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EXPERIMENTAL VALIDATION OF THE STRUCTURAL FUSE CONCEPT

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ABSTRACT:

Seismic design relies on inelastic deformations through hysteretic behavior. However, this translates into damage on structural elements, permanent system deformations following an earthquake, and possibly high cost for repairs. An alternative design approach is to concentrate damage on disposable and easy to repair structural elements, while the main structure is designed to remain elastic or with minor inelastic deformations. The implementation of the structural fuse concept into actual buildings would benefit from a systematic and simple design procedure. Such a general procedure is proposed in this paper for designing new or retrofitted structures. The proposed structural fuse design procedure for MDOF structures relies on results of a parametric study, considering the behavior of nonlinear SDOF systems subjected to synthetic ground motions. This procedure is illustrated as an example of application using Buckling-restrained braces (BRBs) as metallic structural fuses. To verify and validate the developed design procedure, an experimental project was conducted on the shaking table at University at Buffalo, which consists of a three-story frame designed with BRBs working as metallic structural fuses. This experimental project also assesses the replaceability of BRBs designed as sacrificeable and easy-to-repair members.

KEYWORDS: Steel structures, seismic design, metallic dampers, inelasticity, experimental response

1. INTRODUCTION

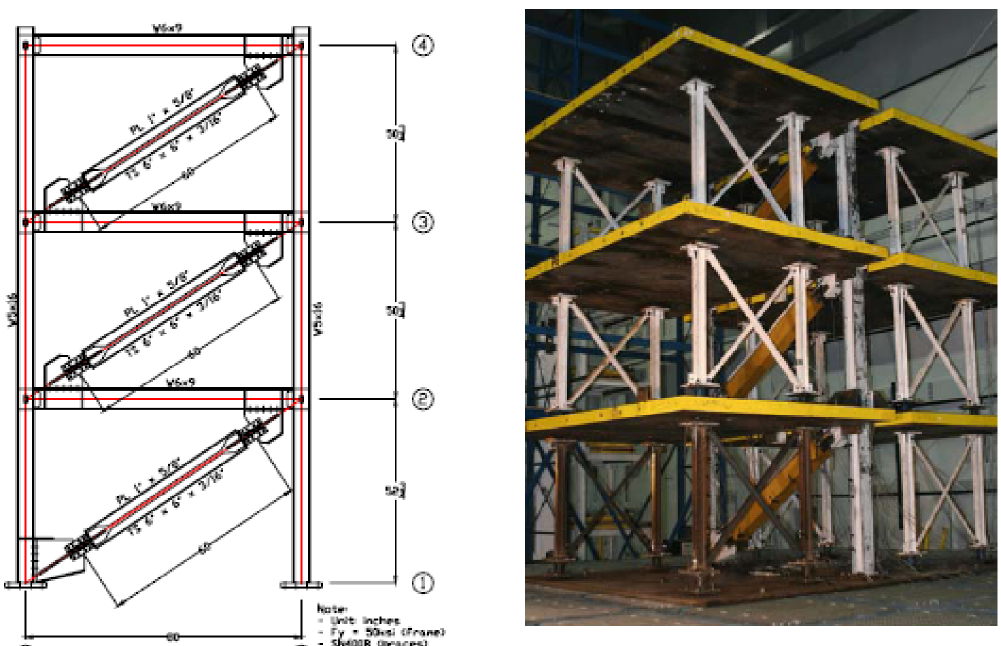
Passive energy dissipation (PED) devices have been implemented in recent years to enhance structural performance by reducing seismically induced structural damage (and, indirectly to some extent, non-structural damage). Soong and Spencer (2002) reported that, in the last 16 years, more than one hundred buildings in North America have been either retrofitted or built using PED devices. In the meantime, Japan has employed these structural protective systems in hundreds of buildings.

PED metallic dampers (a.k.a. hysteretic dampers) dissipate energy via inelastic deformations. Since their response is not sensitive to the frequency of loading, they are also called rate-independent dampers, or displacement-dependent dampers. The amount of damping they provide is somewhat proportional to the magnitude of their inelastic deformations. Although they also increase the stiffness of the primary structure to some degree, the possible increase in input energy due to the added stiffness is dissipated as part of the total hysteretic behavior of properly designed dampers, resulting in a net reduction on the response of the structural system in terms of lateral displacements, compared to response of the system without dampers. Accelerations and lateral forces are either increased or reduced depending on the ground motion and system features. Metallic dampers are defined here to be structural fuses when they are designed such that all damage is concentrated on the PED devices, allowing the primary structure to remain elastic.

In a Vargas and Bruneau (2006a) a procedure to design and retrofit structural fuse systems is presented, based on a parametric analysis conducted for SDOF systems. As a proof of concept to the developed design procedure, this paper describes an experimental testing on the shaking table at University at Buffalo of a three-story frame designed with buckling-restrained braces (BRBs) acting as metallic structural fuses. This experimental project also assesses the replaceability of BRBs designed as sacrificeable and easy- to-repair elements. Eccentric gusset-plate especially designed to prevent performance problems observed in previous experimental research (e.g., local instability of concentric gusset-plates as reported by Tsai et al. 2004, Mahin et al. 2004, and Uriz, 2005) are used for the connection of BRBs. Furthermore, a series of uniaxial static tests were conducted to experimentally determine the cyclic characteristics of the BRBs (for brevity purposes, only results corresponding to the experimental tests are presented in this paper).

2. FRAME AND BRBs DESCRIPTION

The three-story one-bay frame specimen tested is a model from one of the design examples presented in Vargas and Bruneau (2006b) as prototypes. Frame members were designed using steel with a yield stress of 345 MPa (50 ksi). The model is a two-dimensional structure designed with BRBs manufactured by Nippon Steel Corporation, as shown in Figure 1a. Note that the bare frame was designed to be used in repeatable tests with two sets of BRBs. A general view of the experiment setup can be seen in Figure 1b.



(a) (b)

Figure 1. Experiment Setup: (a) Frame with Nippon Steel BRBs, (b) General View of the Experiment

BRBs core consists of a rectangular plate (16 mm x 25 mm) made of SN400B steel (F_y = 235 MPa, and F_u = 400 MPa), which expands at the ends to form a cruciform section. A steel tube (HSS 6 x 6 x 3/16) filled with mortar surrounds the core to prevent buckling of the plate and ensure a similar behavior in tension and compression of the brace.

2.1 Scaling

Due to loading equipment constraints, specimen components and mass were scaled using a scale factor of 1/3 for geometric quantities and 1/18 for the mass (i.e., S_L = 1/3, and S_M = 1/18). Since the gravity loads for the model were carried by an independent gravity columns system (described later), the acceleration scale factor, S_A, can be established different to one, according to the following similitude relation: $S_A = S_L^2/S_M = (1/3)^2/(1/18) = 2$. Accordingly, time scale factor, S_T, was: $S_T = (S_L/S_A)^{1/2} = ((1/3) / 2)^{1/2} = 0.4082$, which implies that the ground motion ordinates and time step were multiplied by 2 and 0.4082, respectively. The components properties of the model are shown in Table 1.

Table 1. Summary of Components for the Model System

Story	Beams	Columns	BRB (mm)
3	W6 x 9	W5 x 16	PL 25 x 16
2	W6 x 9	W5 x 16	PL 25 x 16
1	W6 x 9	W5 x 16	PL 25 x 16

Figure 2 shows the pushover curves corresponding to the bare frame, BRBs, and the total base shear capacity of the system. Yield displacements of 7 mm and 34 mm for the BRBs and the bare frame, respectively, may be observed on this plot. In Figure 2, it can also be noted that BRBs do not yield simultaneously, since all braces have identical properties, which is a consequence of the physical constraints in the model.

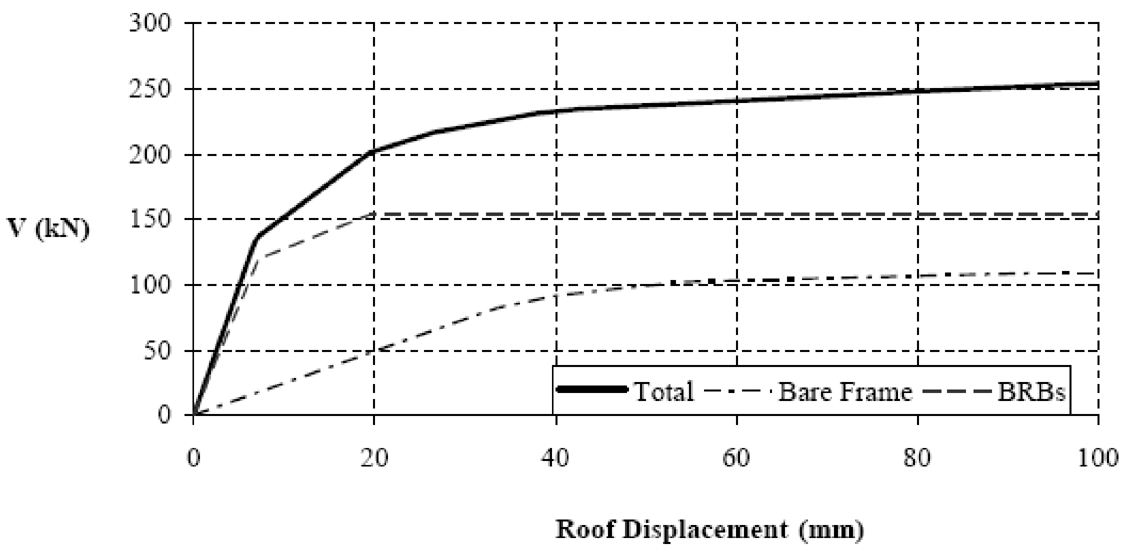


Figure 2. Pushover Curve for the Model

2.2 Gusset-plates Description

To achieve the objective of facilitating the replaceability of BRBs designed to work as metallic structural fuses, gusset-plates were also designed as removable elements bolted to frame members. Typical gusset-plates details for the model are shown in Figure 3. Note that the gusset-plates are eccentrically connected only to beams with a separation of 76 mm (3 in) from the columns. Although this is an eccentric connection, gusset-plates were designed such that center line of braces, beams, and columns coincide at the work point (i.e., intersection point between beams and columns center lines).

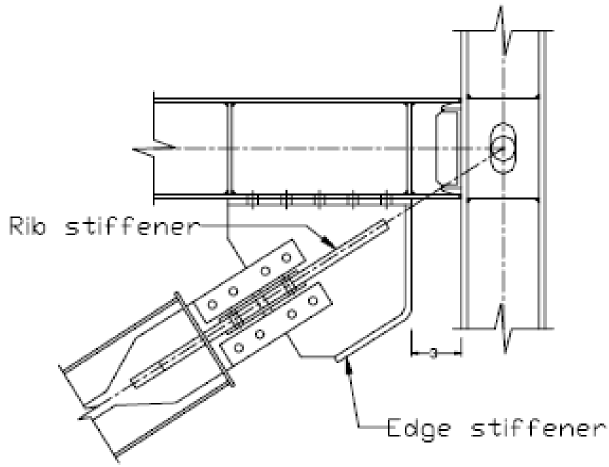


Figure 3. Typical Gusset plate details

These eccentric gusset-plates were also used to prevent performance problems that have been observed in previous experimental studies of buckling-restrained braced frames with concentric connections (Tsai et al. 2004, Mahin et al. 2004, and Uriz, 2005). Local buckling of gusset-plates may occur when the angle between beam and column closes due to lateral displacements. In this experimental project, the 76 mm gap corresponds to half of the beam depth, and was selected to avoid any contact between gusset-plates and columns during frame sway.

Rib stiffeners were also added to the gusset-plates to improve local buckling capacity, as shown in Figure 3, and the free edges of the gusset-plates were restrained by lateral stiffeners to prevent out-of-plane buckling of the plates (Tsai et al. 2004).

2.3 Gravity Column System

The gravity columns system is a set of frames designed to separate the lateral resisting system from the vertical load resisting system, and has been used in various projects for structures near collapse at University at Buffalo (Kusumastuti et al. 2005). These gravity frames consist of columns connected to rocking supports with the frame free to displace unrestrained in the longitudinal direction, and diagonally braced in the transverse direction. This set of gravity frames has been designed such that only vertical loads can be carried out by the columns.

Note that this gravity columns system is physically unable to support lateral loads. When the model is dynamically excited, lateral loads are transmitted to the testing frame by ϕ38 mm (1 ½ in) bolts. A machined hole was made at the mid-point of the column web at each beam level to match these pins. Doubler plates were used at both sides of the columns web to reinforce the panel zone, and the holes were designed vertically larger than the pins to avoid transmission of gravity loads through the pins during the tests.

2.4 Instrumentation

Instrumentation for this experimental project has been designed to measure global response of the frame, and local performance of beams, columns and braces. Global response of the structure in terms of floor accelerations and displacements was obtained from accelerometers and string-pots installed at the base of the frame and at every floor. Optical coordinate tracking probes (Krypton sensors) were also distributed on the first story to measure displacement response at specific points.

Seismic response of beams and columns was obtained from strain gages installed at critical points, to determine whether these members remain elastic during the test, recalling that one of the objectives of this experiment is to assess the effectiveness of the structural fuse concept to prevent damage in beams and columns. Axial deformations of the BRBs were measured with temposonic sensors installed in parallel with the braces and connected to the gusset-plates. A detailed description of the instrumentation plan is presented in Vargas and Bruneau (2006b).

2.5 Test Protocol

One of the spectrum compatible synthetic ground motions generated for the parametric study in Vargas and Bruneau (2006a) was used in this experiment as the input ground motion. For similitude purposes, this ground motion was scaled according to the scale factors presented earlier. In the test protocol for the experiment, the amplitude of the ground motion is increased by 0.25 g in each test until the capacity of the shaking table is reached (i.e., which is about 1.0 g for this experiment). Note that a *PGA* of 1.0 g is the upper level test value for the experiment. White noise tests (with *PGA* of 0.10 g) were also performed before and after every earthquake simulation test to identify the dynamic properties of the structure. Two sets of braces were tested following this protocol to examine the replaceability of BRBs as structural fuses.

3. EXPERIMENTAL RESULTS

3.1 Global Response

White noise tests performed after every earthquake simulation test to identify the dynamic properties of the testing frame indicated that the natural frequency of the BF alone (i.e., f_n = 1.52 Hz, T_f = 0.66 s) was less than for the cases in which additional stiffness is provided by the inclusion of the BRBs (i.e., f_n = 3.74 Hz, T_f = 0.27 s). Knowing that the stiffness ratio between BF and BRB frame is inversely proportional to the period ratio to the square (i.e., $K_f / K_1 = (T_1 / T_f)^2$), it results that the BRB frame was approximately six times stiffer than the BF.

Furthermore, damping ratio was determined using the logarithmic decrement method from the free vibration portion of the motions at the end of every earthquake level simulation. Average damping ratios of 2% and 5% were obtained for the BF and the BRB frame, respectively. The increase in the damping ratio as a function of increases in the magnitude of frame deformations is consistent with what has been observed by others (e.g., Vian and Bruneau, 2001). Note that the analyses were performed using a damping ratio of 2%, which coincides with the measured values at low amplitude tests, but it is significantly different than the values obtained at higher amplitude tests. This may explain some of the discrepancies observed between experimental and analytical results, as discussed below.

Seismic demand in terms of frame ductility, μ_f, and global ductility, μ, is presented in Table 2. Note that in every story of the BRB frame, the frame ductility is less than one (i.e., $\mu_f < 1$), which is one of the requirements to satisfy the structural fuse concept (recalling that beams and columns remain elastic when the frame ductility is less than one).

Table 2. Ductility Demand

Test \ PGA (g) (1)	0.25 (2)	0.50 (3)	0.75 (4)	1.00 (5)	0.25 (6)	0.50 (7)	0.75 (8)	1.00 (9)
	Frame Ductility (μ_f)				Global Ductility (μ)			
Bare Frame	0.799	1.179	1.685	2.249	0.799*	1.179*	1.685*	2.249*
BRB Frame 1	0.158	0.296	0.421	0.559	0.766	1.437	2.044	2.713
BRB Frame 2	0.149	0.279	0.421	0.631	0.726	1.354	2.044	3.067

* Frame and global ductility have the same values for the Bare Frame

Maximum seismic demand in terms of base shear and roof displacement for every earthquake level is presented in Figure 4, along with the theoretical pushover curves for the BRB frame and the BF. This figure shows a good correlation between the experimental seismic demand and the analytical pushover curve obtained for the BRB frame.

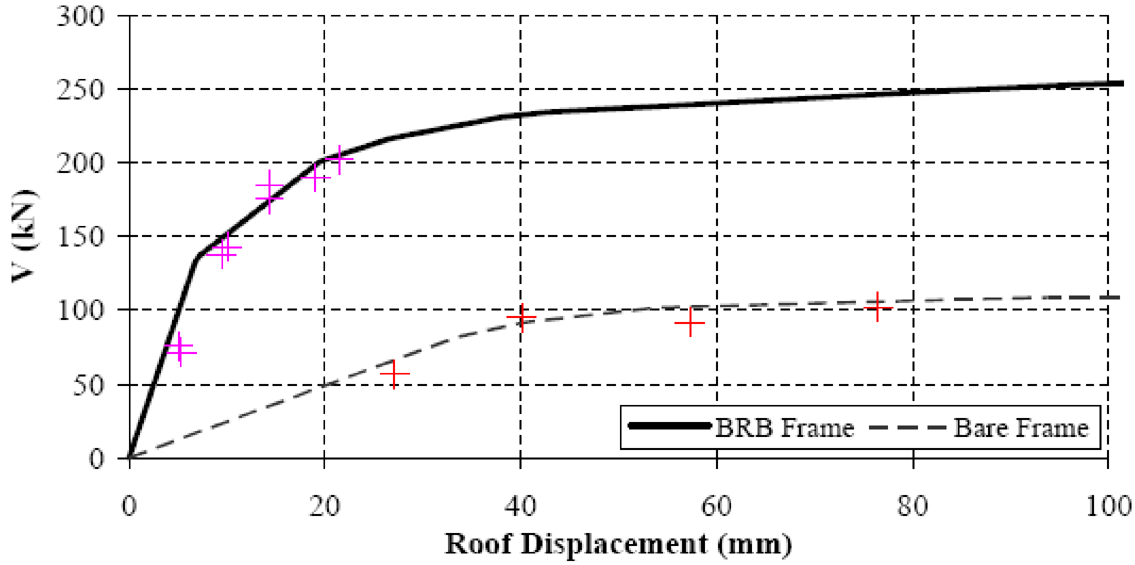


Figure 4. Seismic Demand for Bare Frame and BRB Frame

3.2 Local Response

3.2.1 Beams and columns response

Bending moments were calculated from strain gage results at critical locations of beams and columns to assess whether these elements remained elastic throughout the earthquake simulation. From these moments, and using equilibrium equations obtained from a free body diagram of the columns, it was possible to calculate the shear force at every one of those locations. Then, column shear forces were calculated at every story and results are plotted versus inter-story drifts in Figure 5a for the strongest earthquake level. The elastic behavior exhibited by the frame confirms that the objective of frame protection intended by the structural fuse concept was met. Since the results were experimentally obtained, note that the story shear response is not exactly a straight line (as it was predicted by the analytical model), and some scattering may be observed. However, response points appear to be sufficiently close to the straight line corresponding to the elastic behavior of the frame.

3.2.2 Buckling-restrained braces response

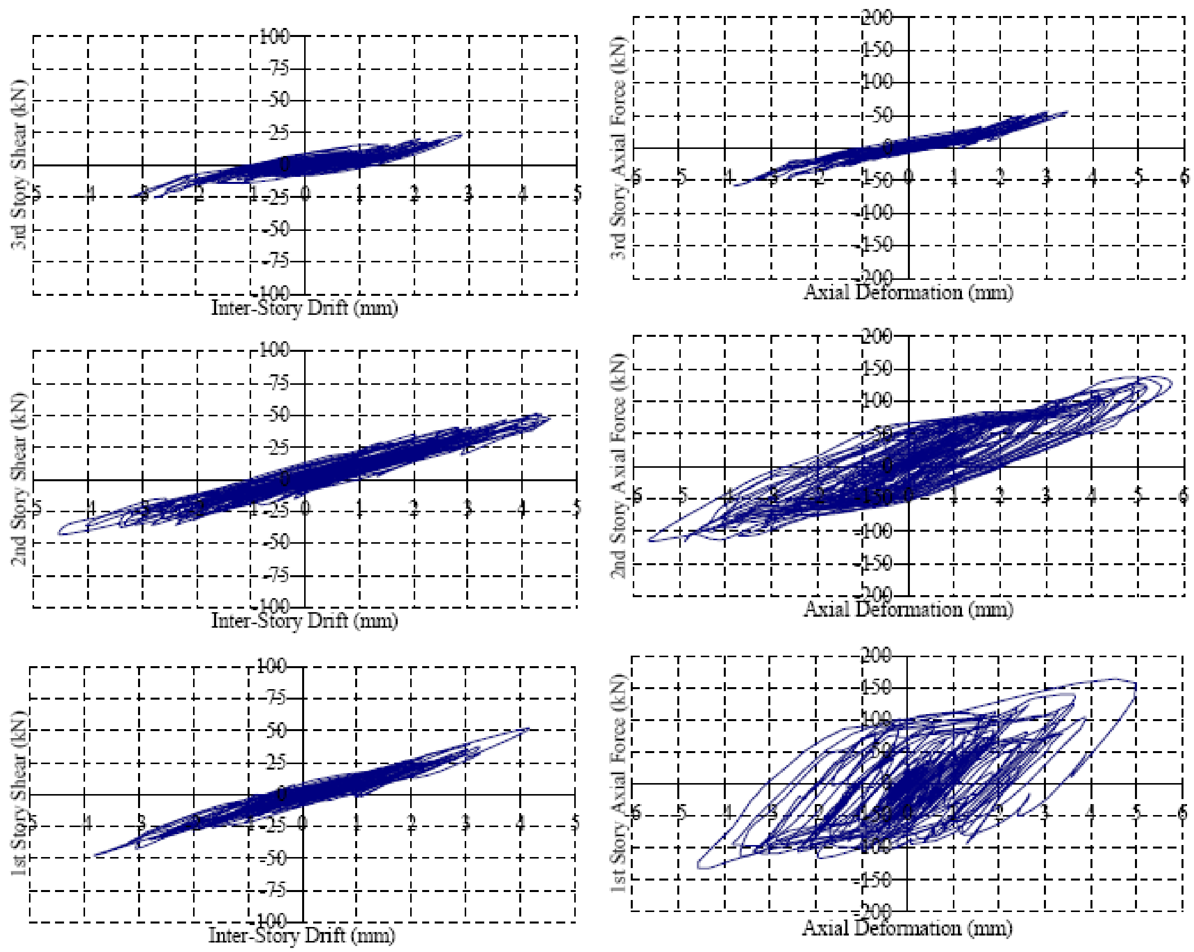
BRBs axial forces were indirectly obtained from the previously calculated internal forces for the beams, as described below. Beams moments and axial forces were determined from strain gages installed at the ends of the beams. From these moments, and using equilibrium equations obtained from free body diagrams of the beams, shear forces at the ends of the beams were calculated. Then, BRBs axial forces were calculated from equilibrium equations obtained from the free body diagrams including beams and braces.

Table 3 presents a summary of maximum axial deformation of BRBs, at every earthquake level, along with the corresponding ductility. An average ductility of 4.6 can be observed for first and second story BRBs at the strongest level of earthquake. BRBs axial forces and deformations were combined to plot the hysteresis loops presented in Figure 5b. Note that BRBs at first and second stories exhibited inelastic behavior with a ductility of 4.6, while third story brace remained basically elastic. Note also the apparent increase in the stiffness of the

BRB at story one versus the third story brace, which may be attributed to the type of connection used at the base of the frame. Base plates clamped to the shake table were use to connect the frame, and this may have increased the stiffness of the first story with respect to the other two stories.

Table 3. BRBs Axial Deformation

Story \ PGA (g) (1)	0.25 (2)	0.50 (3)	0.75 (4)	1.00 (5)	0.25 (6)	0.50 (7)	0.75 (8)	1.00 (9)
	Axial Deformation (mm)				Ductility (μ)			
	Test 1							
3	0.49	0.96	1.06	1.19	0.42	0.82	0.91	1.02
2	0.79	2.62	4.00	5.64	0.68	2.25	3.43	4.84
1	0.84	1.53	3.46	4.98	0.72	1.31	2.97	4.28
	Test 2							
3	0.51	0.86	1.12	1.27	0.43	0.74	0.96	1.09
2	0.77	1.87	3.50	5.45	0.66	1.60	3.00	4.67
1	0.87	1.69	3.20	5.40	0.75	1.45	2.74	4.64



(a) (b)

Figure 5. Hysteretic Response: (a) Columns Story Shear; (b) BRBs Hysteresis Loops

4. CONCLUSIONS

Experimental results presented in this paper indicate that the objectives of the structural fuse concept were successfully achieved (i.e., beams and columns performed elastically, while BRBs worked as metallic fuses and dissipated the seismically induced energy). In general, analytical models reasonably predicted maximum response values for the BRB frame, although some discrepancies were observed in the trend itself. These differences were attributed in part to the fact that the analyses were performed using a damping ratio of 2%, which was found to be lower than the actual values obtained at higher amplitude tests.

Replaceability of BRBs was also found to be feasible by examining this aspect in a test-assessment-replacement-test sequence using four sets of braces connected to the frame by removable eccentric gusset-plate, which were also found to be effective to prevent performance problems observed in other experimental studies with BRBs (e.g., local and out-of-plane buckling of concentrically connected gusset-plates). Incidentally, the proposed eccentric gusset-plates detail was found to be effective to prevent performance problems observed in other experimental studies, such as local buckling and out-of-plane buckling of the plates at the connection point. However, since the BRBs were not tested to failure in place in the frame, final conclusion regarding the performance of that proposed gusset detail should be the subject of further research.

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