

Article

Effect of End-Plate Thickness on the Seismic Performance of K-Shaped Eccentrically Braced Frames with End-Plate Connections

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Abstract

To investigate the effects of end-plate thickness on the seismic performance and damage evolution of K-shaped eccentrically braced frames with end-plate connections (EPEBFs), two scaled specimens with different end-plate thicknesses were tested under low-cycle reversed loading. The influence of end-plate thickness on mechanical behavior and seismic response was evaluated from failure modes, hysteretic response, skeleton curves, ductility, energy dissipation, and stiffness degradation, together with damage assessment using different models. Refined finite element models were then established for parametric analysis. The results show that EPEBFs exhibit a well-defined plastic development path. Damage in the reference specimen was mainly concentrated in the link and its end-plate connection region, thereby protecting the frame columns, beams, and braces. For the two thicknesses tested, the specimen with thicker end plates showed higher lateral resistance, greater ultimate deformation capacity, and better late-stage resistance retention, but lower displacement ductility, indicating enhanced absolute deformation capacity but reduced post-yield deformation reserve. Thicker end plates also altered stress transfer and plastic development in the connection regions, causing damage to extend toward the frame-beam ends. Among the damage models considered, the elastic–plastic energy dissipation ratio model better captured the accumulation of energy dissipation and plastic deformation. The finite element results agreed well with the tests. Within the investigated range, increasing end-plate thickness mainly improved ultimate deformation capacity and late-stage resistance retention, without a clear monotonic effect on lateral resistance. The findings provide a basis for prefabricated design, damage control, and post-earthquake replacement and repair of EPEBFs.

Keywords: K-shaped eccentrically braced frames; end-plate connections; end-plate thickness; seismic performance; damage assessment



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1. Introduction

Installing diagonal braces in moment-resisting steel frames (MRFs) is a common strategy for improving structural lateral stiffness. According to the connection position

and force-transfer mechanism between the braces and frame beams, braced steel frames can generally be classified into concentrically braced frames (CBFs) and eccentrically braced frames (EBFs) [1,2]. Concentrically braced frames can readily satisfy lateral stiffness requirements. However, under strong earthquakes, the braces are prone to lateral buckling during repeated tension–compression cycles. This behavior leads to rapid degradation of lateral stiffness and energy dissipation capacity, making it difficult to develop a stable ductile energy dissipation mechanism. In contrast, eccentrically braced frames introduce a shear link between the brace and the frame beam, so that seismic damage is mainly concentrated in the plastic deformation region of the link. This mechanism prevents premature brace buckling and provides favorable stiffness, ductility, and energy dissipation capacity under different seismic hazard levels. Therefore, eccentrically braced frames are regarded as a seismic structural system with a well-defined damage path and favorable energy dissipation characteristics [3–5].

Regarding the mechanical mechanisms and seismic performance of EBFs with different configurations, Popov et al. [6] conducted a series of studies on single-diagonal and gabled EBFs. They investigated the failure mechanisms and energy dissipation characteristics of EBFs. Design methods for links were also proposed in their studies. These studies clarified the advantages of EBFs in terms of lateral stiffness and ductility. To improve the uniformity of plastic deformation in EBF links along the height of high-rise buildings, Bosco et al. [7,8] proposed a dual eccentric system. This system consisted of EBFs and moment-resisting frames and was developed based on capacity design principles. The performance-based design parameters were defined according to the dynamic response of the frame structure. A comparison was then conducted between the proposed system and code-designed EBFs. Lian et al. [9,10] designed K-shaped and Y-shaped EBFs with hybrid high-strength steel configurations. In these frames, Q460C steel was used for beams, columns, and braces, whereas Q345B steel was adopted for links. Their results showed that the hybrid high-strength steel EBFs exhibited slightly lower lateral stiffness and energy dissipation capacity than conventional EBFs made entirely of Q345B steel. However, the use of high-strength steel reduced the steel consumption. To achieve the seismic fortification objectives and shear distribution demands of high-strength steel EBFs under different seismic intensity levels, Li et al. [11] proposed a multi-objective performance-based plastic design method. This method was developed based on monotonic and cyclic loading test results. Dynamic response analyses of a 10-story high-strength steel EBF showed that the structure remained elastic under minor earthquakes. Under moderate earthquakes, the links acted as the first seismic defense line and dissipated energy. Under severe earthquakes, the structure developed an ultimate plastic state, and the link regions eventually resisted a larger proportion of the shear demand. Montuori et al. [12] further developed the theory of plastic mechanism control and proposed an optimized design method for K-shaped EBFs. In this method, the plastic rotation of links was adopted as the kinematic design parameter. The expected damage mechanism could therefore avoid unfavorable collapse modes. The calculation procedure was efficient and robust, and the structural economy was improved through optimization. Pushover analyses and incremental dynamic analyses of structures with different heights verified the accuracy of the optimized design method based on the theory of plastic mechanism control.

Fully welded joints in steel structures are susceptible to brittle fracture under strong earthquakes. Accordingly, semi-rigid bolted connections have gradually become an important connection form in steel structures [13,14]. Among various semi-rigid connection types, end-plate connections have been widely used in steel beam-to-column joints because of their simple fabrication, high construction efficiency, and broad applicability [15–17]. End-plate connections are also suitable for EBFs, particularly because they can provide

links with favorable energy dissipation capacity and post-earthquake replaceability. For end-plate-connected EBF links, Dubina et al. [18] developed a removable link design using flush end-plate bolted connections. Their results showed that EBFs with end-plate connections experienced a certain reduction in initial stiffness. However, they still satisfied seismic requirements in terms of serviceability, ultimate, and collapse-prevention limit states. This configuration also facilitated post-earthquake repair. Ioan et al. [19] performed pseudo-dynamic tests on a three-story EBF structure with end-plate connections. They analyzed the deformation response of the structure under different seismic input levels. Repair strategies were further proposed according to the residual deformation of the replaceable shear links. Yin et al. [20,21] developed a welding access hole design for I-shaped shear links in EBFs with end-plate connections. Their results indicated that the welding access holes improved the stability of hysteretic curves and produced a more uniform stress distribution over the cross-section. The proposed restoring force model also showed good applicability for evaluating the response of stiffened I-shaped shear links.

To compare the performance of end-plate-connected shear links with different cross-sectional forms in EBFs, Aziz et al. [22] proposed bearing capacity prediction equations for short I-section and box-section links. The interchangeability of these two types of shear links was verified through an equivalent evaluation of hysteretic performance and energy dissipation capacity. Li et al. [23,24] proposed two mid-span splice configurations for end-plate-connected shear links in EBFs. High-strength bolts were used in both splice configurations, and slotted holes further improved the energy dissipation capacity of the shear links. These splice configurations effectively reduced end-plate deformation and facilitated the post-earthquake removal and replacement of shear links. The length ratio of shear links is a key factor affecting the seismic performance of EBFs. Ma et al. [25] investigated the effectiveness of very short shear links with end-plate connections. Calculation formulas for shear capacity and overstrength behavior were derived, and a collaborative design method for end-plate-connected very short shear links was proposed. The above studies indicate that end-plate connections in existing EBFs are mainly applied to links. However, frame beams and diagonal braces may still be damaged under rare and extremely rare earthquakes. Therefore, it is necessary to develop fully prefabricated EBFs based on bolted connections to further improve seismic resilience and post-earthquake recoverability.

To further investigate the influence of end-plate thickness on the mechanical behavior and seismic performance of EBFs with end-plate connections, two prefabricated K-shaped EBF specimens were designed in this study. All member-to-member connections in the specimens were assembled using end-plate connections. Only the end-plate thicknesses at the link-to-frame-beam and brace-to-frame-beam connections were varied, while the remaining connection details were kept unchanged. End-plate thickness was therefore selected as the primary design variable. Low-cycle reversed loading tests were conducted to compare the macroscopic failure phenomena, hysteretic response, stiffness degradation, and energy dissipation capacity of the specimens with different end-plate thicknesses. The damage evolution of the specimens was further evaluated using the Hwang model [26], modified Park–Ang model [27], and elastic–plastic energy dissipation ratio model [28]. Based on the experimental results, a refined finite element model was established and validated. Extended parametric analyses were then performed to further reveal the influence of end-plate thickness on the seismic performance of K-shaped EBFs with end-plate connections. The results provide a reference for the seismic design, damage control, and prefabricated application of EBFs with end-plate connections.

2. Experimental Program

2.1. Specimen Design

Seismic performance tests were conducted on prefabricated K-shaped eccentrically braced frames. The test specimens were derived from a six-story steel frame structure, and the selected location is shown in Figure 1. The prototype structure was designed in accordance with the Standard for Design of Steel Structures (GB 50017-2017) [29] and the Code for Seismic Design of Buildings (GB 50011-2010) [30]. The story height, span, and height-to-span ratio were 3.6 m, 6.0 m, and 0.6, respectively. The structure was located in a region with a seismic fortification intensity of 8 degrees. The site category was Class II, and the design earthquake group was Group 2. Owing to laboratory space limitations, the overall geometry of the test specimens was scaled by 1:2, whereas the end plates, high-strength bolts, and associated connection details were retained at practical dimensions to ensure constructability and engineering representativeness. Although some scale effects on the local connection response cannot be completely excluded, the same scaling strategy was adopted for both specimens, with all connection details other than the investigated end-plate thickness kept unchanged. Therefore, the two specimens provide a consistent basis for comparative evaluation of the effect of end-plate thickness. Two prefabricated K-shaped EBF specimens were designed, each with a height of 1.8 m and a span of 3.0 m. The specimens consisted of frame columns, frame beams, braces, and links, all of which were fabricated from H-shaped steel sections. The cross-sections of the frame columns, frame beams, braces, and links were $H200 \times 200 \times 8 \times 12$, $H250 \times 125 \times 6 \times 9$, $H125 \times 125 \times 6.5 \times 9$, and $H250 \times 125 \times 6 \times 9$, respectively. Previous studies have shown that reducing the yield strength of the links in EBFs can improve their plastic deformation and energy dissipation capacities, thereby protecting the primary structural members, including the frame columns, frame beams, and braces. Therefore, Q345B steel was used for the frame columns, frame beams, and braces, whereas Q235B steel was used for the links. To satisfy the requirements for prefabricated engineering applications of EBFs, end plates were provided at the ends of the frame columns, frame beams, braces, and links. All members were assembled using Grade 10.9 M20 high-strength bolts, thereby forming a fully prefabricated frame. All high-strength bolts were pretensioned using a torque wrench, with an applied pretension force of 155 kN. The tightening procedure was conducted in accordance with the Technical Specification for High Strength Bolt Connections of Steel Structures (JGJ 82-2011) [31] and included initial tightening followed by final tightening.

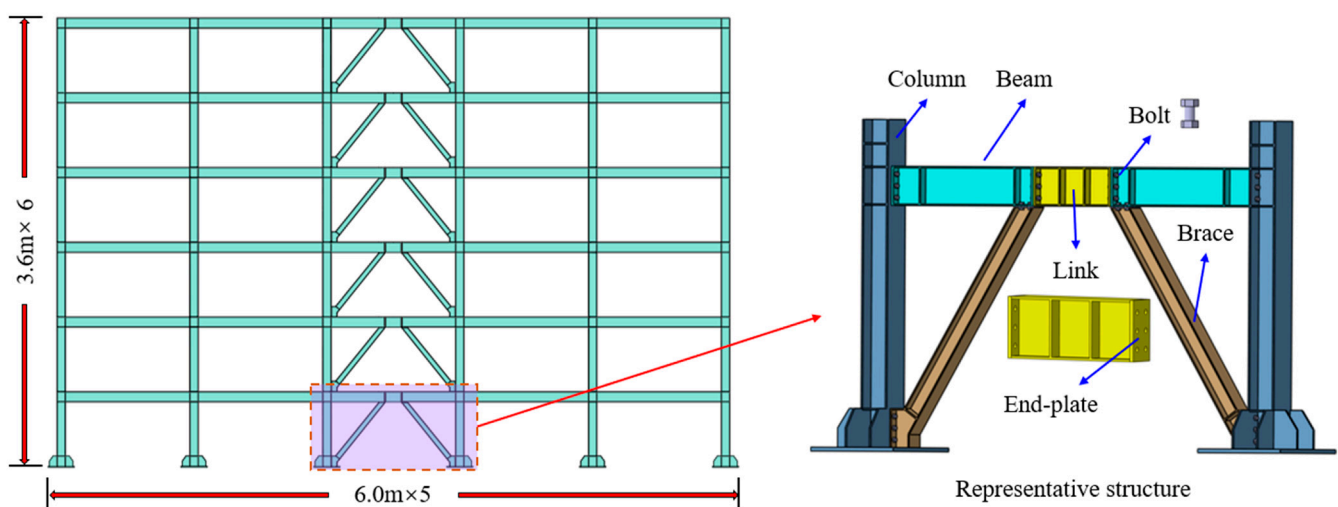


Figure 1. Prototype structure.

The link length ratio, ρ , is an important parameter affecting the seismic performance of EBFs. The design method for determining the link length ratio is specified in AISC 341-10 [32] and AISC 360-10 [33], as expressed in Equation (1).

$$\rho = e \frac{V_p}{M_p} \quad (1)$$

where e is the link length, M_p is the plastic flexural capacity of the link, and V_p is the plastic shear capacity of the link. The values of M_p and V_p are calculated using Equations (2) and (3), respectively.

$$M_p = f_y W_p \quad (2)$$

$$V_p = 0.58 f_y h t_w \quad (3)$$

where f_y is the yield strength of steel, W_p is the plastic section modulus of the link, h is the effective web depth of the link, and t_w is the web thickness of the link. The yielding mode of the link can be classified according to the link length ratio ρ . The link is classified as a shear link when $\rho \leq 1.6$, a flexural-shear link when $1.6 < \rho < 2.6$, and a flexural link when $\rho \geq 2.6$.

The dimensions of the test specimens designed in this study are shown in Figure 2. The link length, e , was 600 mm for both specimens, and the corresponding link length ratio, ρ , was calculated as 1.57. End-plate thickness is an important structural parameter affecting the mechanical behavior of the end-plate connection regions and the seismic response of the structure. According to the design requirements for the end-plate thickness, d , specified in the Standard for Design of Steel Structures (GB 50017-2017) [29], the reference specimen EPEBF1 was first designed. The end-plate thicknesses at both the link-to-frame-beam and brace-to-frame-beam connections were set to 16 mm. Specimen EPEBF2 was then designed based on EPEBF1, with the end-plate thicknesses at these two connections increased to 24 mm to investigate the influence of end-plate thickness on the seismic performance of the structure. Except for the end-plate thickness, the end-plate dimensions, bolt arrangement, and other connection details were kept unchanged for the two specimens. The primary structural parameters of the specimens are listed in Table 1.

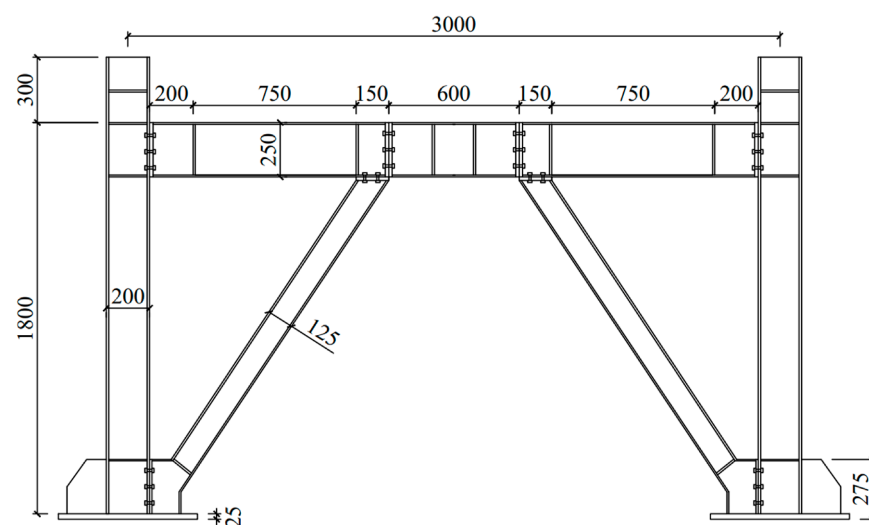
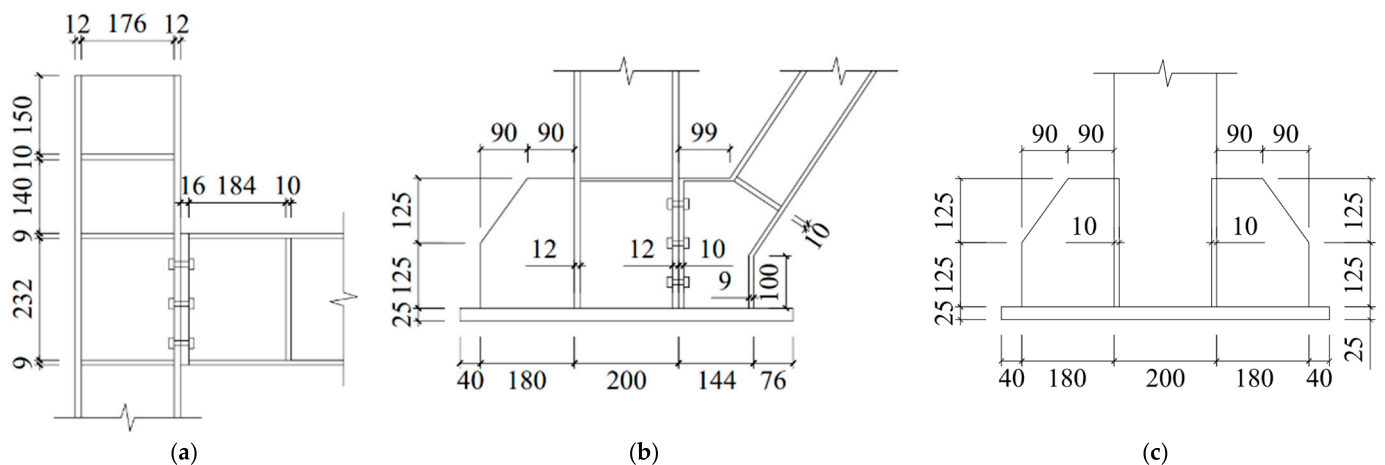
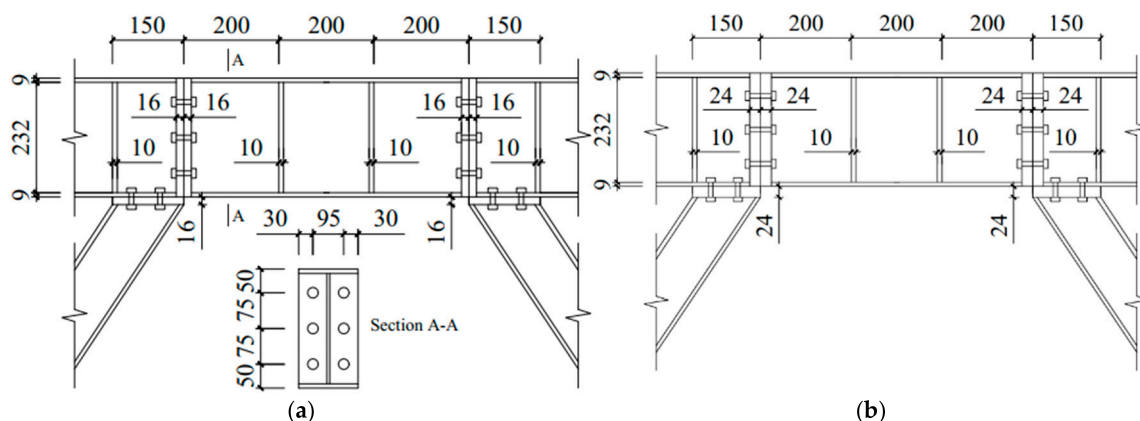


Figure 2. Dimensions of EPEBF1 (unit: mm).

Table 1. Construction information of test specimens.

Specimen Number	e/mm	ρ	d/mm	Number of Bolts
EPEBF1	600	1.57	16	44
EPEBF2	600	1.57	24	44

Base plates with a thickness of 25 mm were installed at the column bases to increase the contact area between the column bases and the top surface of the foundation. Column-base stiffener plates with a thickness of 10 mm were arranged between the column ends and the base plates to reduce the bending moments in the base plates and satisfy the bearing capacity requirements of the column bases. Pre-drilled end plates were provided at the ends of all members, and the frame was fully assembled using high-strength bolts. The bolt spacing satisfied the relevant code requirements. The connection details of EPEBF1 are shown in Figure 3 as an example. Each link was fabricated by welding end plates to an H-shaped steel section. Web stiffeners with a thickness of 10 mm were symmetrically arranged on both sides of the link web, and the stiffener spacing was 200 mm for both specimens. The detailed configurations of the links are shown in Figure 4.

**Figure 3.** Connection details of EPEBF1 (unit: mm). (a) Beam–column joint. (b) Front view of column-base and brace joint. (c) Side view of column-base.**Figure 4.** Dimensions of links (unit: mm). (a) EPEBF1. (b) EPEBF2.

2.2. Material Properties

Tensile coupons were extracted from the flange and web of each structural component. Three standard coupons were prepared for each sampling location and tested at room

temperature in accordance with Metallic Materials-Tensile Testing-Part 1: Method of Test at Room Temperature (GB/T228.1-2021) [34]. The test results for each group were averaged, and the resulting mechanical properties of the steel are listed in Table 2. The representative experimental stress–strain curves of Q345B and Q235B steel are shown in Figure 5.

Table 2. Material properties of the steel.

Steel Type	Position	Thickness/mm	Yield Strength f_y /MPa	Ultimate Strength f_u /MPa	Elastic Modulus/GPa	Yield Strain/%	Elongation/%
Q345B	Column web	8	379	543	224	0.210	30
Q345B	Column flange	12	337	526	224	0.179	34
Q345B	Beam web	6	369	536	201	0.180	32
Q345B	Beam flange	9	356	530	227	0.180	29
Q345B	Brace web	6.5	334	461	220	0.165	28
Q345B	Brace flange	9	289	460	225	0.167	27
Q235B	Link web	6	271	447	236	0.163	35
Q235B	Link flange	9	261	427	241	0.157	30
Q235B	Link end plate	16	253	458	218	0.155	34
Q235B	Link end plate	24	241	445	217	0.153	32
Q235B	Link web stiffener	10	258	462	211	0.162	35

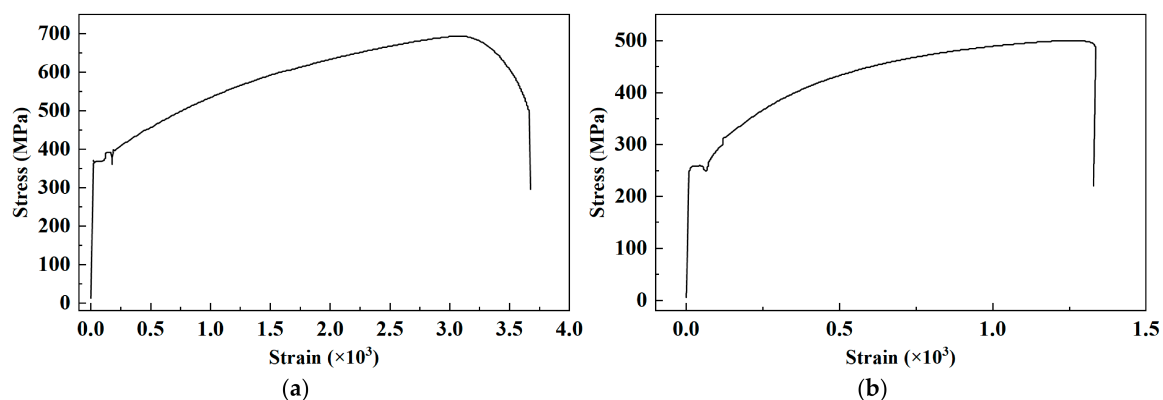


Figure 5. Stress–strain curves. (a) Q345B. (b) Q235B.

2.3. Loading System and Measuring Point Layout

The test setup is shown in Figure 6. All specimen members were prefabricated in the factory and transported to the laboratory, where they were assembled using high-strength bolts. The column bases were connected to the test foundation using anchor bolts to provide fixed boundary conditions. Horizontal loading was applied at the beam end. A 1000 kN horizontal actuator was connected to the east beam end of the specimen to apply reversed lateral loads. Two 2000 kN vertical actuators were positioned above the two column tops to apply axial compression. Roller assemblies were installed at both ends of the vertical actuators to maintain a constant axial load during the lateral displacement of the specimen. Lateral restraint frames were arranged on both sides of the specimen to prevent out-of-plane deformation during loading. A low-cycle reversed loading protocol was adopted. The pushing and pulling directions were defined as positive (+) and negative (−), respectively. A combined force–displacement control method was used. The vertical load was determined based on the gravity loads of the prototype structure in accordance with the Load Code for the Design of Building Structures (GB 50009-2012) [35]. After accounting for the dead loads, live loads, and self-weight of the structural members, a

constant axial load of 200 kN was applied to each column through the vertical actuators before lateral loading, resulting in a total vertical load of 400 kN on the specimen.

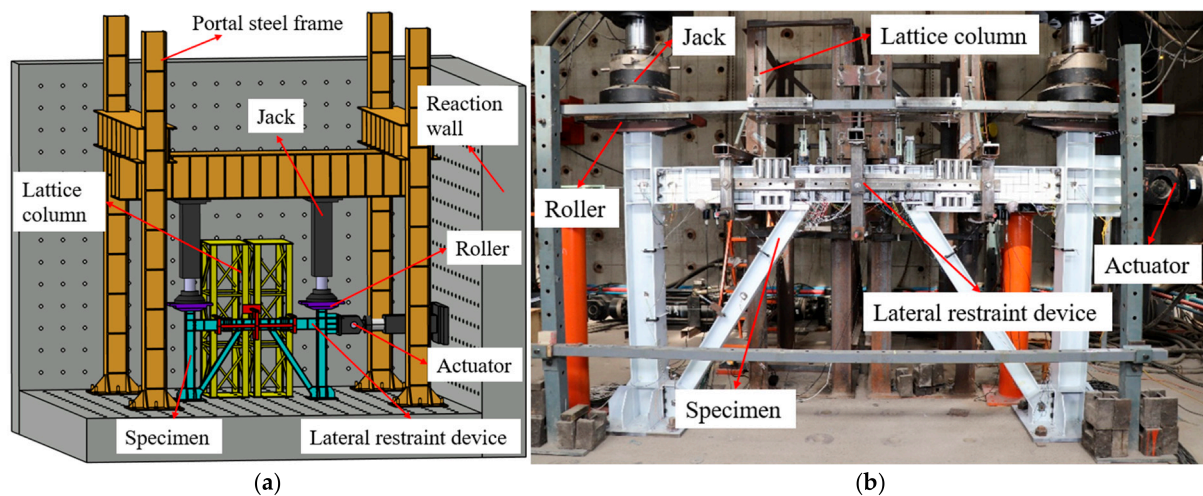


Figure 6. Loading device diagram. (a) Schematic diagram. (b) Photo.

The loading protocol was established in accordance with the Specification for Seismic Test of Buildings (JGJ/T 101-2015) [36]. A combined force–displacement control method was adopted. Force control was used before yielding. The yield state of each specimen was determined from the strain gauges. When the strain at a critical location first reached the corresponding material yield strain, the associated lateral displacement was taken as the yield displacement, Δ_y . After yielding, the loading mode was switched to displacement control, and the displacement amplitude was progressively increased in multiples of Δ_y . In accordance with JGJ/T 101-2015, three loading cycles were performed at each displacement level after yielding. The test was terminated when the lateral resistance decreased to 85% of the peak resistance or when obvious buckling, tearing, or other significant failure phenomena were observed. The loading protocol is shown in Figure 7.

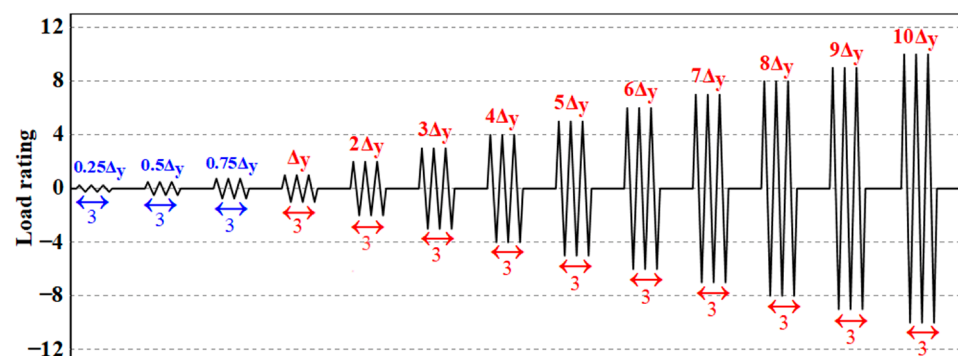


Figure 7. Cyclic loading protocol.

The measured responses included the global lateral displacement and local strains of the specimens. The instrumentation layout of the displacement transducers and strain gauges is shown in Figure 8. Each specimen was equipped with six displacement transducers. Among them, D1 and D2 were horizontally arranged along the upper portion of the frame beam to measure the lateral displacement of the specimen. Four vertical displacement transducers were installed on both sides of the link-to-frame-beam connection region to monitor the shear deformation of the link. A total of 43 strain measurement locations were arranged on each specimen. Because the link was expected to be the primary

plastic deformation region, 17 strain measurement locations were placed in the link region. Since the shear response of the link was mainly characterized by web buckling and the development of shear strains, strain rosettes were installed in the panel zone and on the link web to capture the local strain response.

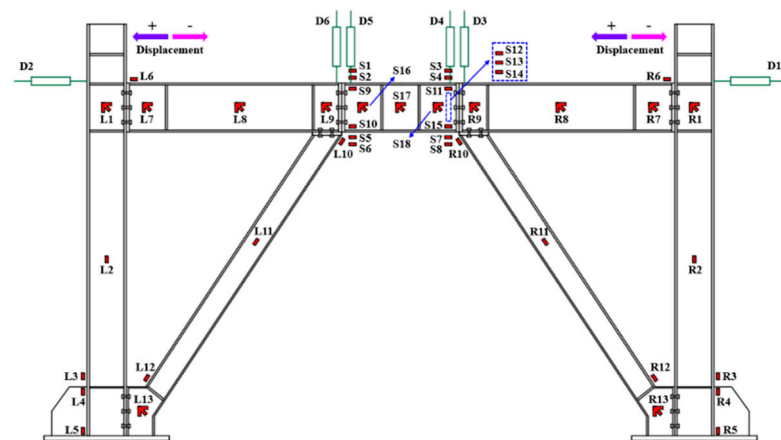


Figure 8. Layout of displacement transducers and strain gauges.

3. Test Results and Analysis

3.1. Experimental Macroscopic Phenomena

Based on the yield criterion described in Section 2.3, the yield displacement of EPEBF1 was determined to be 2.89 mm. The first yielding occurred in the link web adjacent to the end plate. No obvious damage was observed during the initial loading stage. However, audible noises were heard during loading, which were mainly attributed to bolt slip and contact between the connection surfaces. When the loading displacement reached 5.8 mm, local buckling began to develop in the link flange. As the loading displacement increased, relative slip gradually occurred at the connections between the structural members. Owing to the local deformation of the link end plate, a distinct gap developed at the frame-beam-to-link end-plate connection. At a loading displacement of 11.6 mm, paint spalling was observed at the column base under the action of base shear. When the displacement reached 26.1 mm, noticeable out-of-plane deformation developed in the frame column, and the gap at the frame-beam-to-link end-plate connection increased to approximately 12 mm. At a loading displacement of 29 mm, the lower flange of the east-side link fractured completely at its connection to the end plate, and the test was terminated. The link exhibited pronounced shear deformation under both positive and negative loading. The final damage was mainly concentrated along the upper and lower edges of the link end plate and the adjacent regions. The typical failure mode of EPEBF1 at the final loading stage is shown in Figure 9a.

Using the same criterion, the yield displacement of EPEBF2 was determined to be 6.04 mm. The first yielding also occurred in the link web adjacent to the end plate. During the early loading stage, local buckling developed in the link end plate, accompanied by gap opening at the link-to-frame-beam connection. Meanwhile, local yielding occurred at the column base when the loading displacement reached 7.2 mm. At a loading displacement of 9.6 mm, pronounced local deformation developed in the lower flange of the west-side link near the end plate. Owing to the increased end-plate thickness, the out-of-plane restraint in the connection region was enhanced, resulting in more severe local deformation and damage in the link flange. When the loading displacement reached 14.4 mm, out-of-plane bulging developed in the link web and gradually evolved into diagonal wrinkling. Because the end-plate thickness at the frame-beam-to-link connection was also increased, local

buckling occurred in the frame beam flange at a loading displacement of 16.8 mm. Gap opening was also observed at the frame-beam-to-brace end-plate connection. As plastic deformation continued to accumulate near the contact interface between the frame beam and the link end plate, the buckling deformation of the flange and web in this region became progressively more severe. The final failure was characterized by complete fracture of the lower flange of the west-side frame beam at its connection to the end plate, accompanied by diagonal cracking of the web extending from the fracture location. The typical failure mode of EPEBF2 at the final loading stage is shown in Figure 9b.

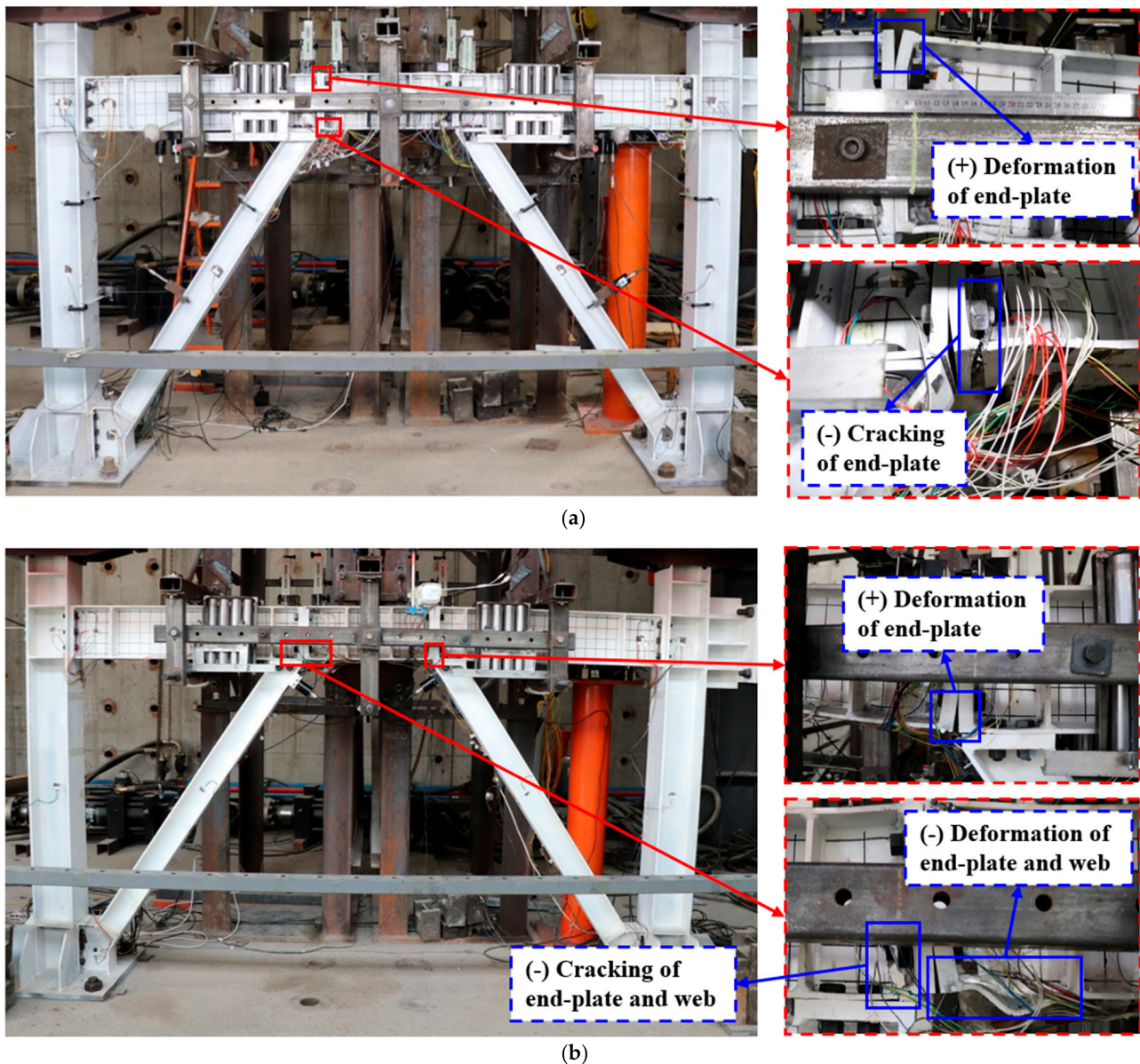


Figure 9. Observed failure modes of the specimens. (a) EPEBF1. (b) EPEBF2.

After the tests, the links were removed for inspection. Obvious frictional slip marks were observed around the bolt holes in the end plates, together with varying degrees of bearing deformation along the hole walls. These observations indicate that the bolts in the end-plate connection regions experienced significant slip and bearing damage under the combined action of shear force, bending moment, and axial force, resulting in degradation of the local load-bearing capacity. For EPEBF1, the main damage was concentrated in the link and its end-plate connection region, whereas no severe damage occurred in the primary

structural members, including the frame columns, frame beams, and braces. This indicates that the proposed connection configuration provided a well-defined damage localization mechanism and exhibited potential for post-earthquake removal and replacement. In contrast, the use of thicker end plates in EPEBF2 enhanced the restraint effect in the connection region and increased the yield displacement, ultimate deformation capacity, and late-stage resistance retention of the specimen. However, the increased end-plate stiffness also altered the stress-transfer mechanism and plastic development path in the connection region. Consequently, local stresses were transferred from the link and its end-plate connection region toward the frame-beam end, eventually causing fracture of the beam flange and diagonal cracking of the beam web. Therefore, although increasing the end-plate thickness can enhance the restraint capacity of the connection region, excessive end-plate stiffness may cause the damage location to shift toward the primary structural members, thereby compromising member protection and limiting the development of structural ductility.

3.2. Hysteretic Curve Analysis

Figure 10 presents the hysteretic curves of the specimens. The horizontal axes represent the loading displacement and inter-story drift ratio, respectively, whereas the vertical axis represents the lateral resistance. To clearly illustrate the difference in connection configuration between the two specimens, the end-plate thickness, d , is indicated in the figure in millimeters. The two specimens exhibited generally similar hysteretic responses. Before shear yielding of the links, both specimens remained in the elastic stage, and the lateral load increased approximately linearly with the horizontal displacement. As loading continued, the links underwent shear yielding and entered the strain-hardening stage, while the lateral resistance of the frames continued to increase with the loading displacement. With the progressive accumulation of plastic deformation in the links and their end-plate connection regions, the resistance of the specimens gradually reached its peak. Subsequently, damage such as local buckling, widening of the connection gaps, and local tearing continued to develop. After entering the plastic stage, the hysteretic curves of both specimens exhibited different degrees of pinching, which were mainly attributed to high-strength bolt slip, relative slip between the end-plate contact surfaces, and repeated opening and closing of gaps in the connection regions.

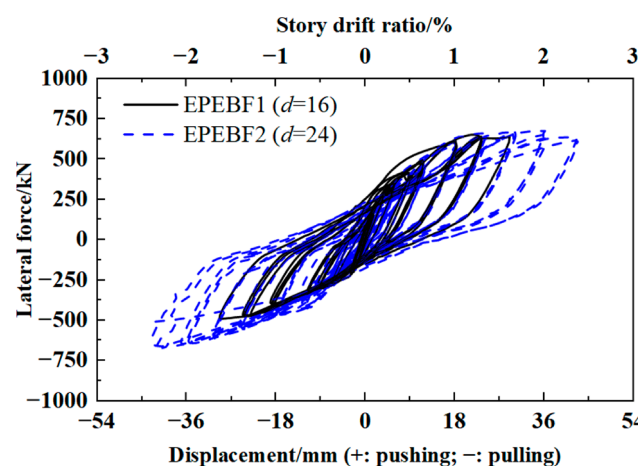


Figure 10. Comparison of hysteretic curves.

When the inter-story drift ratio reached 0.015 rad, severe damage had developed in the link and its end-plate connection region of EPEBF1, significantly limiting the specimen's capacity to sustain further shear deformation. However, no pronounced reduction in lateral

resistance was observed before the test was terminated. A comparison of the hysteretic curves of the two specimens shows that EPEBF2 with thicker end plates exhibited higher lateral resistance. However, the degree of improvement depended on the loading direction, and the thicker end plates had only a limited effect on mitigating the pinching behavior of the hysteretic response. EPEBF2 reached an inter-story drift ratio of approximately 0.025 rad at the ultimate state, and its lateral resistance did not begin to decrease until the inter-story drift ratio exceeded 0.020 rad. This indicates that increasing the end-plate thickness enhanced the restraint provided by the connection regions and improved the deformation capacity and late-stage resistance retention of the specimen. However, the observed failure modes show that a relatively large end-plate stiffness may alter the stress-transfer mechanism and plastic development path in the connection region, causing damage to extend from the link and its connection region toward the frame-beam end. Therefore, the end-plate thickness should be designed in coordination with the link length, bolt arrangement, and load-bearing capacity of the beam end to achieve an appropriate balance among connection restraint, structural deformation capacity, and protection of the primary structural members.

3.3. Skeleton Curve Analysis

The skeleton curves of the specimens are shown in Figure 11. The two specimens exhibited different resistance responses under positive and negative loading, indicating a certain degree of loading-direction dependence caused by slip in the connection regions, variations in the end-plate contact conditions, and the development of local damage. Compared with EPEBF1, EPEBF2 with thicker end plates generally exhibited a higher lateral resistance. However, the degree of improvement depended on the loading direction. The increase in resistance was more pronounced under negative loading, whereas the improvement under positive loading was relatively limited. In addition, the ultimate horizontal displacement of EPEBF2 was considerably greater than that of EPEBF1. This indicates that, for the two end-plate thicknesses investigated in the tests, the thicker end plates enhanced the restraint effect in the connection regions, resulting in higher lateral resistance and greater ultimate deformation capacity of EPEBF2.

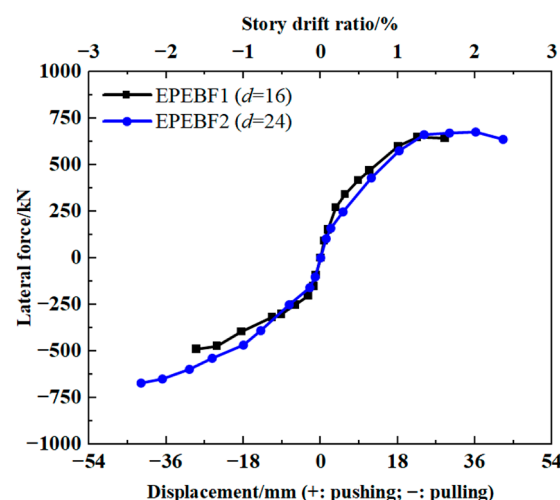


Figure 11. Comparison of skeleton curves.

3.4. Ductility Analysis

The characteristic parameters and displacement ductility factors of the specimens during loading are listed in Table 3. Each characteristic value was obtained by averaging the results under positive and negative loading. Here, F_y , Δ_y , and θ_y denote the yield load,

yield displacement, and yield inter-story drift ratio, respectively; F_m , U_m , and θ_m denote the peak load, peak displacement, and peak inter-story drift ratio, respectively; F_u , U_u , and θ_u denote the ultimate load, ultimate displacement, and ultimate inter-story drift ratio, respectively; and μ is the displacement ductility factor, calculated as $\mu = U_u / \Delta_y$. Compared with EPEBF1, EPEBF2 with thicker end plates exhibited higher yield, peak, and ultimate loads, with the peak resistance increasing by 37.0%. This result indicates that, for the two specimens tested, the stronger restraint provided by the increased end-plate thickness was beneficial to the lateral load-bearing capacity. However, the observed increase may also have been influenced by connection slip, loading-direction dependence, and differences in failure mode. Meanwhile, the horizontal displacements of EPEBF2 at the yield, peak, and ultimate states were all greater than those of EPEBF1, indicating an improvement in its absolute deformation capacity. Nevertheless, the displacement ductility factor of EPEBF2 was 29.0% lower than that of EPEBF1. This reduction occurred because the increase in yield displacement resulting from the thicker end plates was greater than the corresponding increase in ultimate displacement, thereby reducing the relative post-yield deformation reserve of the specimen. These results demonstrate that thicker end plates can improve the load-bearing and ultimate deformation capacities of the specimen, but may also alter the distribution of plastic deformation and reduce its post-yield ductility.

Table 3. Characteristic parameters and displacement ductility factors.

Specimen	F_y/kN	Δ_y/mm	$\theta_y/\%$	F_m/kN	U_m/mm	$\theta_m/\%$	F_u/kN	U_u/mm	$\theta_u/\%$	μ
EPEBF1	207.0	2.89	0.16	490.6	29.01	1.61	490.6	29.01	1.61	10.04
EPEBF2	246.0	6.04	0.34	672.3	41.91	2.33	610.7	43.09	2.39	7.13

3.5. Energy Dissipation Capacity Analysis

The area enclosed by a hysteretic loop can be used to characterize the single-cycle energy dissipation capacity of a specimen under cyclic loading. The hysteretic loop areas of the two specimens at different loading displacements are presented in Figure 12. As the loading displacement increased, the hysteretic loop areas of both specimens gradually increased, indicating that their single-cycle energy dissipation increased with deformation. At the same displacement amplitude, the hysteretic loop areas of EPEBF1 and EPEBF2 were generally similar. Although that of EPEBF2 was slightly smaller than that of EPEBF1, the difference was not significant. These results indicate that, within the range of end-plate thicknesses investigated experimentally, end-plate thickness had a relatively limited influence on the hysteretic loop area and single-cycle energy dissipation of the specimens. Although increasing the end-plate thickness enhanced the restraint provided by the connection regions, it may also have restricted the development of plastic deformation in the link to some extent. The interaction between these two effects resulted in no appreciable change in the hysteretic energy dissipation of the specimens.

3.6. Stiffness Degradation Analysis

As the loading displacement increased, the lateral stiffness of the specimens gradually decreased, indicating pronounced stiffness degradation. In this study, stiffness degradation under low-cycle reversed loading was evaluated using the loop stiffness K , calculated from the positive and negative peak loads of each hysteretic loop and their corresponding displacements. The stiffness degradation curves of the two specimens are shown in Figure 13.

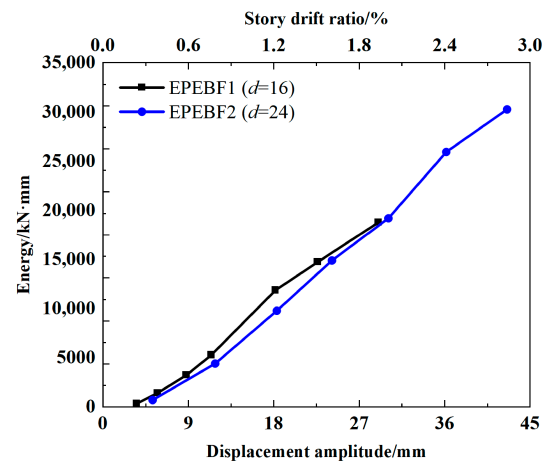


Figure 12. Comparison of energy dissipation capacity.

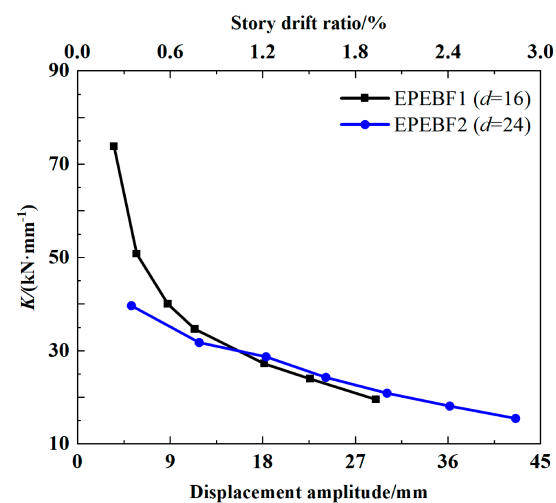


Figure 13. Comparison of stiffness degradation curves.

Based on the experimentally identified yield values in Table 3, the global secant stiffnesses F_y/Δ_y were approximately 71.6 kN/mm for EPEBF1 and 40.7 kN/mm for EPEBF2, respectively. Thus, the secant stiffness of EPEBF2 at the identified yield state was approximately 43% lower than that of EPEBF1. This difference was mainly associated with the substantially larger yield displacement of EPEBF2 relative to the increase in its yield load. It should be noted that the yield states in Table 3 were determined from strain-gauge measurements. Therefore, F_y/Δ_y represents the global secant stiffness up to the experimentally identified first-yield state rather than the loop stiffness used for stiffness-degradation evaluation. For the fully bolted specimens, the global deformation before yielding was also influenced by bolt slip, relative movement of the end-plate contact surfaces, gap opening and closing, and local connection deformation. Moreover, the increased end-plate thickness altered the stress-transfer mechanism and the development of local yielding.

After the specimens entered the plastic stage, the cyclic secant stiffness of EPEBF2 gradually became higher than that of EPEBF1, and its stiffness degradation was generally slower. This indicates that increasing the end-plate thickness enhanced the restraint effect in the connection regions and delayed the development of connection deformation and local damage, thereby improving the stiffness retention capacity of the specimen in the plastic stage. However, as indicated by the previously observed failure modes, a relatively large end-plate stiffness may also alter the distribution of plastic deformation. Therefore,

the improvement in structural stiffness should be evaluated together with the associated damage-transfer effect.

3.7. Damage Assessment

Cumulative damage induced by cyclic seismic loading is an important cause of performance degradation and eventual failure in steel structural members [37,38]. The aforementioned indicators, including hysteretic response, energy dissipation capacity, and stiffness degradation, can characterize the overall seismic performance of the specimens but cannot quantitatively describe the damage accumulation process. Therefore, three representative damage index models were selected to further investigate the damage evolution of EPEBFs. According to their primary governing variables, existing damage models can generally be classified as deformation-based, energy-based, and energy–deformation-based models. Compared with single-parameter models that consider either deformation or energy dissipation alone, energy–deformation-based models incorporate both deformation demand and cumulative hysteretic energy dissipation, thereby providing a more comprehensive description of damage development under cyclic loading. Although the Hwang and modified Park–Ang models were originally developed mainly for reinforced-concrete members, their formulations are primarily based on structural response quantities such as deformation and hysteretic energy dissipation, which are also closely associated with damage accumulation in steel structures under cyclic loading. Previous studies have also applied Hwang and Park–Ang-type damage models to steel structural connections and demonstrated their ability to characterize progressive damage under cyclic loading [39]. Accordingly, the Hwang model [26], modified Park–Ang model [27], and elastic–plastic energy dissipation ratio model [28] were adopted in this study for comparative damage assessment. Their suitability for the present EPEBF specimens was further examined by comparing the calculated damage evolution with the experimentally observed damage progression.

The damage assessment results of the two specimens obtained using the three damage models are shown in Figure 14. Overall, the two EPEBFs exhibited similar damage evolution trends. The damage indices calculated by all three models gradually increased with increasing loading displacement, reflecting the continuous accumulation of damage under cyclic loading. The end-plate thickness of EPEBF2 was 8 mm greater than that of EPEBF1. At the same loading displacement, EPEBF2 generally exhibited lower damage indices than EPEBF1, and the difference was more pronounced at small-to-moderate displacement levels. This indicates that increasing the end-plate thickness enhanced the restraint effect in the connection region and delayed early-stage damage accumulation to some extent. However, the previously observed failure modes showed that a relatively large end-plate stiffness altered the stress-transfer mechanism and plastic development path at the later loading stage, causing damage to shift from the link and its end-plate connection region toward the frame-beam end. Therefore, a lower overall damage index does not necessarily imply a corresponding reduction in the damage risk of the primary structural members. The influence of end-plate thickness on damage control should be evaluated comprehensively by considering both the damage location and the failure mode.

A comparison of the assessment results obtained using the three damage models reveals clear differences in their damage evolution trends, index ranges, and discrimination capabilities. Overall, all three models captured the progressive accumulation of damage with increasing loading displacement, indicating that these models are feasible for the comparative assessment of damage evolution in the EPEBF specimens investigated in this study. However, their suitability differs in terms of the index range, progressive characteristics, and ability to distinguish between specimens. The damage curves calculated using the Hwang model exhibit a nonlinear accelerating trend and clearly distinguish the

damage levels of the two specimens. However, the damage indices obtained from this model cannot be consistently bounded within the range of 0–1. At the early loading stage, the model markedly underestimates the damage level of the specimens, whereas the damage index increases rapidly at the later loading stage. This behavior deviates to some extent from the progressive damage accumulation observed during the tests. The damage curves obtained using the modified Park–Ang model increase approximately linearly, and the damage indices of the two specimens are relatively close at the final loading stage. Consequently, this model cannot adequately capture the differences in damage evolution induced by variations in end-plate thickness and therefore exhibits limited discrimination capability. In contrast, the damage curves calculated using the elastic–plastic energy dissipation ratio model show a progressive nonlinear increase with loading displacement, while the corresponding damage indices remain consistently within the range of 0–1. Moreover, the damage evolution curves of the two specimens exhibit clear differences, enabling the influence of end-plate thickness on the damage accumulation process to be more effectively identified. Therefore, all three models provide a feasible representation of the overall damage evolution of the present steel specimens. Among them, the elastic–plastic energy dissipation ratio model exhibits a more reasonable index range and more consistent progressive characteristics in describing the damage evolution of the EPEBFs investigated in this study.

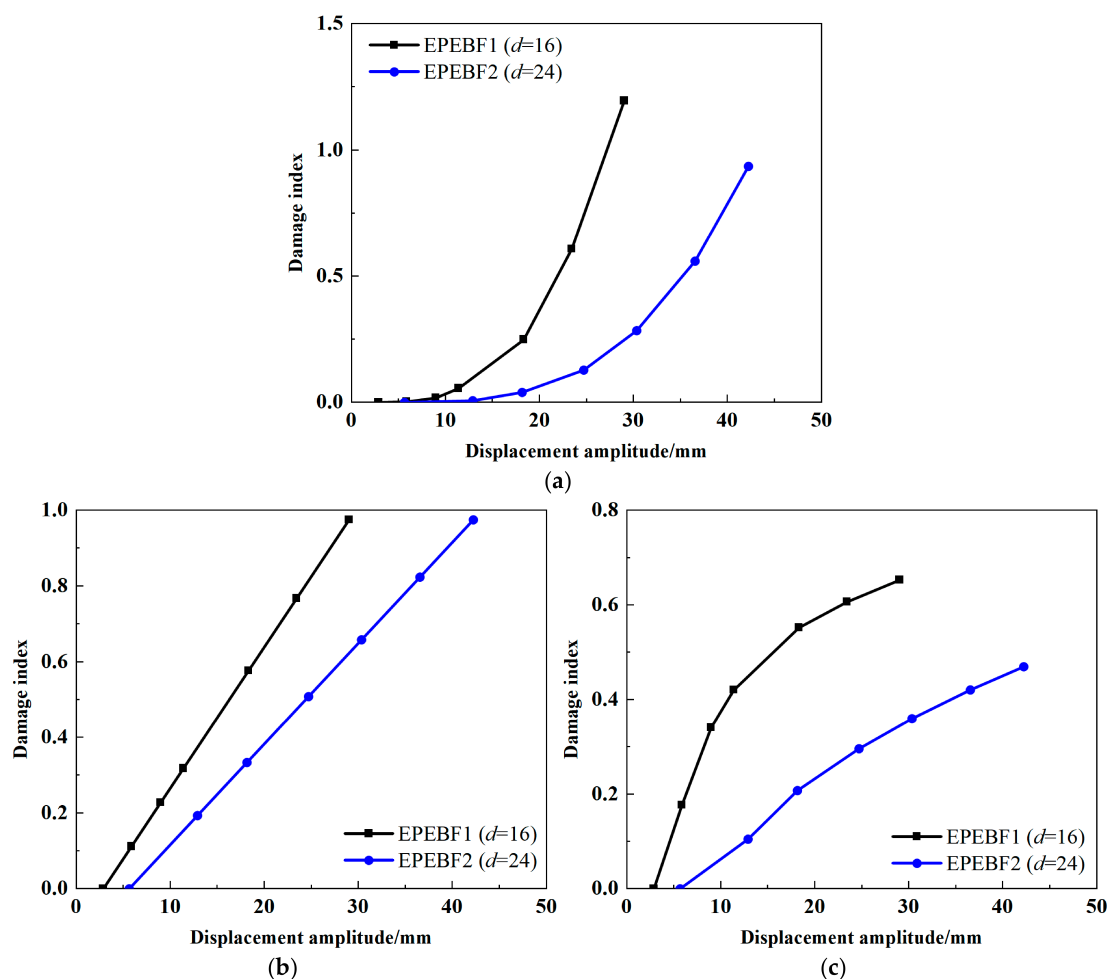


Figure 14. Comparison of damage index curves. (a) Hwang model. (b) Modified Park–Ang model. (c) Elastic–plastic energy dissipation ratio model.

4. Finite Element Simulation of Test Specimens

4.1. Establishment and Validation of the Finite Element Model

Owing to the limited number of test specimens, the influence of end-plate thickness on the seismic performance of K-shaped eccentrically braced frames with end-plate connections could not be systematically investigated based on the experimental results alone. Therefore, refined finite element models of the test specimens were developed using Abaqus 6.14, and extended parametric analyses were subsequently conducted after model validation. The finite element model configuration and boundary conditions are shown in Figure 15.

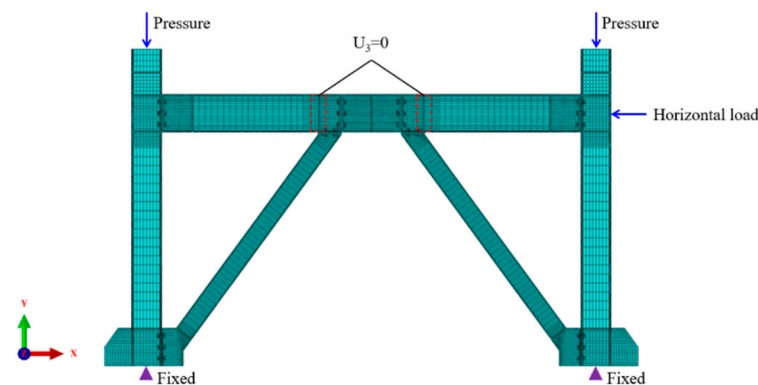


Figure 15. Finite element model.

The nonlinear constitutive behavior of the steel was defined based on the material test results. The engineering stress–strain relationships obtained from the tensile coupon tests were converted into true stress–strain relationships and introduced into Abaqus following the approach adopted in previous studies [39]. All structural components were modeled using C3D8R solid elements, with mesh sizes ranging from 10 to 20 mm. Local mesh refinement was adopted in the joint regions. Q235B steel was used for the links, whereas Q345B steel was adopted for the frame columns, frame beams, and braces. According to the actual contact conditions of the test specimens, the tangential behavior of the contact interfaces between different members was simulated using frictional contact with a coefficient of friction of 0.3, while hard contact was adopted in the normal direction. The material properties of the high-strength bolts were defined according to the product specifications. The elastic modulus, yield strength, tensile strength, Poisson's ratio, and elongation were 210 GPa, 950 N/mm², 1060 N/mm², 0.3, and 20%, respectively.

A pretension force of 155 kN was applied to each high-strength bolt using the Bolt Load function in Abaqus, consistent with the experimental setup. The welded connections between the end plates or stiffeners and the primary structural members were modeled using shared nodes at the corresponding interfaces, while the influence of weld fillet dimensions was neglected. A reference point was established at the center of the loading connection plate, through which the cyclic lateral displacement was applied. The plastic behavior of the steel was described using a kinematic hardening model. The finite element analyses adopted the same displacement sequence and displacement amplitudes as those used in the tests, while one complete reversed cycle was applied at each displacement level to improve computational efficiency. Vertical loads were applied at the tops of the two columns, fixed boundary conditions were assigned to the column bases, and out-of-plane restraints were imposed on both sides of the frame beam to reproduce the column-base fixity and lateral restraint conditions adopted in the tests. All numerical analyses were performed using Abaqus/Standard. The numerical ultimate state was identified when severe local failure and plastic deformation developed in the structural members and the

analysis could no longer converge. The corresponding lateral displacement was defined as the numerical ultimate displacement.

Figure 16 compares the hysteretic curves obtained from the finite element simulations with the experimental results, whereas Figure 17 compares the experimentally observed damage with the simulated stress (s , Mises) and equivalent plastic strain (PEEQ) distributions. During the shear deformation of the specimens, pronounced stress concentrations developed in the high-strength bolts at the ends of the links. However, bolt stresses cannot directly characterize the yielding and plastic development of the links. Therefore, the bolts are omitted in Figure 17 to more clearly illustrate the stress and plastic strain distributions in the links and their connection regions. The PEEQ distributions in Figure 17 further show that plastic deformation in EPEBF1 was mainly concentrated in the link-end connection regions, whereas that in EPEBF2 extended toward the adjacent frame-beam ends. These plastic strain concentration regions correspond well to the principal damage locations observed experimentally, providing additional support for the ability of the finite element models to reproduce the main plastic deformation patterns of the specimens. The comparison shows that the simulated hysteretic response and failure mode of EPEBF1 agree well with the experimental results. For EPEBF2, the increased end-plate thickness altered the stress-transfer mechanism in the connection region, causing plastic damage to extend from the link and its end-plate connection region toward the frame-beam end. This change further affected the bolt force distribution and slip behavior at the connection interfaces. Consequently, a certain discrepancy in the degree of pinching was observed between the simulated and experimental hysteretic curves, indicating some limitations in reproducing the pinching behavior of EPEBF2. Nevertheless, the relative errors in peak lateral resistance were less than 12.1%, and the simulated damage locations and failure modes agreed reasonably well with the experimental observations. Therefore, the finite element models can reasonably reproduce the main mechanical behavior and failure characteristics of EPEBFs and can be used for the subsequent comparative parametric analysis of end-plate thickness, while the numerical trends related to pinching are interpreted with caution.

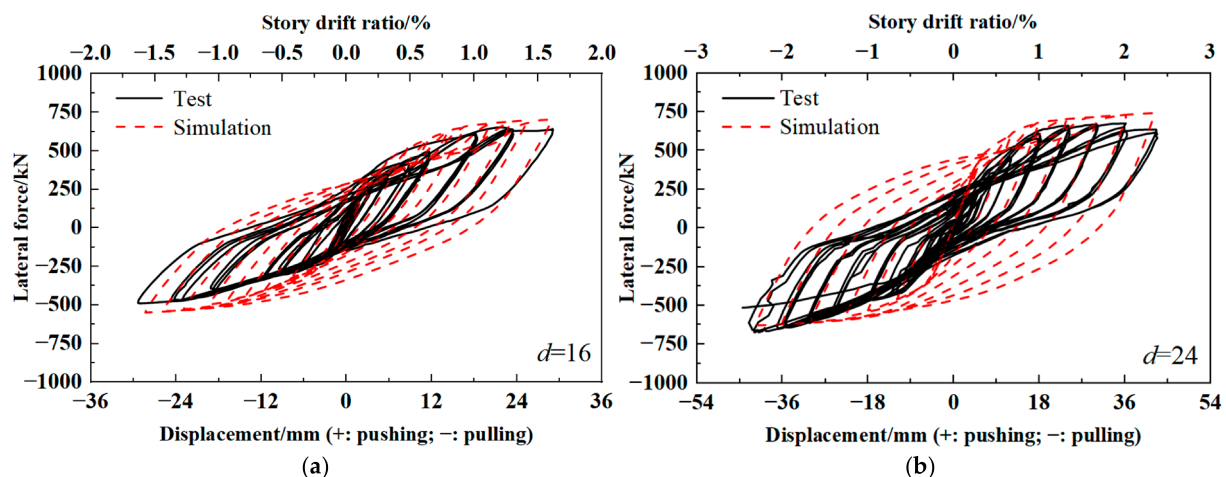


Figure 16. Comparison of experimental and simulated hysteretic curves. (a) EPEBF1. (b) EPEBF2.

To further clarify the influence of end-plate thickness on the seismic performance of EPEBFs, two additional parametric models, EPEBF3 and EPEBF4, with end-plate thicknesses of 12 mm and 20 mm, respectively, were developed based on the validated finite element model. Except for the end-plate thickness, the member dimensions, material properties, bolt arrangements, contact interactions, boundary conditions, and loading protocol of the two extended models were kept identical to those of the finite element models of the test

specimens to ensure the comparability of the numerical results. The configurations of the link connection regions in the parametric models are shown in Figure 18.

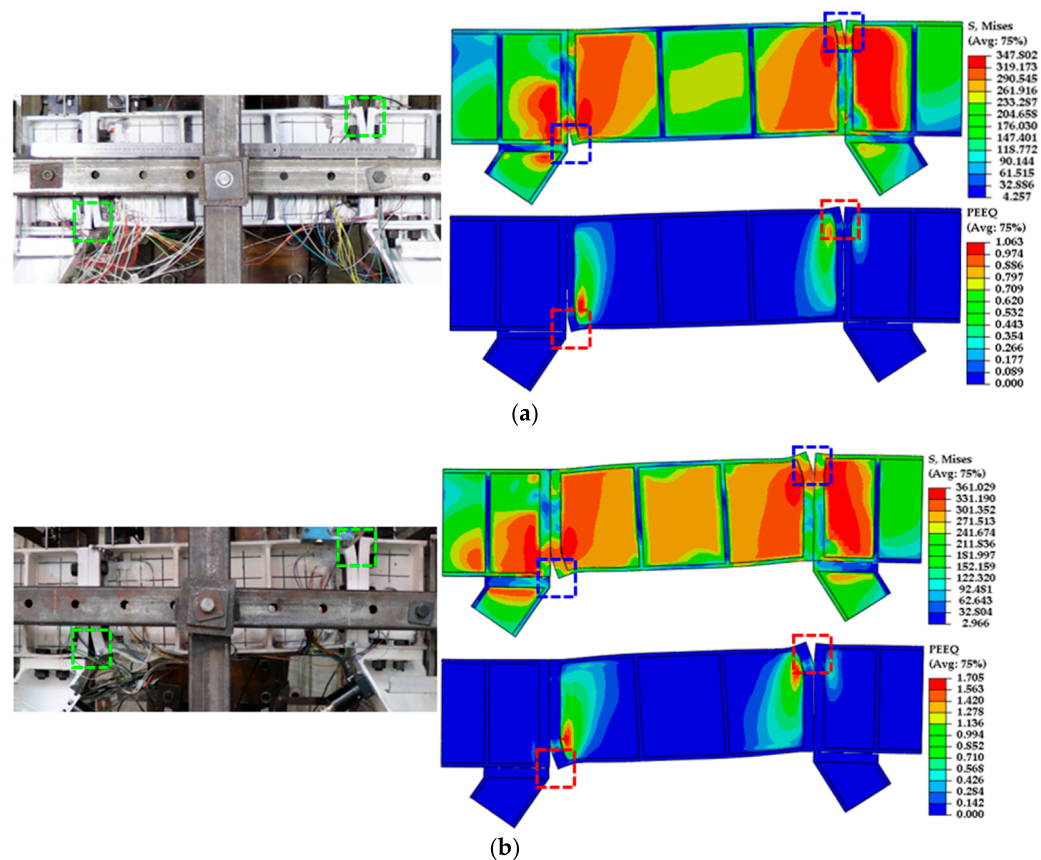


Figure 17. Comparison of experimental damage, von Mises stress, and equivalent plastic strain distributions. (a) EPEBF1. (b) EPEBF2.

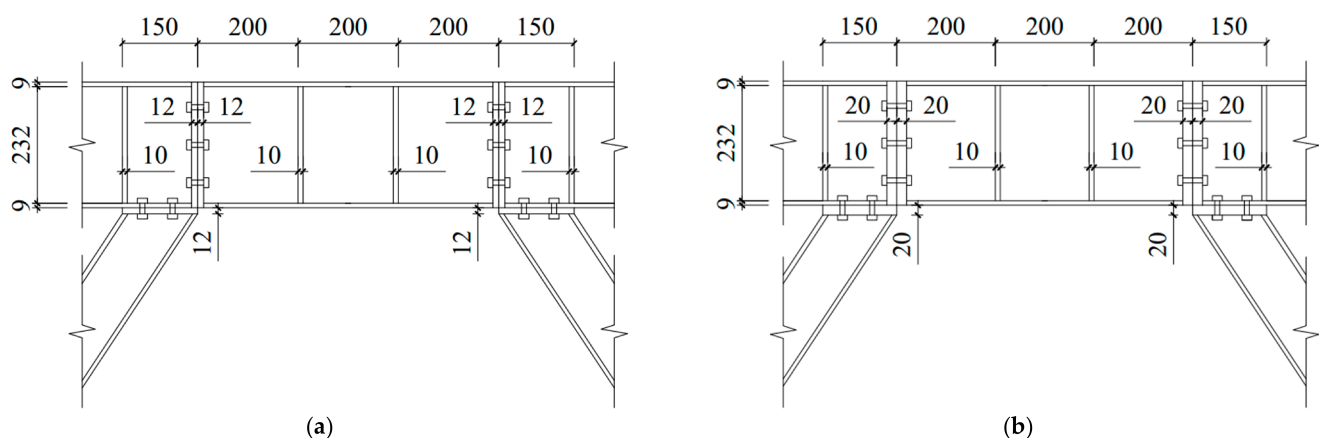


Figure 18. Dimensions of the link connection regions in the finite element models (unit: mm). (a) EPEBF3. (b) EPEBF4.

4.2. Hysteretic Curve Analysis

The hysteretic and skeleton curves of all finite element models are shown in Figures 19 and 20, respectively. The hysteretic curves of the four models exhibit varying degrees of pinching, indicating that slip at the connection interfaces, repeated gap opening and closing, and local plastic deformation significantly affect the cyclic response of the structure. A comparison of the models in ascending order of end-plate thickness,

namely EPEBF3, EPEBF1, EPEBF4, and EPEBF2, shows that their skeleton curves were generally similar in the elastic stage and near the peak resistance. Moreover, the peak lateral resistance did not exhibit an evident monotonic increase with increasing end-plate thickness. These results indicate that, within the numerical models and the end-plate thickness range of 12–24 mm considered in this study, end-plate thickness had a relatively limited overall influence on the lateral resistance of the structure.

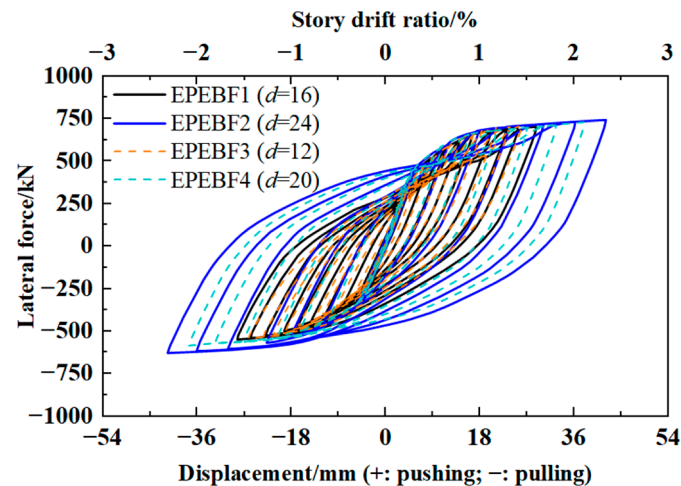


Figure 19. Comparison of hysteretic curves.

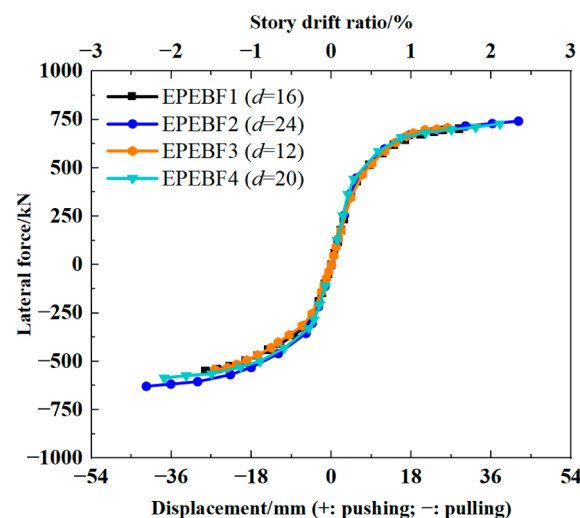


Figure 20. Comparison of skeleton curves.

The higher peak resistance observed for EPEBF2 than for EPEBF1 in the tests may also have been affected by interface slip at the connections, loading-direction dependence, and changes in the final failure mode. Therefore, the difference in resistance between the two specimens should not be attributed solely to end-plate thickness. In contrast, end-plate thickness had a more pronounced influence on the deformation capacity of the structure. As the end-plate thickness increased, the ultimate displacement of the models generally increased, and their late-stage resistance retention improved. The numerical hysteretic curves exhibited some differences in loop fullness with increasing end-plate thickness. Considering the limitations of the finite element model in reproducing local cyclic connection behavior, these differences are interpreted qualitatively. More consistently, the ultimate displacement and late-stage resistance retention generally increased with end-plate thickness. Overall, within the investigated range, increasing the end-plate thickness

mainly improved the ultimate deformation capacity and late-stage resistance retention, whereas its systematic effect on peak lateral resistance remained limited.

4.3. Energy Dissipation Capacity Analysis

The hysteretic loop areas at each loading displacement were used to evaluate the single-cycle energy dissipation capacity of the models with different end-plate thicknesses, as shown in Figure 21. As the loading displacement increased, the hysteretic loop areas of all models exhibited a nonlinear increasing trend, indicating that the single-cycle energy dissipation gradually increased with deformation. At the same displacement amplitude, the hysteretic loop areas of the different models were generally similar and did not show a clear monotonic variation with increasing end-plate thickness. This indicates that, within the parameter range investigated in this study, end-plate thickness had a relatively limited influence on the hysteretic energy dissipation at a given displacement level. Meanwhile, the ultimate displacement of the models generally increased with increasing end-plate thickness, allowing the models with thicker end plates to undergo larger deformations and develop greater hysteretic loop areas during the later loading stages. Therefore, increasing the end-plate thickness did not significantly alter the single-cycle energy dissipation at the same displacement level. Its primary effect was to improve the ultimate deformation capacity of the structure, thereby prolonging the energy dissipation process and increasing the amount of energy that could be dissipated before the model reached its ultimate state.

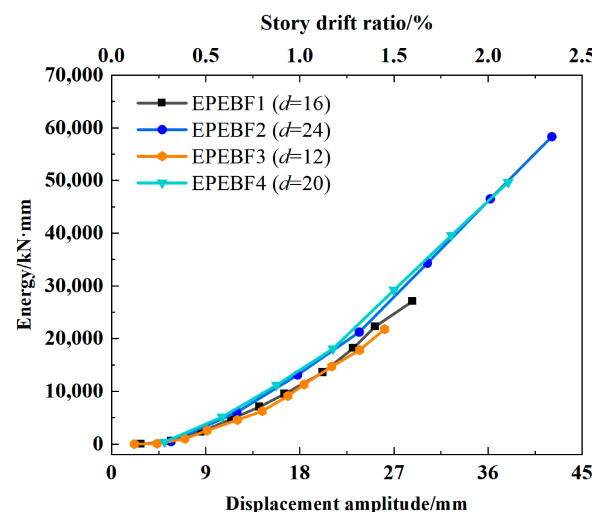


Figure 21. Comparison of energy dissipation capacity.

4.4. Stiffness Degradation Analysis

The stiffness degradation curves of all finite element models are shown in Figure 22. As the loading displacement increased, the cyclic secant stiffness of all four models continuously decreased. The stiffness degraded rapidly during the initial loading stage and then gradually approached a relatively stable rate, indicating generally similar stiffness degradation characteristics among the models. A comparison of the models with different end-plate thicknesses shows that EPEBF4 and EPEBF2, which had thicker end plates, generally exhibited slightly higher stiffness during the initial loading stage than the models with thinner end plates; however, the differences among the models were relatively small. After the models entered the plastic stage, the stiffness degradation curves gradually converged, and both the cyclic secant stiffness and its degradation trend at the same displacement level were generally consistent. This indicates that end-plate thickness has only a limited influence on the overall structural stiffness and its degradation rate. It should be noted that the stiffness degradation curves of the models with thicker end plates extended to

larger displacement levels. This was mainly attributed to the enhanced ultimate deformation capacity resulting from increased restraint in the connection regions, rather than to a significant improvement in stiffness retention. Overall, increasing the end-plate thickness primarily improves the structural deformation capacity, while its influence on the degradation of the overall lateral stiffness remains limited.

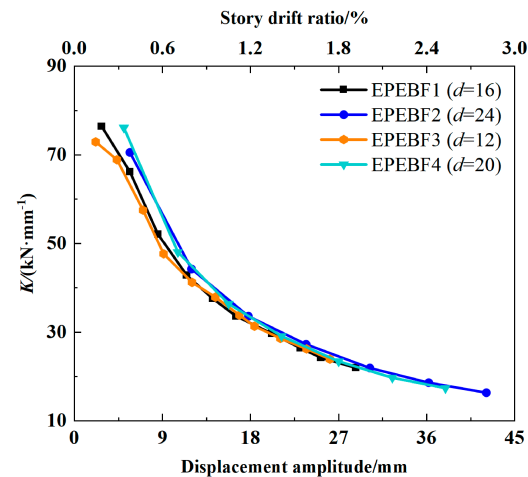


Figure 22. Comparison of stiffness degradation curves.

5. Conclusions

Low-cycle reversed loading tests, damage assessment, and finite element parametric analyses were conducted on K-shaped eccentrically braced frames with end-plate connections. The study focused on the effects of end-plate thickness on the failure modes, hysteretic performance, load-bearing and deformation capacities, energy dissipation characteristics, stiffness degradation, and damage evolution of the frames. The experimental and finite element results were reasonably consistent in terms of peak lateral resistance and principal failure characteristics, providing a basis for the comparative parametric analysis of end-plate thickness within the investigated range. The following conclusions can be drawn:

(1) EPEBFs exhibited a well-defined plastic development path. During cyclic loading, high-strength bolt slip, relative movement between the end-plate contact surfaces, and repeated opening and closing of the connection gaps caused pinching of the hysteretic response. Damage in the reference specimen was mainly concentrated in the link and its end-plate connection region, thereby effectively protecting the primary structural members, including the frame columns, frame beams, and braces. Increasing the end-plate thickness enhanced the restraint provided by the connection region. However, it also altered the stress-transfer mechanism and plastic development path, causing damage to extend toward the frame-beam end.

(2) For the two specimens tested, the specimen with thicker end plates exhibited higher lateral resistance, greater ultimate deformation capacity, and improved late-stage resistance retention. However, its displacement ductility factor was reduced. This indicates that the increase in yield displacement caused by the thicker end plates exceeded the corresponding increase in ultimate displacement, thereby reducing the relative post-yield deformation reserve of the specimen. Therefore, although increasing the end-plate thickness can enhance connection restraint and improve certain aspects of seismic performance, it may also alter the distribution of plastic deformation and increase the risk of damage to the primary structural members.

(3) All three damage models captured the general trend of progressive damage accumulation with increasing loading displacement, but differed in their index ranges and evolution characteristics. Among the three models, the elastic–plastic energy dissipation ratio model exhibited a more reasonable index range and more consistent progressive characteristics. These characteristics provided a clearer comparative description of the damage evolution of the specimens investigated in this study.

(4) The finite element parametric analysis showed that the peak lateral resistance did not increase monotonically with increasing end-plate thickness within the investigated range. This indicates that end-plate thickness had a relatively limited systematic effect on the lateral resistance of EPEBFs. In contrast, increasing the end-plate thickness generally improved the ultimate deformation capacity and late-stage resistance retention. Therefore, its primary influence was on cyclic deformation performance rather than on lateral resistance. End-plate thickness should be designed in coordination with the link length, bolt layout, and load-bearing capacity of the frame-beam ends.

(5) The present study is subject to limitations associated with the small experimental matrix and the scope of the numerical validation. Accordingly, the findings should primarily be interpreted within the investigated parameter range. The conclusions are most applicable to the K-shaped EPEBFs with shear links considered in this study, with end-plate thicknesses ranging from 12 to 24 mm. Further experimental validation over a broader range of structural parameters is required to establish their wider applicability.

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Data Availability Statement: The testing and analysis data used to support the findings in this study are included within the article.

Conflicts of Interest: Author Gaofei Huang was employed by the company Luoyang Gaofei Bridges and Tunnels Machinery Co., Ltd. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

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