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Abstract: A novel dual system composed of blind bolted end plate concrete filled steel tube (BECFT) composite frames and buckling-restrained braces (BRBs) was proposed and presented in this paper. The direct displacement-based design (DDBD) method was modified and used to design the BECFT composite frames with BRBs (BRB-BECFT) at a certain seismic hazard level. Meanwhile, a series of pseudo-dynamic tests (PDTs) were conducted on two specimens of 2/3-scaled two-story, one-bay BRB-BECFT composite frames. The test observations at different seismic hazard levels were recorded. It was indicated that the BRB-BECFT frame system exhibited reasonable failure mode, good hysteretic behavior, high ductility and sufficient energy-dissipating capacity. The installation of BRBs effectively enhanced the lateral stiffness and resistance of the dual system in comparison with their bare counterparts. Moreover, the inter-story drift responses of both specimens at the frequent occurrence earthquake (FOE) and the maximum considered earthquake (MCE) levels were less than the pre-defined values, and the experimental base shears were also close to the analytical results at the MCE level, suggesting the modified DDBD method could provide good control of drifts for the dual system at a target loading level. These research results presented a good alternative for application in earthquake resistant design of braced frame

structures.

Keywords: Buckling-restrained braces (BRBs); Concrete-filled steel tube (CFT); Blind bolt; Direct displacement-based design (DDBD) method; Pseudo-dynamic test (PDTs)

1 Introduction

Buckling-restrained braces (BRBs) as a displacement-type metallic yielding damper have been rapidly developing since firstly invented in Japan in 1970s. A typical BRB is usually composed of an inner steel core surrounded by an outer restrainer with an unbonding layer or air gap between the two units to reduce the friction developed on the steel core and accommodate transverse expansion in compression. Unlike traditional steel braces, a BRB can develop its full yield strength in both tension and compression with the help of the outer restrainer preventing the steel core from buckling in compression, resulting in a more stable and almost symmetric inelastic hysteretic behavior [1-7]. Therefore, BRBs are currently recognized as a very cost-effective lateral resistance member and are widely used in new and retrofit buildings which located in high seismic regions.

Previous studies [8-10] pointed out that the concrete filled steel tube (CFT) frame structure exhibited high vertical load-bearing capacity and superior constructability but a low lateral stiffness. In order to overcome this limitation and motivate the application of CFT structures in high- and even super high-rise buildings, research work on the CFT frame with BRBs (BRB-CFT) has been carried out [11-13] to evaluate the overall seismic performance of the dual system. Tsai and Hsiao [11] tested a full-scale three-story, three-bay BRB-CFT frame under hybrid pseudo-dynamic and quasi-static cyclic loadings. Two specimens of 1/4 scaled one-story, one-bay BRB-CFT frames were conducted by Ren et.al [12] under quasi-static cyclic loadings. Jia et.al [13] also reported a cyclic loading test of a 1/3-scaled two-story, one-bay BRB-CFT frame. The experimental and analytical studies confirmed that BRBs could not only improve the structural lateral stiffness and strength, but also absorb seismic energy and thus promote the seismic behavior of the structure after BRBs stepped into elastic-plastic stage.

However, the diaphragm connections and through-beam connections, which are classified as rigid connections, were still dominantly adopted in the beam-to-column joints of these CFT frames [8-13]. Meanwhile, considerable in situ welding or complicated embedded components are required for most rigid beam-to-column joints, and brittle fractures of welded connections were found during the post-earthquake survey [14]. To avoid the need of in situ welding and ensure the connection reliability, the blind bolted end plate joints to CFT columns with the advantage of installation on one side were proposed and tested by many researchers, such as Wang et.al [15, 16], Ataei et.al [17], Wang et.al [18], Tao et.al [19, 20] and Waqas et.al [21]. The analytical results [15-21] showed that properly designed blind bolted joints exhibited semi-rigid characteristics with relatively high initial stiffness, large rotation capacity and good ductility. In addition, the corresponding blind bolted CFT frames with or without concrete slabs on steel beams were systematically studied by the authors [22-26]. Nevertheless, the finding of relatively smaller lateral stiffness of the pure blind bolted CFT frames, when compared with those of rigid CFT frames, may result in the inability to be used in high-rise buildings.

Many country's design codes, such as European [27], United States [28] and Chinese [29], allow the use of semi-rigid joints for seismic applications under some conditions. However, since blind bolted CFT frame structures are still relatively new, little attention has been paid to develop a dual system of blind bolted CFT composite frames equipped with BRBs (BRB-BECFT) to efficiently resist the vertical gravity load, lateral seismic action, and improve energy dissipation capacity. On the other hand, it should be noted that the connection between BRBs and boundary members is of crucial importance due to the fact that axial tension and compression of the BRB need to be effectively delivered to frames through gusset plates to resist earthquake action. Previous system-level experiments [11, 30, 31] have shown that gusset plates were subjected not only to BRB axial force, but also to frame action resulting from opening and closing of beam-to-column joint. This behavior was found to cause that gusset plates became more susceptible to weld fracture near the gusset tips or buckling deformation prior to failure of the BRBs, thus impairing the seismic performance of the dual system. More importantly, the blind bolted CFT joints could normally exhibit higher rotation capacity [15-21], which may lead to more serious frame action of beam-to-column joints. Therefore, it is essential to check the mechanical performance of gusset plates in the blind bolted CFT frames.

Furthermore, an appropriate performance-based design procedure is also essential to determine properties of this type of semi-rigid braced frame structure and obtain its responses, like inter-story drift, under a certain seismic hazard level. Although the force-based design method is widely used in current building codes [27-29]. The design procedure generally offers rather simplified methods to obtain the design base shear from the codes specified spectral accelerations by assuming elastic behavior for the structure. Nevertheless, the given target inter-story drift and the required stiffness and strength of semi-rigid joints can not be directly considered when sizing each individual component of a semi-rigid frame. A more rational alternative, displacement-based design method which allows the design to a specific performance level, would overcome these drawbacks and expand the design scope available to engineers and clients.

Against above background, this paper presented a seismic design and an experimental study on the blind bolted CFT composite frame with BRBs, aiming to evaluate the design procedure and seismic response of the dual system at different loading levels. Firstly, the direct displacement-based design (DDBD) method [32-34] was modified and used to obtain the base shear of the BRB-BECFT composite frame. The dual system members, including BRBs, CFT columns, steel beams with steel-bars truss deck (SBTD) concrete slabs, blind bolted end plate composite joints and gusset plates, were designed based on gravity loads and seismic action. Then, two 2/3-scaled two-story, one-bay BRB-BECFT frames were constructed and a series of pseudo-dynamic tests (PDTs) were conducted. Subsequently, the test observations at different seismic hazard levels were recorded. The inter-story displacement response across the test time and hysteretic loops of both specimens were analyzed, and the experimental data was also employed to evaluate efficiency of the modified DDBD method. Finally, the stiffness degradation, ductility and energy-dissipating capacity of the BRB-BECFT frame were discussed.

2 Specimen design

The prototype structure is designed as the blind bolted CFT composite frames with BRBs and it is assumed to be located in a region with Site Classification II and design ground Group 3, having a basic design peak ground acceleration (PGA) of 0.3g with Seismic Intensity of 8 [35]. This 4-story structure is used to store books except the first story for parking, thus the height of the first story is 2.4 m, and the height of each story of the upper three stories is 3.0 m. The plan view of the structure was shown in Fig. 1. In order to obtain the seismic performance of the novel BRB-BECFT frame system, the test specimens with approximate 2/3 scaled versions of the prototype building were fabricated to accommodate the laboratory’s equipment in this experimental research.

The dual system was designed with a pre-defined limit state based on the direct displacement-based design (DDBD) method to predict structural response of the frames under a certain seismic hazard level. In order to investigate the influence of the column section type and the end plate type on the seismic of the novel BRB-BECFT frame system, two specimens of single-bay, two-story BRB-BECFT were manufactured in this paper. They were almost identical except for the shape of CFT columns and the type of end plates, as illustrated in Table 1 and Figs. 2-6. The circular CFT columns and curved end plates were used to specimen BBFD1, while the square CFT columns and rectangular end plates were applied to specimen BBCF2. The detailed design procedure of the BRB-BECFT frame was illustrated in Fig. 7. The test specimen BBCF2 was selected as a typical example to present the design process and corresponding results.

2.1 Development of equivalent single-degree of freedom (SDOF) system

The multi-degree of freedom (MDOF) system should firstly be characterized by a substitute SDOF system (Fig. 7(a)). The properties of a SDOF system can be defined by the design displacement (δ_{eq}), the effective mass (m_{eq}) and the effective height (H_{eq}) as shown in Fig. 7(a) with Eqs. (1)-(3), where m_i is the mass at story i; δ_i is the total design displacement at story i; H_i is the height at story i and n is the number of stories.

Currently, there is no performance criterion for the blind bolted CFT frames with BRBs. Meanwhile, the seismic design code [35] simply stipulates that the design story drift limit (θ_d) of the BRBF should meet the expected deformation control requirements, and its θ_d should be smaller than that of bare counterparts (θ_{df} = 1/50). In view of this, the story drift limit (θ_{di}) of the BRB-BECFT frame system is strictly decided as 1/80 to meet the requirement of the seismic design code [35] and to prevent bookshelves in the frame system from falling under severe earthquakes with Seismic Intensity of 8. Additionally, it is assumed that structural first modal design displacement profile can be simplified into an inverted triangle for the tested 2-story specimens. The mass of the first and second stories of specimen BBCF2 was 61.47 ton and 57.25 ton, respectively. Then, the results of dual system equivalent SDOF properties can be obtained and presented in Table 2.

2.2 Determination of dual system equivalent viscous damping (EVD)

The EVD is defined as a function of the expected hysteretic behavior and the system ductility at the design displacement (Fig. 7(b)). The dual system EVD (ξ_{eq}) can be calculated by the known respective viscous damping of blind bolted CFT frames (ξ_f) and BRBs (ξ_{brb}), as illustrated in Fig. 7(b) with Eqs. (4)-(8) [32, 33], where $\theta_{f,di}$ and $\theta_{f,yi}$ are referred as the design and yield story drift limit at story i for semi-rigid frames, respectively; $\theta_{brb,di}$ and $\theta_{brb,yi}$ are the design and yield story drift limit at story i for BRBs, respectively; μ_f and μ_{brb} represent the ductility of frames and BRBs and they are respectively simplified as the average of μ_{fi} and $\mu_{brb,i}$ for the 2-story braced frame; $\kappa_f = \mu_f^{\lambda}$ and $\kappa_{brb} = \mu_{brb}^{\lambda}$, hereinto λ is equal to 0.617 [33]; $\Delta\xi_{f,ela}$ and $\Delta\xi_{brb,ela}$ are the adjusted elastic damping and equal to the value of 3.0% [33] for both frames and BRBs; $M_{f,OTM}$ and $M_{brb,OTM}$ denote respectively the overturning resistance of frames and BRBs.

The Eqs. (9) and (10) can be used to evaluate the yield story drift for the blind bolted CFT frames [34]:

$$\theta_{fj,yi} = \frac{m_{jR}\phi_{by}}{6}\left(\psi_{jb} + \frac{0.5I_b h_i}{I_c L_b}\right) \tag{9}$$

$$\theta_{f,yi} = \sum_{j=1}^{m}\theta_{fj,yi}M_{jR} \Big/ \sum_{j=1}^{m}M_{jR} \tag{10}$$

where $m_{jR} = M_{jR}/M_{bR}$; $\phi_{by} = M_{bR}L_b/(EI_b)$; $\psi_{jb} = 1 + 6/k_j$; $k_j = S_{jini}/(EI_b/L_b)$; M_{jR} and M_{bR} are respectively the joint and beam moment resistances; I_b and I_c are respectively the beam and column moment of inertia; S_{jini} represents the initial stiffness of joints; L_b and h_i are respectively the beam length and column inter-story height; E is the Young's modulus of steel. It should be noted that M_{jR}, S_{jini}, M_{bR}, and I_b would exhibit different properties under sagging and hogging moments for the composite joints with RC slabs and steel-concrete composite beams. For the blind bolted composite joints to CFT columns, the M_{jR} and S_{jini} can be determined by means of component method by referring to EC3 [36], EC4 [37] and other research results [25, 38-41]. The M_{bR} can be calculated in accordance with GB50017 [29].

The yield story drift of the BRB-BECFT frame can be evaluated when the elastic segment deformation of BRB is ignored [5]:

$$\theta_{brb,yi} = 2\rho\varepsilon_{cy}\alpha_c / \sin 2\varphi \tag{11}$$

where ε_{cy} is the core plate yield strain of the BRB; φ means the inclination angle of the BRB to the horizontal beam; α_c is the BRB effective factor and equals to L_y/L_{wp}, hereinto L_y represents the length of BRB yielding segment, L_{wp} is the work point-to-work point distance; ρ denotes the amplification factor considering the effects of all other member deformation in the same story and assumed to be 1.25.

In addition, the $\theta_{f,di}$ and $\theta_{brb,di}$ were set at 1/80 in this paper. The square blind bolted CFT frame with BRBs (specimen BBCF2) was taken as an example and the computation results of design ductility were listed in Tables 2 and 3 using the dimensions of specimen BBCF2 and tensile coupon test results. Then, the ξ_f and ξ_{brb} were respectively 6.87% and 20.88% according to Eqs. (6) and (7). The overturning moment results from equivalent force profiles were shown in Table 4. Thus, the dual system EVD, ξ_{eq}, was 13.3% following Eq. (8).

2.3 Calculation of effective period from reduced displacement spectrum

Fig. 7(c) illustrated the relationship between design acceleration response spectrum and corresponding reduced displacement spectrum as expressed in Eq. (12), where g is the acceleration of gravity as 9.8 m/s^2; α denotes the seismic effect coefficient and can be presented by using multiple functions [35], as shown in Fig. 7(c); the term αg means the spectral acceleration; $\alpha_{\max}$ is the maximum seismic effect coefficient; T_g is the characteristic period; γ, η_1 and η_2 represent the attenuation index, slope adjustment coefficient and damping adjustment coefficient, respectively [35]:

$$\begin{cases} \gamma = 0.9 + \dfrac{0.05 - \xi_{eq}}{0.3 + 6\xi_{eq}} \\ \eta_1 = 0.02 + \dfrac{0.05 - \xi_{eq}}{4 + 32\xi_{eq}} \geq 0 \\ \eta_2 = 1 + \dfrac{0.05 - \xi_{eq}}{0.08 + 1.6\xi_{eq}} \geq 0.55 \end{cases} \tag{13}$$

The spectrum reduction factor (η_δ) [27] was introduced and used to construct an inelastic design displacement spectrum:

$$\eta_\delta = \sqrt{\frac{0.1}{0.05 + \xi_{eq}}} \tag{14}$$

Based on the site characteristic of buildings, the $\alpha_{\max}$ and T_g under a severe earthquake are 1.2 and 0.45s, respectively. The γ, η_1, η_2 and η_δ are 0.82, 0.01, 0.72 and 0.74, respectively, when the ξ_{eq} was equals to 13.3% according to the dual system described above. Then, the design acceleration response spectrum can be established through the aforementioned Eq. (13), as illustrated in Fig. 8. Subsequently, the inelastic design displacement spectrum was obtained through Eq. (12). Finally, the effective period, T_{eq}, can be found to be 0.445s when the design displacement δ_{eq} was equal to 31.15 mm, as depicted in Fig. 9.

2.4 Determination of design base shear and force distribution along frame height

The design base shear V_{eq} can be obtained by Eqs. (15) and (16) as shown in Fig. 7(d) after finishing above mentioned work. The design base shear was distributed along the frame height by using Eq. (17), then, the lateral design story forces can be obtained. Therefore, for specimen BBCF2 with square CFT columns, the V_{eq}, F_1, F_2 were respectively 657.52 kN, 225.95kN and 431.57kN and were listed in Table 5.

2.5 Identification of members for the BRB-BECFT frame

The cross-sectional area (A_{cp}) of BRB core plate can be determined according to the Eq. (18):

$$A_{cp} = \frac{P_{b\max}}{\omega_b \beta_b f_{cp,y}} \tag{18}$$

where $P_{b\max}$ is the maximum axial force that the BRB needs to resist under a severe earthquake determined by DDBD method; ω_b denotes the strain hardening adjustment factor and can be taken as 1.5 for the Grade Q235B steel in accordance with JGJ99-2015 [42]; β_b is the compression strength adjustment factor and equals to 1.15 [5]; $f_{cp,y}$ represents the yield stress of the core plate and can be obtained from the coupon test.

On the basis of the prescribed story drift limit and previous studies [11-13], it is assumed that 50% and 70% of the first- and second-story shear forces were resisted by the BRBs, thus, the axial force demand, $P_{b\max}$, in the first- and second-story BRB was 408.50kN and 382.22kN, respectively, shown in Table 5. The cross-sectional area (A_{cp}) of BRB in the first and second stories was 817mm^2 and 765mm^2, respectively, when the $f_{cp,y}$ was equal to 289.8N/mm^2 obtained from the coupon test. Therefore, to simplify the manufacture process, the dimension of core plate of the first- and second-story BRB can be both equal to 100×8mm (Fig. 6).

The composite beam and CFT column members were designed considering the capacity design requirements. The selection of square column is the concrete filled 200mm width steel tube with a wall thickness of 8mm. The primary beams are chosen as commercial H-shape steel beam with a cross-section of HN300×150×6.5×9mm, and 100mm steel-bars truss deck (SBTD) concrete slabs are installed in the first- and second-story beams by means of shear studs to form full shear connected steel-concrete composite beams, as illustrated in Figs. 2-4 and Table 1. The SBTD is composed of steel-bars truss and thin-walled steel sheeting (Fig. 3). They were welded together in a workshop, then can be transported to the site. Subsequently, the SBTD can be paved directly on the steel beams markedly reducing the construction workload on site.

For the blind bolted end plate composite joints to CFT columns, the moment resistance of the joint should be checked by using the Eq. (19) [25]:

$$M_{jR} \le M_{bR} \tag{19}$$

The joint design results, m_{jR}, shown in Table 3 satisfied the requirement expressed in Eq. (19). The configuration and dimension of the blind bolted end plate composite joints were presented in Figs. 2, 4 and 5.

Additionally, the design motivation of gusset plates is to suppress buckling behaviors of gusset plates and to avoid failure of the gusset-to-beam or gusset-to-column weld interfaces. Four design check contents were summarized and described as follows:

(1) Yielding check of gusset plates under tension

In order to prevent gusset plate under tension from yielding, it should satisfy the following Eq. (20)[5, 28]:

$$DCR_{gty} = \frac{P_{b\max} / \beta_b}{\phi f_{gp,y} b_e t_{gp}} \le 1.0 \tag{20}$$

where the maximum axial force can be named as $P_{b\max}$ and can be obtained by using Eq. (18); $f_{gp,y}$ and t_{gp} mean the yield strength and thickness of gusset plates, respectively; ϕ = 0.9; $b_e = \min\{W_{whitmore}, W_{actual}\}$, where $W_{whitmore} = 2L_w \tan 30^{\circ} + D_b$, as shown in Figs. 10 and 11.

(2) Buckling check of gusset plates under compression

The compressive strength of gusset plates should meet the requirement of DCR_{gb} given in Eqs. (21) and (22) to prevent gusset plate from happening buckling failure [28]:

$$DCR_{gb} = \frac{P_{b\max}}{\phi b_e t_{gp} P_{gpcr}} \le 1.0 \tag{21}$$

$$P_{gpcr} = \begin{cases} 0.658^{\lambda_c^2} f_{gpy} & \lambda_c \le 1.5 \\ 0.877 f_{gpy} / \lambda_c^2 & \lambda_c > 1.5 \end{cases} \tag{22}$$

where, $\lambda_c = \frac{a_w D_w}{r}\sqrt{\frac{f_{gpy}}{\pi^2 E}}$; $D_w = \frac{L_1 + L_2 + L_3}{3}$; $r = \sqrt{\frac{t_{gp}^2}{12}}$; ϕ = 0.9; a_w = 2.0 for the gusset plates without edge stiffeners [11].

(3) Check of fillet weld size on the gusset-to-beam or column interfaces

The failure of the gusset-to-beam or gusset-to-column weld interfaces should be avoided to ensure force transfer between the BRBs and boundary members [28]:

$$DCR_{cw} = \frac{1.25\sqrt{V_{c,c}^2 + H_{c,c}^2}}{\phi V_{an,c}} \leq 1.0 \tag{23}$$

$$V_{an,c} = 1.4 h_{f,c} L_v (0.6 f_{exx}) \left[1 + 0.5 \sin^{1.5} (\tan^{-1} \left| \frac{H_{c,c}}{V_{c,c}} \right|) \right] \tag{24}$$

$$DCR_{bw} = \frac{1.25\sqrt{(\frac{V_{b,c}}{1.4 h_{f,b} L_h} + \frac{M_{ub}}{1.4 h_{f,b} L_h^2 / 6})^2 + (\frac{H_{b,c}}{1.4 h_{f,b} L_h})^2}}{\phi 0.6 f_{exx}} \leq 1.0 \tag{25}$$

where L_h and L_v are the length and height of gusset plates, respectively; $h_{f,c}$ and $h_{f,b}$ denote the fillet weld size on the gusset-to-column or beam interfaces, respectively; f_{exx} is the welding material strength; ϕ = 0.75; $V_{c,c}$ and $H_{c,c}$ represent the vertical and horizontal combined actions on the gusset-to-column interfaces when the BRB is subjected to compressive loading; $V_{b,c}$ and $H_{b,c}$ are the vertical and horizontal combined actions on the gusset-to-beam interfaces when the BRB is subjected to compressive loading. The four parameters resulting from the BRB and the frame actions can be respectively obtained by using the generalized uniform force method [43] and the improved equivalent strut model [30]. In addition, it should be noted that the extra moment, M_{ub}, resulting from regular shape of gusset plates should be taken into account to check the weld size on the gusset-to-beam interfaces [31].

(4) Check of Von Mises yield criterion on the gusset-to-beam or column interfaces

The stress response on the gusset-to-beam or column interfaces when the BRB reaches to the maximum compressive force can be evaluated as follows [30]:

$$DCR_{c,von} = \frac{\sqrt{(\frac{H_{c,c}}{L_v t_{gp} + w_{sf} t_{sf}})^2 + 3(\frac{V_{c,c}}{L_v t_{gp} + w_{sf} t_{sf}})^2}}{\phi f_{gpy}} \leq 1.0 \tag{26}$$

$$DCR_{b,von} = \frac{\sqrt{\left(\frac{V_{b,c}}{L_h t_{gp} + w_{sf} t_{sf}} + \frac{M_{ub}}{L_h^2 t_{gp}/4}\right)^2 + 3\left(\frac{H_{b,c}}{L_h t_{gp} + w_{sf} t_{sf}}\right)^2}}{\phi f_{gpy}} \leq 1.0 \tag{27}$$

where w_{sf} and t_{sf} are respectively the width and thickness of edge stiffeners on the gusset plates; $\phi =$ 1.0.

The check results of gusset plates of specimen BBCF2 were displayed in Table 6. It was indicated that the other three types of gusset plates satisfied the design check requirement, while the type of gusset plate-1 did not fully meet the requirements of Von Mises yield criterion on the gusset-to-beam interfaces.

As the specimen design mentioned-above mainly involved the blind bolted square CFT composite fames with BRBs (BBCF2), to further understand the seismic performance of the BRB-BECFT frame system, the blind bolted circular CFT composite fames with BRBs (BBFD1) were also studied based on specimen BBCF2. The detailed configuration and dimensions of the two specimens were illustrated in Figs. 2-6 and Table 1.

3 Experimental preparation

3.1 Assembly of the specimen

The dual system members, mainly containing the steel tube columns, steel beams, BRBs, SBTDs, and gusset plates, manufactured in a factory and delivered to the laboratory. The primary steel beams were firstly welded with flush or extended end plates as a member, then they were assembled to the steel tube columns using blind bolts, while the secondary steel beams were connected to columns by means of flange-welded and web-bolted method (Figs. 4 and 5) as the second step. Thirdly, after the gusset plates were welded to the adjacent beams and columns, the SBTDs were paved on the first- and second-story beams. In the next stage, two rows of shear studs with the diameter of 16 mm and height of 90 mm were welded to the primary beams, while single row of shear studs was welded to the secondary beams (Fig. 4). Then, self-consolidating concrete mix was filled in the steel tube columns, and the ordinary concrete was poured into the SBTDs. Finally, the

BRBs were welded to the first- and second-story gusset plates to form the novel dual system.

3.2 Test setup and procedures

The foundation of the BRB-BECFT specimen was fixed to the laboratory's strong floor using post-tensioned anchors and rods (Figs. 12 and 13). Lateral displacements at each floor were imposed by two 1000 kN electro-servo hydraulic actuators mounted horizontally on the left edges of the test structure as shown in Figure 12. Pulling on the BRB-BECFT specimen was achieved by means of four steel rods. Two 2000 kN hydraulic jacks were installed at the upper end of the columns to apply the vertical loads, and the ratio of axial compression on CFT columns was 0.3 taking the upper floor load into account. In addition, the 'Positive Direction' and 'Negative Direction' of displacement and load were illustrated in Fig. 12.

Pseudo-dynamic loading method was used in this paper. Specimens were considered as a two-degree-of-freedom system under an earthquake wave and this theoretical model could be conducted by solving the following equation of motion:

$$M \times a^n + C \times v^n + R^n = -M \times a_g^n \tag{28}$$

where M and C are the mass and viscous damping matrix; R^n represents the restoring force vector at time n; a^n, v^n and $a_g{}^n$ are the nodal acceleration vectors, velocity vectors and ground acceleration at time n, respectively. The story mass was 61.47 ton for the first floor and 57.25 ton for the second floor for each specimen.

The El-Centro earthquake record was chosen as the input ground motion in the PDTs to match the assumed Site Classification II. In order to investigate the performance of the BRB-BECFT structure at different PGA levels, a total of six generated earthquake scenarios were imposed sequentially, as illustrated in Table 7. They approximately represent a frequent occurrence earthquake (FOE, PGA=0.1g) with a 63% chance of exceedance in 50 years (simplified as 63/50), a 10/50 design bass earthquake (DBE, PGA=0.3g), a 2/50 maximum considered earthquake (MCE, PGA=0.5g), MCE-after I (PGA=0.8g), MCE-after II (PGA=1.0g) and MCE-after III (PGA=1.2g). In view of the fact that the process of PDTs was about 1000 times slower than the real time earthquake duration, the 8.5s of first original earthquake record was selected as it contains important acceleration information for the El-Centro record. Then, the original time histories were scaled down into 8.5 seconds multiplying a factor of $1/\sqrt{2/3}$ as specimens are scaled down to its 2/3 size. The scaled ground acceleration time history records were shown in Fig. 14.

3.3 Actual material properties

Material tests were implemented to determine actual mechanical properties of materials used in the specimens. The results of steel material were presented in Table 8.

The concrete specimens were casted in the columns and slabs at size of 150 × 150 × 150 mm and 150 × 150 × 300 mm were tested to obtain the cube compressive strength and elastic modulus, respectively. The average cube compressive strength of concrete poured in the columns is 53.6 MPa at 28 days and the elastic modulus is 34.6 GPa. The average cube compressive strength of concrete used in the slabs is 29.4MPa at 28 days and the elastic modulus is 28.7 GPa.

4 Experiment observations

4.1 Test No. 1 (FOE level, PGA = 0.1 g)

The experimental peak inter-story drift ratio (IDR) of specimen BBFD1 reached to 0.14% and 0.10% in the first and second stories, respectively (Fig. 15(a)). The IDR values were 0.13% and 0.12% in the first and second stories of specimen BBCF2 (Fig. 16(a)). There was no a visible crack in the first and second stories of SBTD concrete slabs, indicating that both specimens existed in a nearly linear elastic stage.

4.2 Test No. 2 (DBE level, PGA = 0.3 g)

At a time step of 2.16 s, the first minor crack near the left column in the first-story slab of specimen BBFD1 could be observed and then propagated to about 120 mm length around the perimeter of column, while no other slab cracks were detected for specimen BBCF2. The inter-story displacement histories versus time of both specimens were shown in Figs. 15(b) and 16(b). The maximum IDRs of specimen BBFD1 were measured as 0.45% and 0.31% in the first and second stories, respectively, and those of specimen BBCF2 were respectively 0.33% and 0.34%.

4.3 Test No. 3 (MCE level, PGA = 0.5 g)

The scheduled ground accelerations were used in the subsequent PDTs. The peak IDRs of the first story for specimen BBFD1 and BBCF2 increased to 0.96% and 0.55%, respectively, and the second-story peak IDRs reached to 0.70% and 0.63%, respectively, as illustrated in Figs. 15(c) and 16(c). In addition, the inter-story displacement response of specimen BBFD1 was greater than that of specimen BBCF2 due to the fact that the inertia moment of square CFT column is larger than that of circular CFT column at the same width and steel ratio of column section. Some small cracks began to appear near the columns in the first story for both specimens whereas no cracks were observed in the second story. Additionally, there was no visible buckling deformation on the BRBs and gusset plates, which illustrated that they can perform well to dissipate input energy under the MCE excitation level.

4.4 Test No. 4 (MCE-after I level, PGA = 0.8 g)

After the first three earthquake loading levels, it appeared that all BRBs had not undergone severe damage and the blind bolted composite CFT joints had not generated apparently nonlinear deformation. Thus, other loading states with higher PGA were imposed on the BRB-BECFT specimens to explore the failure modes of the braces and the frames.

In the test No. 4 of specimen BBFD1, three cracks near the right circular CFT column appeared in the first-story slab at the time step of 1.76 s, and cracks around the left column began to spread to two edges of the slab. The first tiny crack in the roof floor slab was observed around the right column. For specimen BBCF2, the cracks in the first-story slab mainly occurred in the vicinity of CFT columns, and a crack was detected near the gusset plate at a time step of 3.46 s.

Although the out-of-plane buckling of upper gusset plate in the first-story BRB was detected for both specimens, they can still transfer force between the frame and BRB, and there was no obvious drop in the base shear capacity of the BRB-BECFT frame as shown in Figs. 19 and 20. The first-story maximum IDRs of specimen BBFD1 and BBCF2 were 2.86% and 1.47%, respectively, and the second-story maximum IDRs were 1.82% and 1.27%, respectively, as depicted in Figs. 15(d) and 16(d). The maximum IDRs increase greatly by comparing with that in the MCE loading level and all of them were greater than the pre-defined limit state value of 1.25%.

4.5 Test No. 5 (MCE-after II level, PGA = 1.0 g)

It was observed that new cracks appeared and original inclined cracks continuously propagated in the first-story slabs and only a few cracks occurred in the second-story slabs for both specimens under this earthquake loading level. Significant out-of-plane deformation can be found in the first-story upper gusset plates for both specimens, which subsequently caused the local flexural buckling of the BRB at the weaken segment between transition and connection portion as shown in Fig. 18a. As a result, second-story BRBs started to resist larger earthquake action and their upper gusset plates began to buckle due to greater combined forces from frame action and BRB axial forces. Additionally, a slight bending of the curved extended end plate at left joint of specimen BBFD1 occurred, whereas the flat extended end plate at the left joint of specimen BBCF2 had a greater deformation than that of specimen BBFD1.

According to Figs. 15(e) and 16(e), the peak IDRs of specimen BBFD1 in the first and second stories rose to 3.45% and 1.72%, respectively; and the IDR values of specimen BBCF2 were 2.86% and 1.49%, respectively.

4.6 Test No. 6 (MCE-after III level, PGA = 1.2 g)

Although the above-mentioned local damage existed, there was no significant collapse occurring for the frames. Therefore, it was indicated that both specimens still partly have earthquake resistance, then, another test at a hazard level ground motion with PGA of 1.2 g should be performed to reveal the remaining capacity of the specimens.

It could be seen that upper gusset plates of both specimens in the second story happened apparent bending (Figs. 17(a) and 18(a)). Then, the seismic action was rapidly transferred to the frames due to the failure of BRBs. Subsequently, many cracks appeared in the first- and second-story slabs, but the number of cracks in the first-story slab was greater than that in the second-story slab. (Figs. 17(b) and 18(b, c)). Moreover, a relatively obvious bending deformation could be observed on the extended end plates, and the deformation of flat extended end plates was greater than that of curved extended end plates (Figs. 17(c) and 18(d)). The slight local buckling on the left beam flange around the end of first-story upper gusset plate-to-beam was also detected for specimen BBCF2 (Fig. 18(e)).

At the time of 2.47 s, a small fracture was found initially at the left column adjacent to the tip of stiffener-to-column welds and then propagated into a great fracture across the column for the specimen BBFD1 (Fig. 17(d)). Whereas for the specimen BBCF2, there was weld fracture occurring at the tip of gusset-to-column near the base of right column (Fig. 18(f)). The test was terminated when a drop of actuator load happened suddenly. At the end of test No. 6, the maximum IDRs of specimen BBFD1 reached to 10.0% and 3.35% in the first and second stories, respectively, whereas the IDRs of specimen BBCF2 were 7.69% and 4.76%, respectively (Figs. 15(f) and 16(f)).

5 Experimental results and discussions

5.1 Hysteresis curves

The first- and second-story shear versus story drift curves of the two specimens can be obtained from the electro-servo hydraulic actuator system in different PDTs, as illustrated in Figs. 19 and 20. According to Figs. 19(a) and 20(a), the linear story shear versus story drift curves in the first and second stories of both specimens indicated that the BRB-BECFT frame system remained in elastic stage. According to Section 2, as the building located in the region with Seismic Intensity of 8, the input ground motion of PGA = 0.1g was imposed on specimens, which approximately corresponds to a FOE level. Therefore, it met the requirement that the frame structure with BRBs should be kept in an elastic condition under the FOE loading level [35], consequently, BRBs can stably provide lateral stiffness for the global structure.

The curves of inter-story shear versus drift started to exhibit an inelastic behavior in the test No. 2 (Figs. 19(b) and 20(b)), whereas only a crack appeared in specimen BBFD1 and no visible deformation can be observed in both specimens, so it was estimated that BRB core plates occurred yielding, it could prevent the main frames from yielding like a damper having a function of energy dissipation.

As the PGA was increased to 0.5g approximately regarding as an MCE level, both specimens still behaved a good seismic performance in terms of their stable hysteresis behavior as shown in Figs. 19(c) and 20(c). It can be seen that the average maximum base shears of specimen BBFD1 and BBCF2 reached to 643.42 kN and 636.26 kN, respectively. According to Table 5, it was found that the analytical base shears of specimen BBCF2 by using the DDBD method was 657.52kN. The ξ_{eq} and T_{eq} of specimen BBFD1 could be respectively determined to be 12.6% and 0.436s by using Eqs. (8) and (12), then the base shear of specimen BBFD1 can be given to be 684.94 kN. A good agreement between experimental and analytical results of specimen BBFD1 and BBCF2 was respectively achieved each other, which indicated that the base shear can be predicted well by using the DDBD method under a certain earthquake loading states.

During the following loading scenarios, the out-of-plane deformation was successively occurred in the first- and second-story BRBs. Moreover, it can be observed that the number of slab cracks increased along with end plates and steel beam flanges deforming. Although the stiffness of both specimens gradually declined, the hysteretic loops still exhibit in a relatively plump condition as shown in Figs. 19 and 20, which demonstrated that the seismic performance of the proposed BRB-BECFT frame system was good.

5.2 Skeleton curves

The inter-story shear versus drift skeleton curves of both specimens can be formed by connecting the peak points of loadings at various seismic hazard levels from PGA of 0.1g to 1.2g in Figs. 19 and 20. According to Fig. 21, it can be illustrated that both specimens exhibited higher initial lateral stiffness and bearing capacity in comparison with pure blind bolted CFT composite frames reported by the authors [23]. The three characteristic points, including yield point, peak and failure point, on the inter-story skeleton curves were listed in Table 9. The first inflection point of skeleton curves is defined as yield point depicted in Fig. 21. The failure point could be equal to the 85% of its peak loading or the corresponding loading when the test stopped.

Comparing to specimen BBFD1, the maximum bearing capacities of the first and second story of specimen BBCF2 increased from 12.98 to 24.11% and from 23.25 to 37.17%, respectively. The main reason is that the inertia moment of square CFT column is greater than that of circular CFT column at the same width and steel ratio of column section. In addition, the inter-story maximum bearing capacities of specimen BBFD1 in the first and second story showed a significant enhancement in the range of 100.97 to 131.42% and 94.88 to 111.41% higher than those of the pure blind bolted circular CFT frames (specimen SBFD2) in accordance to the inventory data of Ref. [23], indicating that the energy dissipation device BRB could improve the lateral resistance more effectively.

5.3 Stiffness degradation

The stiffness degradation of both specimens at different hazard levels can be evaluated in the Eq. (29) as follows:

$$K_i = \frac{|+F_i| + |-F_i|}{|+\delta_i| + |-\delta_i|} \tag{29}$$

where $+F_i$ and $+\delta_i$ are the positive inter-story peak lateral load and its corresponding drift at the i^{th} hazard level; $-F_i$ and $-\delta_i$ are the negative inter-story peak lateral load and drift at the i^{th} hazard level.

The first and second inter-story stiffness degradation were presented in Fig. 22. It was showed that the inter-story stiffness of both specimens gradually and stably decreased with PGA increasing from 0.1g to 1.2g owing to occurrence of cracks on the slabs, deformation of blind bolted joints and the failure of BRBs. As a whole, the positive inter-story stiffness at each seismic hazard level was less than the negative value, especially in the last three seismic loading levels. This may be due to that the axial compression capacity of BRB started to decline at the last three loading process when the actuators exerted positive loads on the specimen, whereas the BRB could still withstand tension load when the actuators imposed negative loads on the specimen.

According to Fig. 22, as the stiffness of specimen BBCF2 with square CFT columns was larger than that of specimen BBFD1 with circular columns at almost each seismic hazard level, it was thought that the type of column cross section had an impact on the stiffness degradation. On the other hand, inter-story elastic stiffness of specimen BBFD1 in the first and second story was respectively about 6.2 and 3.4 times higher than those of pure blind bolted frame SBFD2 according to previous test results [23], whereas for the inter-story elastic stiffness of specimen BBCF2, they were respectively 4.8 and 2.5 times greater than those of the pure blind bolted frame SBFD1.

5.4 Ductility

For the Test No. 1, the first and second peak inter-story drift of specimen BBFD1 reached to 0.14% and 0.10%, respectively; and the corresponding values were 0.13% and 0.12%, respectively, for specimen BBCF2, as shown in Figs. 15(a) and 16(a). They were close but less than the corresponding pre-designed yield story drift of 0.15% and 0.13% of the first and second stories according to the Eq. (11), suggesting that the both specimens remained in an elastic stage. When specimens were subjected to the scaled EI-Centro earthquake records with PGA = 0.5g closing to a MCE level, the peak inter-story drift of the first story of specimen BBFD1 and BBCF2 increased to 0.96% and 0.55%, respectively, and the corresponding peak IDRs in their second-story were 0.70% and 0.63%, respectively, as illustrated in Figs. 15(c) and 16(c). Therefore, it was found that all the peak IDRs were less than the pre-determined story drift limit value of 1.25% and met the presupposed drift requirement. On the other hand, the inter-story drifts of both specimens were greater than the experimental yield drifts as shown in Table 9, while no apparent damage appeared on the frame members and joints, proving that the yielding of the BRBs can be considered as the first defense to dissipate energy under a severe earthquake to postpone damage of the main frame. According to Table 9 and Fig. 21, it can be known that the average first and second yield inter-story drift was 0.24% and 0.17% for specimen BBFD1, respectively; and the corresponding values of specimen BBCF2 were 0.27% and 0.17%, respectively. Therefore, the ratio of experimental yield drift to pre-set yield drift was respectively 1.57 and 1.78 of the first story whereas that of second story was both 1.29. Thus, to more reasonably predict the yield drift of BRB-BECFT frames, the amplification factor ρ should be amended as a constant of 2.0 according to the Eq. (11). In addition, the experimental ductility existed within a range of 8 to 34, and they were nearly close or greater than the analytical ductility at the magnitude of 8.79. Therefore, it was indicated that the blind bolted CFT frames with BRBs exhibited good ductility performance.

5.5 Energy dissipation

The curves of energy dissipation response versus time history for the two specimens were depicted in Fig. 23. The final details of energy dissipation were summarized in Table 10. It can be shown that a remarkable growth of energy dissipations could occur with seismic hazard level increase, for instance, from 430 kN·mm in the Test No. 1 to 206640 kN·mm in the Test No. 6 demonstrating that the energy dissipation capacity of BRB-BECFT frames was sufficient to resist an actual severe earthquake.

According to the Fig. 23 and Table 10, the energy dissipated in the first and second stories of both specimens in the Test No. 1 was greatly less than the other five tests, suggesting specimens in the test No. 1 stage were almost under an elastic state. The specimen BBFD1 and BBCF2 in the experiments dissipated a total energy of 28184 kN·mm and 23243 kN·mm, and energy dissipation in the first-story accounted for 74% and 68% of that in the entire frame system, respectively, which illustrated that as the first-story BRB absorbed most of seismic energy, it happened failure in advance of the second-story BRB. It can be seen that the total dissipated energy of specimen BBFD1 was about 1.21 times as much as that of specimen BBCF2. It may be due to the fact that although the above mentioned analysis showed that the specimen BBCF2 with square CFT columns exhibited greater stiffness and strength than those of specimen BBFD1, specimen BBCF2 showed relatively small deformation when they subjected to the same earthquake level, resulting in less dissipated energy than that of specimen BBFD1.

6 Conclusions

In order to investigate the seismic behavior of blind bolted CFT composite frames with BRBs, two 2/3 scaled sub-structure modes were designed based on the modified DDBD method. They were subjected to a series of PDTs to obtain their seismic performance under different loading levels. Summaries and conclusions can be drawn as follows on the basis of tests and analysis:

(1) Only a small number of cracks appeared in the first-story SBTD concrete slabs of the BRB-BECFT frame system at the loading states of Test No. 1, 2 and 3, however, the seismic forces were mainly transferred to the frame after the first- and second-story upper gusset plates successively buckled at the loading states of Test No. 4, 5 and 6, therefore, resulting in the damage of the frame including slab cracks, bending deformation of extended end plates and beam flanges, fracture at the tip of stiffener-to-column and gusset-to-column base.

(2) The experimental results indicated that the BRB-BECFT frame system exhibited a stable hysteretic behavior, high ductility and sufficient energy-dissipating capacity. The installation of BRBs effectively enhanced the lateral stiffness and resistance of the dual system in comparison with

their bare counterparts presented in a previous work.

(3) The seismic behavior of the BRB-BECFT frame system was affected by the column section type. At the same width and steel ratio of column section, the strength and stiffness of square CFT braced frames were greater than those of circular CFT braced frames, while it would be opposite in terms of the total dissipated energy under the same earthquake level.

(4) The inter-story drift responses of both specimens at the FOE and MCE levels were less than the pre-defined values, and the experimental base shears were close to the analytical results at the MCE level. The results indicated the efficiency of the modified DDBD design procedure for the BRB-BECFT frame system.

(5) BRBs were in an elastic state and provided lateral stiffness to the blind bolted CFT frame at the FOE level, moreover, they absorbed most of seismic energy to prevent the main frame from premature failure under a severe earthquake.

(6) Although the check results of gusset plates could fulfill the requirement under consideration of BRB forces and frame action, they started to buckle at the loading states of Test No. 4, 5 and 6, suggesting that further research on the design of gusset plate in BRB-BECFT frame system should be imperative.

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