.

These demand parameters parallel those used by Krawinkler et al. [9]. Values for these parameters were calculated for each link in each frame under each earthquake using the cycle data obtained from the rainflow counts.

The links in the first and third stories of frame 3T had the highest overall values and were considered the “critical links”. For each demand parameter a lognormal distribution was used to describe the data from the critical links and appropriate percentile values were used as reasonable values to be represented in the protocol. The total number of cycles, N_t, was based on a 50th percentile value while 90th percentile values were used for other demand parameters.

Table 2 indicates the values for the demand parameters as determined from the analyses. Also shown are demand values from the AISC protocol [1]. The AISC protocol has fewer cycles, but greater cumulative rotation demands than indicated by the data. This indicates the AISC protocol has too many large magnitude cycles. This is also illustrated by the cumulative distribution functions shown in Figure 6.

Table 2 Comparing Protocol Demands with Target Values

Demand Parameter	Appropriate Value from Analyses	AISC Protocol	Proposed Protocol
N_t	36 cycles	24 cycles	36 cycles
N_p [a]	18 cycles	18 cycles	18 cycles
$\Sigma\Delta\gamma_i$	1.10 rad	1.69 rad	1.14 rad
$\Delta\gamma_{max}$	0.18 rad	0.18 rad	0.18 rad
γ_{max}	0.09 rad	0.09 rad	0.09 rad
$\Delta\gamma_i$	See Figure 6	See Figure 6	See Figure 6

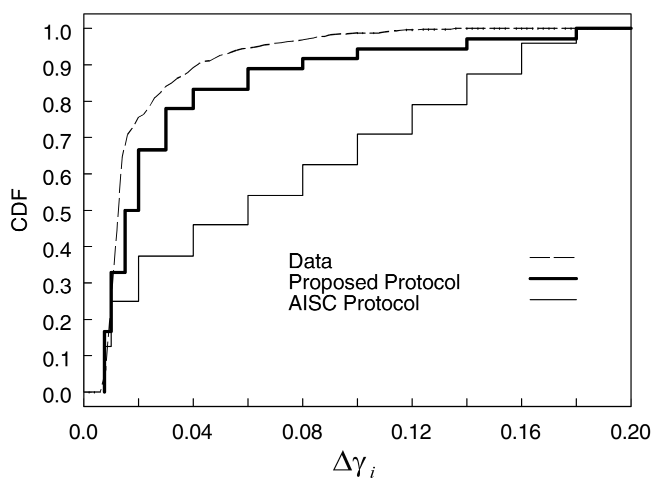
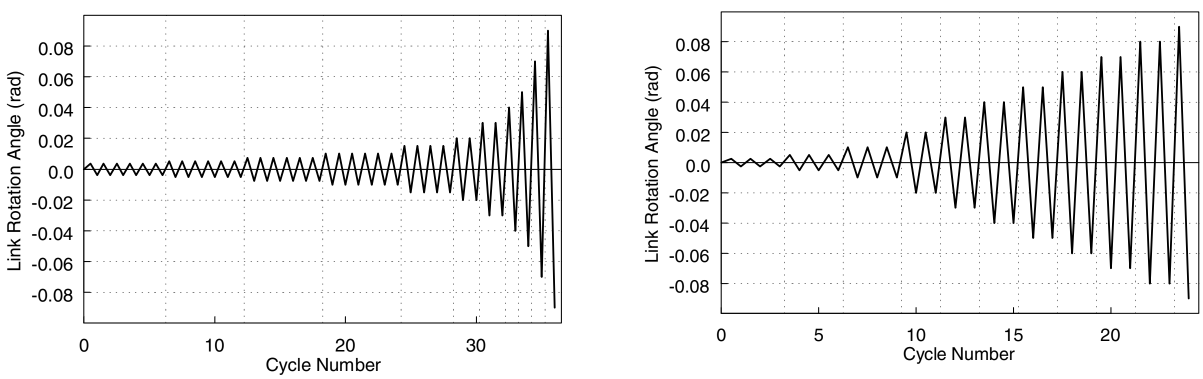


Figure 6 Cumulative distribution functions (CDFs) for data and loading protocols

PROPOSED SHORT LINK LOADING PROTOCOL

A proposed loading protocol for short links was developed based on the demand parameter values from the analyses shown in Table 2. The protocol consists of several rotation amplitude steps, each consisting of a number of symmetric cycles. The proposed protocol is illustrated in Figure 7(a). The AISC protocol [1] is shown in Figure 7(b) for comparison. Cycle magnitudes for the proposed protocol are indicated in Table 3.



(a) (b)

Figure 7 Experimental loading histories for short links: (a) proposed; (b) AISC

Table 3 Proposed Loading Protocol

Load Step	Peak Link Rotation Angle, γ	Number of Cycles
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	6
5	0.015	4
6	0.02	2
7	0.03	2
8	0.04	1
9	0.05	1
10	0.07	1
11[a]	0.09	1

[a]Continue with increments in γ of 0.02, and perform one cycle at each step until failure

The AISC protocol requires 48% more cumulative rotation than the proposed protocol to reach 0.08 rad inelastic rotation. In addition, and perhaps more significant, 72% of the total cumulative rotation in the AISC protocol comes from cycles with ranges greater than 0.1 rad. In comparison, only 37% of the total cumulative rotation comes from cycles with ranges greater than 0.1 rad in the proposed protocol (Figure 7). The damage model assumed in protocol development [9] is based on the principle that large excursions cause much more damage than small excursions. Based on the demand parameters and the assumed damage model, the proposed protocol is significantly less severe than the current AISC protocol.

CONCLUSIONS

There has been a shear link loading protocol specified in the AISC Seismic Provisions since 1997; however, it is a modified version of moment frame loading protocols without any study to justify it. Recent experiments have shown that short links designed according to the AISC Seismic Provisions [1] and tested with this protocol were unable to achieve the design inelastic rotation of 0.08 rad [7].

The objective of this study was to investigate the rotation demands on short links in eccentrically braced frames under design earthquake loading and develop a rational loading protocol for link testing. One 3-story building and one 10-story building, with a total of three unique eccentrically braced frames, were designed according to current U.S. codes and provisions. The designs had links in configurations where one end of each link was connected to a column. Models were developed for each of the frames, and nonlinear time- history analysis was performed using a suite of Los Angeles earthquakes scaled to match the 1997 UBC design spectra with 2% damping.

The model results indicated that the protocol in the AISC Seismic Provisions is overly conservative in representing design earthquake demands. A new loading protocol was developed following the same general procedure as was used in developing the SAC moment frame loading protocol [9]. The proposed protocol calls for significantly less cumulative rotation and fewer large inelastic cycles to reach the link maximum design rotation.

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