Research Article

Experimental Study on the Seismic Behavior of a Steel-Concrete Hybrid Structure with Buckling Restrained Braces

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The damage to a concrete wall caused by a strong earthquake is generally concentrated at the bottom of the concrete wall, which seriously threatens the safety of the steel-concrete hybrid structure and is very difficult to repair after an earthquake. In this paper, a steel-concrete hybrid structure with buckling restrained braces at a scale of 1/10 is constructed and tested on a shaking table. First, the mechanical properties of the BRBs are obtained through a static reciprocating loading test. Then, the dynamic properties and seismic response of the steel-concrete hybrid structure with BRBs are obtained through shaking table tests. The results show that (1) the energy dissipation capacity of the BRBs is very good, and none of the BRBs buckle during the shaking table tests; (2) the steel beams and columns are basically in an elastic state; (3) all the cracks on the concrete wall are microcracks, which are widely distributed in floors 1–8 of the concrete walls; (4) the maximum interstory drift angle reaches 1/40, which indicates that the ductility of the steel-concrete hybrid structure is very good. In conclusion, BRBs can significantly improve the seismic performance of the steel-concrete hybrid structures.

1. Introduction

The conventional steel-concrete hybrid structure is generally composed of the steel frame and the concrete tube, which is considered to be one of the most efficient structural systems by using the concrete core to resist the lateral load and the steel frame to resist the vertical load [1–3]. Three steel-concrete hybrid structure buildings, i.e., the West Hotel, the Cordova Building, and the Hill building, were seriously damaged during the 1964 Alaskan earthquake. Therefore, the steel-concrete hybrid structure was considered unsuitable to be built in seismic regions in the United States [4–6].

Unlike the US, the steel-concrete hybrid structures are applied for the high-rise structures widely in seismic zones in China [7]. Therefore, many efforts have been made to investigate the seismic performance of the steel-concrete hybrid structures in China. Li et al. [8] conducted shaking table tests on a 1/20 scaled 25-story steel-concrete hybrid structure, and the results showed that both the bottom of

the concrete tube and connections between the steel beam and the concrete tube were severely damaged. Besides, Lu [9] performed quasi-dynamic tests on a 1/10 scaled 15-storey steel-concrete hybrid structure, and many horizontal cracks were found at the bottom of the concrete core. Zhou et al. [10] carried out a shaking table test of a scaled 20-story high-rise hybrid structure building and found that the connection between the frame and the tube was damaged at tension. Li et al. [11] proposed to reduce the structural damage of the steel-concrete hybrid structure by MR dampers. Li and Zhou [12] established a macroelement to predict the global response of the steel-concrete hybrid structure under earthquake. Furthermore, Ding et al. [13] investigated the failure phenomenon of steel-concrete hybrid structures, and the results indicated that the steel frames were still in an elastic state when the concrete shear walls were severely damaged.

Both the Alaskan earthquake damage and the experimental results indicate that the seismic behavior of the steel-concrete hybrid structure is not satisfactory because the

cracks of the concrete tube occur early and concentrate at the bottom of the concrete tube under the action of strong earthquake. Due to the undesired failure, the bottom of the concrete tube may develop into a plastic hinge and result in the large residual deformation of the structure, as shown in Figure 1(a). Therefore, it is significant to find the effective measures to reduce the damage of the steel-concrete hybrid structure when subjected to severe earthquake.

Buckling restrained braces (BRBs), which can provide lateral resistance as well as energy-dissipation capacity to the frameworks, have the functions of both the conventional steel brace and the metal energy dissipation damper [14–19]. Hector et al. [20] performed a shaking table test on a reinforced concrete frame equipped with BRBs and found that the BRBs increased the damping of the structure and reduced the damage of the concrete frame. Besides, Hu et al. [21] studied the effect of BRBs on the seismic performance of steel frame by sharing the shear force and dissipating a large amount of seismic energy and found that BRBs can reduce the deformation of steel frame under earthquake. The BRBs can be replaced easily after the earthquake, making it easy to repair structures after the earthquake. Li [22] studied the seismic performance of the steel-concrete hybrid structure with BRBs by numerical simulation method and proposed that the failure on the concrete wall is a global failure mode rather than a local failure mode, as shown in Figure 1(b). However, this conclusion has not been proved by test results.

In order to reveal the seismic performance of the steel-concrete hybrid structure with BRBs, a 10-story steel-concrete hybrid structure with BRBs at a scale of 1/10 is tested on a shaking table. The dynamic properties, seismic response, and the failure process are investigated in this paper. The experimental results testify the conclusion in reference [23] that the deformation of the steel-concrete hybrid structure with BRBs belongs to the bent-shear deformation, and the cracks are widely distributed on the concrete wall rather than being concentrated at the bottom of the concrete tube. The seismic performance of the steel-concrete hybrid structure is significantly improved after the installation of BRBs.

2. Design of the Specimen

2.1. Geometry of the Specimen. A 10-story office building, which is designed by the Xi'an Nonferrous Metallurgy Design and Research Institute according to the Chinese codes [1, 24, 25], is selected as the original structure of the specimen. The similarity coefficients of the specimen by means of dimension analysis [26] are listed in Table 1. For the sake of simplicity, the concrete tube is simplified into two shear walls, and the braced frame and steel frame are simplified into two one-span ten-story plane braced frames and two one-span ten-story plane steel frames, respectively, which are connected by horizontal coupling beams at every floor, as shown in Figure 2. The plane size of the specimen is 1000 mm * 2525 mm, and the total height of the specimen is 3500 mm.

The braced frame is composed of column (CZ-1), beam (GL-1), and BRBs. The steel frame is composed of column (KZ-1) and beam (GL02). XL01 is used to transmit the shear force between the braced frame and the steel frame, and XL02 is used to transmit the shear force between the steel frame and the shear wall.

2.2. Materials of the Specimen. The steel frame and BRBs are made of structural steel Q235B. The tested properties of Q235B are listed in Table 2. Microconcrete is used for constructing the shear wall. The cubic crushing strength of the microconcrete was tested to be 10.47 MPa. According to the recommended method in Section C.2 of the Chinese code GB50010-2010 [24], the σ - ϵ curves of the microconcrete are obtained, as illustrated in Figure 3, and the main mechanical parameters are listed in Table 3.

2.3. Details of the Specimen

2.3.1. Beam and Column. The sections of the steel components are listed in Table 4. The details of the coupling beams are shown in Figures 4 and 5.

2.3.2. BRBs and Shear Wall. The BRBs are connected to the beam and column with gusset plates by welding, as shown in Figure 6. The details of the BRB are illustrated in Figure 7.

The width of the shear wall is 450 mm. The thickness of the shear wall is 100 mm at the two ends and 50 mm in the middle, as shown in Figure 8. Two H-shaped steel components are embedded into the shear wall at the two ends, which may significantly improve the bearing capacity and ductility of the concrete wall. Shear studs are used to pledge a strong connection between the concrete and steel component.

2.3.3. Beam-Wall and Beam-Column Joint. As shown in Figure 8, only one bolt (M18) is used to connect the steel beam (XL02) and the embedded steel column (HZ-1). So the connection between steel and concrete wall is regarded as pinned connection, which may rotate freely. The internal force on the beam XL02 is directly transferred to the embedded column HZ-1 by connecting plate (-118X60X10), and the concrete around the joint does not bear any force.

As shown in Figure 9, two high-strength bolts with a diameter of 16 mm (M16 bolt) are used for the connection between GL01 and CZ-1, for the connection between GL03 and CZ-1, for the connection between GL02 and KZ-1, and for the connection between GL04 and KZ-1, which are all regarded as moment-transfer connection.

3. Construction of the Specimen

The construction of the specimen follows four steps: (1) the braced frame, the steel frame, and the steel skeleton of the shear wall are constructed and welded to two 20 mm rectangular steel plates, as shown in Figure 10(a); (2) the

FIGURE 1: Deformation and damage of the steel-concrete hybrid structure. (a) Conventional hybrid structure. (b) Hybrid structure with BRBs.

TABLE 1: Similarity coefficients of the specimen.

Parameter types	Length	Elastic modulus	Acceleration	Stress	Density	Mass	Period	Velocity
Equations	S_L	S_E	S_a	$S_\sigma = S_E$	$S_\rho = S_E/(S_L S_a)$	$S_m = S_\rho S_L^3$	$S_t = S_L(S_\rho S_E^{-1})^{1/2}$	$S_v = (S_E/S_\rho)^{1/2}$
Values	0.1	0.60	1.58	0.60	3.80	0.004	0.28	0.40

concrete wall is completed when the fluid microconcrete inside the wood formwork is hardened, as shown in Figure 10(b); (3) the vertical loads are applied on the specimen by 30 additional weight boxes, as shown in Figure 10(c); (4) 40 BRBs in total are welded to the braced frame, as shown in Figure 10(d).

4. Testing Program

4.1. Measurement Scheme

4.1.1. Strain Measurements. There are 30 strain meters distributed on the specimen to record the strains of the specimen at the critical positions, as shown in Figure 11(a). The strain meters denoted as S1–S9 are used to measure the strains of the braced frame. The strain meters denoted as S10–S18 are used to measure the strains of the steel frame. The rosette strain gauges denoted as S19–S21, S22–S24, S25–S27, S28–S30 are used to measure the strains of the concrete wall.

4.1.2. Displacement Measurements. Dynamic displacement meter D1 is fixed at the top surface of the base plate to record the time-history displacement of the shaking table. Dynamic displacement meters D2–D12 are fixed at the corresponding position of GL04 to record the time-history displacement from the first to the tenth floor, as shown in Figure 11(b).

4.1.3. Acceleration Measurements. Acceleration meters (A00–A20) are used to record the time-history acceleration

of the specimen, as shown in Figure 11(c). A00 is fixed on the base plate to record the time-history acceleration of the shaking table. A1–D20 are used to record the time-history acceleration from the first to the tenth floor.

4.2. Loading Schedule

4.2.1. Vertical Loads. The two steel base plates at the bottom of the specimen are fixed firmly on the top of the shake table by 48 anchor bolts. 30 steel boxes, which are filled with solid steel blocks, are fixed on the specimen as additional masses, as shown in Figure 12. The values of the additional masses are provided in Table 5. Noticeably, the masses on the top floor are larger than the masses on the other floors, which is intentionally due to the weight of the heavy facilities that are generally attached to the roof of the building.

4.2.2. Earthquake Action. The specimen is tested on the shaking table in the key lab of the Structure Engineering and Earthquake Resistance, Xi'an University of Architecture and Technology, China. The dimension of the shake table is $4.1\text{ m} \times 4.1\text{ m}$, and the allowed maximum mass of the specimen is 20 t . The peak acceleration is 1.5 g . The maximum displacement is 150 mm in the X direction, 250 mm in the Y direction, and 100 mm in the vertical direction.

Three types of input earthquake waves (i.e., El Centro, Shanghai, and Tianjin) are selected to test the seismic response of the specimen. The comparison between the design spectrum and the ground motions spectrum used in the tests is obtained while the damping ratio is 3% and the peak

FIGURE 2: Geometry of the specimen. (a) Layout of the columns, braces, and shear walls. (b) Layout of the beams and shear walls. (c) Elevation of the specimen.

TABLE 2: Properties of Q235B.

Steel material	Elastic modulus E (MPa)	Yield strength f_y (MPa)	Yield strain ϵ_y	Ultimate strength f_u (MPa)	Poisson's ratio ν
Q235B	2.01×10^5	251.4	0.0015	441	0.3

ground acceleration is 28.4 cm/s^2 , as shown in Figure 13. The GPA in Table 6 is obtained by scaling the suggested amplitude of the peak acceleration in the Chinese code GB50011-2010 [25] by 1.580 (the similarity coefficient in Table 1).

White noise scanning is used to get the basic frequency and the damping ratio of the specimen, which is performed before the test and then repeated at every level of earthquake intensity. All the shaking table tests are conducted in the X direction (parallel to the orientation of the BRBs).

5. Test Results and Discussion

5.1. BRB Performance. Three BRBs, which are randomly selected from the BRBs for the specimen, are loaded by the push-pull fatigue tester (MTS 880), as shown in Figure 14. One of the hysteresis curves of the BRBs is presented in Figure 15. The average values of the main mechanical parameters are listed in Table 7.

None of the BRBs buckled during the shaking table tests, and the gusset plates between the BRBs and the frame

FIGURE 3: Stress-strain relation obtained from the Chinese concrete code.

TABLE 3: Properties of the microconcrete.

Material	Compression		Tension	
	Stress σ_c^0 (N/mm ²)	Strain ε_c^0	Stress σ_t^0 (N/mm ²)	Strain ε_t^0
Microconcrete	10.47	0.00126	0.0015	0.000065

TABLE 4: Sections of the steel components.

Component type	Component symbols	Cross-section	Net length (mm)
Columns	CZ-1	□50 * 50 * 5.0 * 5.0	3500
	KZ-1	□80 * 80 * 5.0 * 5.0	3500
	HZ-1	H 50 * 50 * 5.0 * 5.0	3500
Main beams	GL01	□50 * 70 * 5.5 * 5.5	650
	GL02	□50 * 70 * 5.5 * 5.5	450
	GL03	□50 * 70 * 5.5 * 5.5	850
	GL04	□50 * 70 * 5.5 * 5.5	820
	GL05	□30 * 60 * 4.0 * 4.0	850
Secondary beams	GCL01	H 60 * 30 * 5.0 * 5.0	850
	GCL02	H 60 * 30 * 5.0 * 5.0	450
	GCL03	H 60 * 30 * 5.0 * 5.0	190
Coupling beams	XL01	See Figure 4.	265
	XL02	See Figure 5.	240

remained in good shape. It is very pity that the yield and energy dissipation of the BRBs cannot be measured during the shaking table tests.

5.2. Dynamic Property Evolution

5.2.1. Basic Frequency and Period. Figure 16 illustrates the changes in the basic frequencies and periods of the specimens, respectively, which are obtained from acceleration records of specimens under the action of white noise input by employing MATLAB software.

The basic frequencies of the specimen gradually decrease with increasing seismic intensity (the sequence number is

FIGURE 4: Details of XL01.

FIGURE 5: Details of XL02.

related to seismic intensity), which indicates that the failure of the specimen is increasingly severe as the earthquake intensity increases. As shown in Figure 16(a), the decline process of the basic frequency experiences three stages: (1) the value of the basic frequency is 5.08 from 17 to 33; (2) the value of the basic frequency is approximately 4.84 from 37 to 53; and (3) the value of the basic frequency is 4.69 at 57. As the reciprocal of the basic frequencies, the basic periods of the specimen get larger as the seismic intensity increases, as shown in Figure 16(b).

5.2.2. Damping Ratio. Figure 17 shows the changes in the damping ratios of the specimen as the earthquake intensity increases, which are also obtained from the acceleration records of the specimen under the action of white noise input by employing MATLAB software.

The increase process of the damping ratio also experiences three stages: (1) the value of the damping ratio is approximately 2.79 from 1 to 33; (2) the value of the damping ratio is approximately 3.37 from 37 to 45; and (3) the damping ratio rapidly increases from 3.94 to 5.67 when the sequence changes from 49 to 57. Therefore, the recommended value of the damping ratio of the steel-concrete hybrid structure with BRBs is 3% under the action of frequent earthquake and is 4% under the action of rare earthquakes in engineering design.

5.2.3. Evolution of the Basic Mode. According to the changes in the basic frequencies in Section 5.2.1 and the changes of the damping ratios in Section 5.2.2, the Fourier amplitudes of the specimen when the sequence number is 29, 41, and 57 are

FIGURE 6: Single-layer brace frame.

FIGURE 7: Details of the BRB.

selected and compared. Both the Fourier amplitude and basic frequency decrease when the sequence number changes from 29 to 57, as shown in Figure 18. The basic modes of the specimen, which are affected by the respective dynamic properties of the three parts (braced frame, steel frame, and shear wall), are also compared when the sequence number is 29, 41, and 57, and it has been found that the basic modes change obviously at the second, fourth, sixth, and eighth floors, as shown in Figure 19.

5.3. Strain Distribution

5.3.1. Braced Frame and Steel Frame. According to the theory of the structural mechanics and material mechanics, the strains on the braced frame are mainly caused by axial force, and the strains on the steel frame are mainly caused by

bending. The peak strains in the braced frame and steel frame, obtained from the test, are plotted in Figure 20. It has been found that

- (1) The peak strains mainly caused by axial forces in the braced frame (strain meters S1–S9) are much larger than the peak strains mainly caused by the bending moment in the steel frame (strain meters S10–S18).
- (2) The maximum value of the peak strains is located at the position of S5 (column of seventh floor), and the second largest value of the peak strain is located at the position of S7 (column of tenth floor).
- (3) The peak strains of S5 and S7 are larger than the yield strain of steel material when the sequence number is 56, and the remaining stains are much smaller than those of the steel.

Thus, it has been concluded that the braced frame and the steel frame basically remain in an elastic state.

5.3.2. *Concrete Wall.* The peak strains of the concrete wall are shown in Figure 21, and the corresponding values are listed in Table 8. It has been found that

- Thus, it has been concluded that the braced frame and steel frame basically remain in an elastic state.
2. *Concrete Wall.* The peak strains of the concrete wall shown in Figure 21, and the corresponding values are ed in Table 8. It has been found that
- (1) When the sequence numbers are 26 and 27, all the strains are less than 65E-06 (the cracking strain of the concrete in tension), and the concrete walls are in an elastic state. The first crack on the concrete wall is located at the position of S22 (third floor) when the sequence number is 28.
 - (2) When the sequence number is 38, the peak values of strain meters S19, S20, S21, S22, and S26 are larger than the strain of 65E-06 (see Table 8), which means that cracking occurs in the concrete at the second, fourth, and sixth floors.
 - (3) When the sequence number is 56, the maximum strain of the concrete wall is located at position S27 (the fifth floor), and the second largest strain is located at position S22 (the third floor).

- located at the position of S22 (third floor) when the sequence number is 28.
- (2) When the sequence number is 38, the peak values of strain meters S19, S20, S21, S22, and S26 are larger than the strain of 65E-06 (see Table 8), which means that cracking occurs in the concrete at the second, fourth, and sixth floors.
- (3) When the sequence number is 56, the maximum strain of the concrete wall is located at position S27 (the fifth floor), and the second largest strain is located at position S22 (the third floor).

FIGURE 10: Construction process of the specimen. (a) Construction of the steel frame and wall skeleton. (b) Making the concrete wall. (c) Installation of the additional mass. (d) Installation of the BRB.

- (4) The maximum compression strain of the concrete wall is $370E-06$ when the sequence number is 55, which is less than the cracking strain of the concrete in compression, so it has been concluded that all the cracks on the concrete wall are caused by tension.

As shown in Figure 22(a), the cracks appear at the first, second, fourth, and seventh floors when the sequence number is 29. A new crack appears at the fifth floor when the sequence number is 41, as shown in Figure 22(b). Many cracks occur from the first to the ninth floor of the concrete wall when the sequence number is 57, as shown in

Figure 22(c). Most of the cracks are horizontal cracks, which are caused by the bending moment on the concrete wall. Besides, the cracks in the concrete wall are not concentrated at the bottom of the shear wall; instead, the cracks are widely distributed on the concrete wall. All the cracks are micro-cracks after a severe earthquake. Most of the cracks are in the horizontal direction, which indicates that the damage to the concrete wall is mainly caused by the bending moment. Noticeably, there is no crack found on the concrete surrounding the connecting plate between the steel beam XL02 and the embedded column HZ-1 during the tests.

When the period change (see Figure 16(b)) was considered together with the crack evolution on the concrete

FIGURE 11: Measurement scheme. (a) Strain meters and rosettes. (b) Displacement meters. (c) Acceleration meters.

FIGURE 12: Specimen with additional masses. (a) Picture. (b) Drawing.

TABLE 5: Additional mass.

Floor	Additional mass (t)		
	Braced-frame	Frame	Shear wall
1 ~ 9	0.55	0.35	0.5
10	1.5	1.0	1.4
Mass on each part	6.45	4.15	5.9
Total mass	16.45

FIGURE 13: Comparison between the response spectrum of selected ground motions and design spectrum.

wall during the test, it has been found that the basic period of the specimen is 0.191s when the specimen has not yet been damaged. When SN arrives at 29, a small number of cracks are observed simultaneously on the 1st, 2nd, 4th, and 7th floors of the concrete shear wall (see Figure 22(a)), resulting in a decrease in structural stiffness and an increase of the period to 0.197 s. When SN equals 41, more cracks are found on the 1st, 2nd, 3rd, 4th, 5th, and 7th floors of the concrete wall (see Figure 22(b)). When SN is 57, lots of cracks are observed on the 1st, 2nd, 3rd, 4th, 5th, 6th, 7th, and 9th floor of the wall, and the cracks are evenly distributed on the concrete wall (see Figure 22(c)), and the value of the period arrives at 0.223 s.

5.4. Acceleration Response. The amplification factor of the acceleration is the ratio of the peak acceleration of each layer to the peak acceleration of the base plate. It is observed that the acceleration amplification factors increase when the sequence number changes from 26 to 38; however, they severely decrease when the sequence number changes from 38 to 54, as shown in Figure 23(a). As illustrated in Figure 23(b), the acceleration amplification factors between the first and fourth floors increase when the sequence number changes from 27 to 39; however, they decrease between the fifth and tenth floors when the sequence number changes from 27 to 39. When the sequence number changes from 39 to 55, the acceleration amplification factors significantly decrease between the first and fifth floors and

TABLE 6: Test sequence.

SN	Input	PGA (cm/s ²)
1	WN	29.4
2	EL	28.4
3	SH	
4	TJ	
5	WN	29.4
6	EL	77.4
7	SH	
8	TJ	
9	WN	29.4
10	EL	86.9
11	SH	
12	TJ	
13	WN	29.4
14	WN	29.4
15	EL	173.8
16	SH	
17	TJ	
18	WN	29.4
19	EL	197.5
20	SH	
21	TJ	
22	WN	29.4
23	EL	221.2
24	SH	
25	TJ	
26	WN	29.4
27	WN	29.4
28	EL	464.5
29	SH	
30	TJ	
31	WN	29.4
32	EL	489.8
33	SH	
34	TJ	
35	WN	29.4
36	EL	619.4
37	SH	
38	TJ	
39	WN	29.4
40	WN	29.4
41	EL	805.8
42	SH	
43	TJ	
44	WN	29.4
45	EL	979.6
46	SH	
47	TJ	
48	WN	29.4
49	EL	1078
50	SH	
51	TJ	
52	WN	29.4
53	WN	29.4
54	EL	1176
55	SH	
56	TJ	
57	WN	29.4

Note. SN = sequence number; PGA = peak ground acceleration; WN = white noise; EL = El Centro; SH = Shanghai; TJ = Tianjin.

FIGURE 14: Loading of the specimens.

FIGURE 15: Hysteretic curve of the specimen.

increase slightly between the sixth and tenth floors, as shown in Figure 23(b). In Figure 23(c), the changes in the acceleration amplification factors are more significant when the sequence number changes from 28 to 40, and the maximum amplification factor is 8.80 at the third floor, as listed in Table 9. When the sequence number changes from 40 to 56, the acceleration amplification factors decrease significantly.

By comparing Figures 22(a)–22(c), it has been found that when SN is 39 and 40, respectively (under the action of

Shanghai and Tianjin seismic waves), the acceleration amplification factors of the 1–4 floors of the specimen increase significantly, which explains why some cracks are found on the concrete wall of the 1–4 floors after the earthquake when SN is 29, as shown in Figure 22(b). When SN is 54, 55, and 56, respectively, lots of cracks are observed on the concrete wall of the 1–8 floors, which result in a decrease in the stiffness of the specimen and an increase in the basic period of the specimen. Therefore, the measured acceleration amplification factors on every floor of the specimen when SN

TABLE 7: Mechanical parameters of the BRB.

Specimen	k_e (kN/mm)	Tension				Compression			
		F_y (kN)	δ_y (mm)	$F_{t,u}$ (kN)	$\delta_{t,u}$ (mm)	F_y (kN)	δ_y (mm)	$F_{c,u}$ (kN)	$\delta_{c,u}$ (mm)
BRB	130.2	35.2	0.29	51.78	1.70	36.6	0.28	52.27	1.50

Note. k_e is the elastic stiffness of the BRB; F_y is the yield strength of the BRB; δ_y is the yield displacement of the BRB; $F_{c,u}$ is the ultimate strength of the BRB; and $\delta_{c,u}$ is the ultimate displacement of the BRB.

(a)

(b)

FIGURE 16: Evolution of basic frequency and period of the specimen. (a) Basic frequency. (b) Basic period.

is 54, 55, and 56 are smaller than the acceleration amplification factors when SN is 26, 27, and 28.

5.5. Displacement Response. Figures 24(a)–24(c) show the time-history displacement response of the top floor under the action of the El Centro, Shanghai, and Tianjin earthquakes, respectively. Figures 25–27 show the maximum layer

displacement and the peak interstory drift angle of the specimen. The maximum peak interstory drift angles of the specimens are listed in Table 10. The displacement response of the specimen becomes larger as the earthquake intensity increases. Noticeably, the maximum interstory drift angle is 1/40 for the El Centro earthquake, 1/54 for the Shanghai earthquake, and 1/53 for the Tianjin earthquake, which indicates that the ductility of the steel-concrete hybrid structure is very good.

FIGURE 17: Damping ratio of the specimen.

FIGURE 18: Fourier amplitude and frequency relation of the specimen.

FIGURE 19: Basic modes of the specimen.

FIGURE 20: Peak strains of the braced frame and steel frame. (a) Sequence number from 26 to 28. (b) Sequence number from 38 to 40. (c) Sequence number from 54 to 56.

FIGURE 21: Peak strains of the concrete wall. (a) Sequence number from 26 to 28. (b) Sequence number from 38 to 41. (c) Sequence number from 54 to 56.

TABLE 8: Peak strain values of the concrete wall.

SN	Peak strain (1.0 * E - 06)											
	S19	S20	S21	S22	S23	S24	S25	S26	S27	S28	S29	S30
26	14	48	38	38	8	6	28	48	12	18	4	6
27	16	40	46	46	18	6	34	32	12	20	4	18
28	16	48	62	80	18	6	50	44	18	18	4	8
38	72	66	114	152	12	14	14	106	42	32	18	10
39	126	60	92	272	38	18	32	66	60	54	10	12
40	152	80	142	346	46	28	44	136	316	68	32	178
54	114	96	142	108	32	18	28	76	400	58	40	40
55	190	102	120	124	40	20	38	58	754	106	42	230
56	210	152	190	186	56	54	78	132	1144	100	54	42

FIGURE 22: Crack evolution on concrete wall of axis A. (a) SN = 29. (b) SN = 41. (c) SN = 57.

FIGURE 23: Continued.

FIGURE 23: Envelope diagrams of the acceleration amplification factors. (a) El Centro. (b) Shanghai. (c) Tianjin.

TABLE 9: Maximum amplification factor of the acceleration.

Earthquake wave	El Centro			Shanghai			Tianjin		
SN	26	38	54	27	39	55	28	40	56
Maximum acceleration amplification factor	2.21	2.33	1.49	2.46	2.94	1.84	2.64	8.80	4.7

FIGURE 24: Continued.

FIGURE 24: Time histories of the roof displacement of the specimen. (a) El Centro. (b) Shanghai. (c) Tianjin.

FIGURE 25: Envelope diagrams of the displacement response under El Centro earthquake. (a) Maximum lateral displacement. (b) Interstory drift angle.

FIGURE 26: Envelope diagrams of the displacement response under the Shanghai earthquake. (a) Maximum layer displacement. (b) Interstory drift angle.

FIGURE 27: Envelope diagrams of the displacement response under the Tianjin earthquake. (a) Maximum layer displacement. (b) Interstory drift angle.

TABLE 10: Maximum interstory drift angle.

Earthquake wave	El Centro			Shanghai			Tianjin		
SN	26	38	54	27	39	55	28	40	56
Maximum interstory drift angle	1/397	1/273	1/40	1/154	1/70	1/54	1/254	1/95	1/53

6. Conclusions

According to the shaking table test of a 1/10 scale steel-concrete hybrid structure with BRBs, the following conclusions are drawn in this paper:

- (1) The recommended value of the damping ratio of the steel-concrete hybrid structure with BRBs is 3% under the action of frequent earthquake and is 4% under the action of rare earthquakes in engineering design.
- (2) All the cracks are microcracks under the action of severe earthquakes. No severe damage is observed on the specimen when the maximum interstory drift angle of the specimen arrives at 1/40, which means that the ductility of the steel-concrete hybrid structure with BRBs is very good.
- (3) The cracks on concrete wall are not concentrated at the bottom of the wall (a local failure mode); instead, they are widely distributed in floors 1–8 (a global failure mode).
- (4) No cracks are found on the concrete surrounding the connecting plate between the steel beam XL02 and the embedded column HZ-1 during the tests. The hinge connection with one high-strength bolt between steel and concrete wall may pledge that steel structure and concrete work together.
- (5) The steel frame and braced frame are basically in an elastic state after the severe earthquakes, which may

play the role of second defense line by bearing the increased internal force caused by the damage of the concrete wall.

- (6) None of the BRBs is damaged during the tests, and the gusset plates between the BRBs and the frame all remain in a good condition at the end of the tests.

Data Availability

The data supporting the conclusions of this study are available from the corresponding author upon request.

Additional Points

A steel-concrete hybrid structure with BRBs is tested on a shaking table. None of the BRBs is damaged during the shaking table test. Steel beams and columns are basically in an elastic state. All cracks on the concrete wall are microcracks. The ductility of the steel-concrete hybrid structure is very good.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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References

- [1] GB50017-2017, *Standard for Design of Steel Structures (GB50017-2017)*, China Architecture & Building Press, Beijing, China, 2017.
- [2] American Institute of Steel Construction (AISC), *Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-16)*, American Institute of Steel Construction, Chicago, IL, USA, 2016.
- [3] L. Li, G.-Q. Li, and Y.-S. Liu, "Simplified algorithm of the novel steel-concrete mixed structure under lateral load," *International Journal of High-Rise Buildings*, vol. 1, no. 4, pp. 247–254, 2012.
- [4] M. Jia, D. Lu, L. Guo, and L. Sun, "Experimental research and cyclic behavior of buckling-restrained braced composite frame," *Journal of Constructional Steel Research*, vol. 95, pp. 90–105, 2014.
- [5] D. Wu and Y. Xiong, "Tests study of a 1:20 scale steel-concrete hybrid structure," *Procedia Engineering*, vol. 210, pp. 441–448, 2017.
- [6] L.-H. Han, W. Li, and Y.-F. Yang, "Seismic behaviour of concrete-filled steel tubular frame to RC shear wall high-rise mixed structures," *Journal of Constructional Steel Research*, vol. 65, no. 5, pp. 1249–1260, 2009.
- [7] P. F. Xu, *Structural Design of Complex High-Rise Buildings*, China Architecture & Building Press, Beijing, China, 2005.
- [8] G.-Q. Li, X.-M. Zhou, and X. Ding, "Shaking table study on a model of steel-concrete hybrid structure tall buildings," *Journal of Building Structures*, vol. 22, no. 2, pp. 2–7, 2001.
- [9] T.-J. Lu, *Theoretical Analysis and Pseudo Dynamic Experimental Study on Tall Building Tubular Structure (Dissertation of PhD)*, Department of Civil Engineering, Central South University, Wuhan, China, 2008.
- [10] Y. Zhou, J. Yu, X.-L. Lv et al., "Shaking table model test of a high-rise hybrid structure building with steel frame-concrete core wall," *Journal of Earthquake Engineering and Engineering Vibration*, vol. 2, pp. 98–105, 2012.
- [11] Z.-X. Li, Y. Lv, L.-H. Xu, Y. Ding, and Q. Zhao, "Experimental studies on nonlinear seismic control of a steel-concrete hybrid structure using MR dampers," *Engineering Structures*, vol. 49, pp. 248–263, 2013.
- [12] X. Zhou and G. Li, "A macro-element based practical model for seismic analysis of steel-concrete composite high-rise buildings," *Engineering Structures*, vol. 49, pp. 91–103, 2013.
- [13] Y. Ding, M. Wu, L.-H. Xu, H.-T. Zhu, and Z.-X. Li, "Seismic damage evolution of steel-concrete hybrid space-frame structures," *Engineering Structures*, vol. 119, pp. 1–12, 2016.
- [14] M. Abhilasha, M. R. Eatherton, and R. Matsui, "Experimental investigation of miniature buckling restrained braces for use as structural fuses," *Journal of Constructional Steel Research*, vol. 127, pp. 54–65, 2016.
- [15] M. Dehghani and R. Tremblay, "An analytical model for estimating restrainer design forces in bolted buckling-restrained braces," *Journal of Constructional Steel Research*, vol. 138, pp. 608–620, 2017.
- [16] R. Tremblay, M. Dehghani, L. Fahnestock et al., "Comparison of seismic design provisions for buckling restrained braced frames in Canada, United States, Chile, and New Zealand," *Structures*, vol. 8, pp. 183–196, 2016.
- [17] L. Li, T.-H. Zhou, J.-W. Chen et al., "A new buckling-restrained brace with a variable cross-section core," *Advances in Civil Engineering*, vol. 2019, Article ID 4620430, 15 pages, 2019.
- [18] Y.-L. Guo, J.-S. Zhu, P. Zhou, and B.-L. Zhu, "A new shuttle-shaped buckling-restrained brace. Theoretical study on buckling behavior and load resistance," *Engineering Structures*, vol. 147, pp. 223–241, 2017.
- [19] L. Li, X.-F. Peng, and T.-H. Zhou, "Research on design method and mechanical behavior of square section buckling restrained brace with two yield stages," *Journal of Central South University (Science and Technology)*, vol. 47, no. 8, pp. 1–9, 2016.
- [20] G. Hector, T. Ji, J. A. Escobar et al., "Effects of buckling-restrained braces on reinforced concrete precast models subjected to shaking table excitation," *Engineering Structures*, vol. 163, pp. 294–310, 2018.
- [21] D.-Z. Hu, G.-Q. Li, and F.-F. Sun, "Full-scale shaking table tests on a hinge-connected steel frame with buckling-restrained braces," *China Civil Engineering Journal*, vol. 43, pp. 37–46, 2010.
- [22] L. Li, *Design Approach and Seismic Behavior Study on Novel Multi-Lateral Resistant Steel-Concrete Mixed Structure (Dissertation of PhD)*, Department of Civil Engineering, Tongji University, Shanghai, China, 2012.
- [23] H. Wu, G.-W. Zhang, J. Zhao et al., "Seismic performance of existing RC frame structures reinforced with buckling-restrained braces," *China Civil Engineering Journal*, vol. 7, pp. 37–46, 2013.
- [24] GB50010-2010, *Code for Design of Concrete Structures (GB50010-2010)*, China Architecture & Building Press, Beijing, China, 2010.
- [25] GB50011-2010, *Code for Seismic Design of Buildings (GB50011-2010)*, China Architecture & Building Press, Beijing, China, 2010.
- [26] Y. Zhou and X. Lv, *Method and Technology for Shaking Table Model Test of Building Structures*, Science press, Beijing, China, 2014.