Finite Element Parametric Analysis of High-Strength Eccentrically Braced Steel Frame with Variable-Cross-Section Replaceable Link

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Abstract: This paper proposes a new type of variable-cross-section replaceable link that can isolate the plastic deformation. The link and the frame beam connect with the expanding section by a bolt along the flange and web separately. It is easy to design an elastic bolt, reduce bolt slippage, and facilitate link replacement after an earthquake. Considering the large elastic deformation range of high-strength steel and the superior plastic deformation of the new type of replaceable link, a high-strength eccentrically braced steel frame with a variable-section replaceable link is raised by setting a new type of variable-section replaceable link in the eccentrically braced steel frame. Then, the existing end plate connection replaceable link test specimen is simulated and verified by using ABAQUS software. The finite element model of the high-strength eccentrically braced steel frame with a variable-cross-section replaceable link is established, and the bearing capacity, stiffness, plastic rotation, plastic distribution, and other bearing mechanisms of the structure are studied by cyclic loading. The length of the energy-consuming region ($e$), the steel strength of the link and other components, and the length of the replaceable link ($e'$) are compared and analyzed with regard to the seismic performance of the structure. The results are of great significance for understanding and exploring the force mechanism, energy dissipation characteristics of the new variable-section replaceable link, and the seismic performance of the high-strength eccentrically braced steel frame, and it also provides a reference for subsequent research.

Keywords: replaceable link; eccentrically braced frame; high-strength steel; parameter analysis; seismic performance

1. Introduction

Structures have always been designed to meet the collapse prevention requirements under rare earthquakes; however, due to excessive local or overall deformation, the majority of existing structures are no longer suitable for further use and need to be rebuilt at a lower cost compared to the repair, and the shape memory alloy has been applied to structures for self-centering, which is a frontier issue for further research [1,2]. The links in traditional eccentrically braced steel frames performing as a fuse are initially damaged, which may affect the overall stability of the structure, causing the fracture of the connected beam [3–5]; furthermore, the links could suffer more complicated repairs even when the beam remains intact.

To solve the complicated repair problem of the link, increasingly more research scholars have proposed replaceable designs rather than additional collapse-resistant designs in recent years. Dubina et al. [6,7] proposed the application of high-strength steel in seismic-resistant structures. Lian and Su [8–13] proposed an end-plate connection and bolted web-connected replaceable link, and they used them in the high-strength-steel framed-tube structures; furthermore, the cyclic behavior and seismic performance of the replaceable link were analyzed by experiment and finite element analysis with the design method. Moreover, Lian [14–17] projected the vertical link in the eccentrically braced steel frames...
and researched the seismic performance by experiment and finite element analysis, and they proposed replaceable links in eccentrically braced steel frames after earthquakes. In addition, the replaceable links can be easily replaced or repaired by fixing the bolt along the end-plate or web, because the vertical link in the structures is not connected to the floor and isolated to the beam. In this paper, a new type of variable-cross-section replaceable link that can be quickly repaired is proposed in a high-strength eccentrically braced steel frame.

In order to ensure the elastic design of the non-energy-dissipation components, during the design process of the eccentrically braced frame, the beams, columns, and braces are designed with an enlarged elastic internal force, which refers to the internal force of the beam, column, and brace when the link reaches the plastic shear bearing capacity. The plastic deformation is naturally isolated in the links, while the rest of the structure remains elastic, thus ensuring that the seismic energy is concentrated on the replaceable link. The frame beams and columns made of high-strength steel ($f_y \geq 460$ MPa) are shown in Figure 1. The sufficient elastic deformation capacity of high-strength steel makes the non-energy-dissipating members of the structure capable of recovering elastic deformation after an earthquake. The cross-section of the frame beams and columns will be large when using ordinary steel; however, the application of high-strength steel will improve the behavior and lower the self-weight, saving steel consumption while reducing the seismic effect and the cost of the foundation. Meanwhile, the residual deformation of the structure is minimal due to the large axial stiffness of the braces. The structure proposed in this paper integrates the advantages of the wider-ranging elastic deformation of high-strength steel and the greater lateral stiffness of the eccentric braces, as well as the better energy dissipation capacity of the new replaceable link.

![Replaceable link with variable cross-section](image)

**Figure 1.** Replaceable link with variable cross-section.

The cross-section of the variable replaceable link can be optimized and designed without considering the section of the frame beam, and it would achieve better economic benefits. The new variable-cross-section replaceable link consists of three parts: an energy-consuming region, bolts connection parts, and variable section. In addition, the replaceable link is characterized by controlled plastic deformation as it concentrates energy dissipation through the intermediate section under earthquakes, and the replaceable link and the frame beam are connected by flange and web bolts to prevent the impact of bolt slippage on the replaceable link.

Based on the previous experimental studies, this paper investigates the seismic performance of the structure considering three parameter variations, and the bearing mechanism and energy dissipation capacity are discussed.

2. **Finite Element Test Verification**

2.1. **Test of Replaceable Link**

2.1.1. **Test Overview**

To validate the accuracy of the simulations of the replaceable link, the finite element model of the end-plate connection replaceable link tests in Ref. [18] was simulated. The specimen was designed as a 1/2 scale with a height of 0.7 m, a span of 1.5 m, and a length of 0.3 m for the replaceable link. An end plate with a pin hinge that could allow the structure...
to rotate freely in the plane was provided at the column base. The beam-to-column joints had a full-welding connection, and the replaceable link was welded with the stiffening ribs and the end plate. Moreover, the replaceable link was connected with the frame beam by bolts through two end plates. In order to make the design of specimen more rational, the flanges of the frame beams were deliberately weakened into circular sections at the beam ends. Meanwhile, square stiffeners were provided at the top of the beam ends, and triangular stiffening ribs were provided at the flanges of the frame columns, at the joints between the end plates, and at the frame beams and the bottom of the frame columns. The equispaced stiffening ribs were uniformly arranged on the web of the replaceable link, as shown in Figure 2.

![Steel moment resisting frame with replaceable link.](image)

**Figure 2.** Steel moment resisting frame with replaceable link.

Welded H-section steel was applied in all of the specimens, and the cross-sections of the frame beam, frame column, and replaceable link were H300 × 150 × 6 × 8, H280 × 280 × 12 × 20, and H170 × 120 × 5 × 12, respectively, where the yield strength and tensile strength of the beam and column were 374 Mpa and 481 Mpa, respectively, and the yield strength and tensile strength of the replaceable link were 301 Mpa and 403 Mpa, respectively.

In order to better apply a cyclic loading, bilateral symmetry restraints and actuators were provided on the specimen. Furthermore, auxiliary restraint devices were also provided in the vertical direction to prevent out-of-plane displacement of the specimen. The loading setup for the specimens is shown in Figure 3. The loading protocol corresponded with the AISC [19] with a total of 6 cycles of 0.00375 rad, 0.005 rad, and 0.0075 rad, followed by 4 cycles of 0.01 rad, and finally 2 cycles of 0.015 rad, 0.02 rad, and 0.03 rad until failure.

![Loading setup.](image)

**Figure 3.** Loading setup.

2.1.2. Test Model

The finite element model (FEM) was established using ABAQUS software 6.12 version, and the C3D8R solid element was selected for the model. A denser mesh size (5 mm) was divided for the replaceable link that causes the concentration of plastic deformation, while the mesh size of the remaining components could be enlarged to 10 mm to save calculation speed. The parameters were calibrated to be consistent with the tests. The finite element model is shown in Figure 4. The tie constraint was defined between the model frame beams, columns, and stiffening ribs. The degrees of freedom for the column base plate
were coupled at one point to simulate the hinged connection of the test. The bolt preload was imposed by the bolt load, and the interaction properties between the end plates and bolt head and bolt rod were defined separately, choosing a hard contact for normal contact and Coulomb friction for tangential contact. Moreover, the friction coefficient was defined as 0.35, which was based on the slip resistance coefficient specified in the Steel Design Standard (GB50017-2017) for friction-type high-strength bolts, which is taken to be 0.35. The tie constraints were applied to the frame beam and column in the U2, UR1, and UR3 directions, and the displacement conditions were applied simultaneously at both ends of the frame column using coupling. The boundary conditions of the model are shown in Figure 5.

![Mesh refinement](image)

**Figure 4.** Model meshing.

![Pinned column base](image)

**Figure 5.** Model boundary condition.

2.1.3. FEM Results

Figure 6 shows the plastic strain distribution of the specimen from the FEM analysis, which is compared with the test failure mode.

As can be seen in Figure 6, the replaceable link showed a large shear deformation, the frame beam ends buckled, and tiny deformation occurred out-of-plane due to the restraint. The equivalent plastic strain from the FEM showed that the replaceable link developed full plasticity at the same time, where the beam ends started to develop plasticity, the upper and lower flanges of the beam presented local buckling, and the failure mode of the structure obtained from the FEM was consistent with the test. It is worth noting that the end plates were subjected to relatively large moments and shear forces during the tests, and this led to the deformation of the bolt rod extrusion hole walls; even more, the bending of bolt holes and some of the bolt rods would probably bring more difficulties for the post-earthquake repair of the replaceable link.
The hysteresis curves from the FEM were compared with the test, as shown in Figure 7; the test and FEM hysteresis curves were generally the same; however, the FEM hysteresis curve was fuller in shape, while the test hysteresis curve descended more obviously. This was mainly due to the tie-bound connections of welding in the FEM model being unable to consider the initial defects in the test. The ultimate loading obtained from the test was 513.25 kN and the ultimate loading obtained from the FEM was 524.76 kN, an error of only 2.2%.

A comparison of the backbone curves between the test and the FEM is shown in Figure 8. The FEM results were in good agreement with the test results, and both could well reflect the elastic, elastic-to-plastic, and plastic phases of the structure, with an initial stiffness of 80.32 kN/mm from the test and 82.65 kN/mm from the simulation generally being equal with an error of only 2.8%.

Overall, the steel frame structure with replaceable links showing better energy dissipation and seismic performance can be quickly recovered or repaired using end plates and bolts. The finite element model established in this paper provides an accurate representation of the structural forces and deformation characteristics. However, with the development of the plastic deformation of the replaceable link, some of the bolt holes and bolts also experienced residual deformation, which would affect the replacement of the links after the earthquake.
2.2. K-Typed Eccentrically Braced Steel Frame Test

2.2.1. Overview

In this subsection, the finite element simulations are performed for the tests in reference [20], and the specimen is loaded by monotonic pushover. The loading setup and the dimensions of the specimen are shown in Figure 9. The lateral displacement of the frame column was imposed in a good distribution by setting the loading beam. The members were made of welded H-beams, the link was made of Q345B steel, and the rest of the members were made of Q460C steel. The link and frame beam were H225 × 125 × 6 × 10, the frame column was H150 × 150 × 6 × 10, and the brace was H125 × 120 × 6 × 10. The beam-column joints were rigidly connected and the link was butt-welded to the frame beam. The material properties tests are given in Table 1, where \( t \) represents the thickness of the steel plate, \( f_y \) is the measured yield strength, \( f_u \) is the measured ultimate strength, and \( E \) is the elasticity modulus. The test started with a column top axial force of 800 kN applied by an oil jack and the vertical load was kept constant during the test. The horizontal load of the test was applied by displacement control.

![Loading setup](image)

**Figure 9.** Loading setup.
Table 1. Material test results.

| Steel Grade | t/mm | $f_y$/MPa | $f_u$/MPa | $E/10^5$/MPa | Elongation/% |
|-------------|------|-----------|-----------|---------------|-------------|
| Q345B       | 6    | 427.40    | 571.10    | 2.01          | 26.53       |
| Q345B       | 10   | 383.33    | 554.40    | 2.00          | 31.01       |
| Q460C       | 6    | 496.90    | 658.57    | 2.08          | 29.73       |
| Q460C       | 10   | 468.77    | 627.97    | 2.02          | 35.88       |

2.2.2. Experimental Model

The structural model of the single-story, single-span, K-typed eccentrically braced steel frame was created using the ABAQUS software, and the C3D8R elements were used to model each member of the structure. The steel principle of the structure was modeled using a double-folded line follower hardening model. By the processing conditions of the specimens, the frame beams and columns, the beams and links, the braces and frame beams, the braces and frame columns, and the stiffening ribs were all connected using welding, and no consideration was given to the effects of cracking and relative slip between the members during the simulation. Therefore, the connections were made using the binding constraint (tie) in the interaction module. The definition of tie binding in ABAQUS software means the strong connection between components, and it is equivalent to the welding connection or fixed connection. At the same time, the degrees of freedom in the X, Y, and Z directions at the column base were constrained, the degrees of freedom in the X direction at the flange of the beam was constrained, and vertical loads were coupled at the top of the column and lateral forces were coupled at the left and right flange of the frame column. The model boundary conditions are shown in Figure 10.

![Figure 10. Boundary conditions of specimen.](image)

2.2.3. FEM Results

The final damage of the test and the plastic strain distribution for the FEM is shown in Figure 11, and the test resulted in large plastic deformation of the link, with severe buckling and deformation of the web in the first zone of the link. Tearing damage occurred in the web at the connection between the link and the frame beam, with vertical relative displacement occurring on the left and right sides of the structure. Compared with the FEM results, the link performed sufficiently plastically at this time, the web buckled, and the plastic development in the same zone was more apparent. The hysteresis curves obtained from the structural tests were compared with the simulated hysteresis curves, as shown in Figure 12.

The hysteresis curve obtained from both the test and the FEM was shuttle-shaped but full, which indicates that the structure had a stable energy dissipation capacity, and the link gave full play to its energy dissipation capacity and consumed seismic energy through plastic deformation. The ultimate loading of the structure acquired from the test was 736.83 kN and the FEM was 791.21 kN, which was slightly higher than the test value.
A single-story, single-span, high-strength eccentrically braced steel frame structure with a variable-section replaceable link was selected from the prototype structure for finite element analysis. The finite element analysis was able to simulate the energy dissipation capacity and seismic performance of the structure well. In general, the K-typed eccentrically braced steel frame had an excellent energy dissipation capacity and an adequate elastic reserve.

3. Base Model Design

The backbone curves of the test and FEM are shown in Figure 13. The simulated backbone curve was always slightly higher than the experimental backbone curve, and the initial stiffness of the structure was 76.13 kN/mm in the test and 83.26 kN/mm in the FEM, an error of less than 10%. In the FEM model, the webs of the links entered plasticity, and the plastic concentration development area was consistent with the test tearing area. The finite element analysis was able to simulate the energy dissipation capacity and seismic energy of the structure well. In general, the K-typed eccentrically braced steel frame had an excellent energy dissipation capacity and an adequate elastic reserve.

Figure 11. Comparison of failure modes.

Figure 12. Comparison of hysteretic curves.

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Figure 13. Comparison of backbone curves.
element simulation, with a frame beam length of 2480 mm, a frame column height of 3600 mm, and a replaceable link length of 1840 mm. The deformations of the replaceable link, connection plate, and bolts were accurately modeled. The FEM models were created using the tension function in ABAQUS software. The cross-section dimensions and lengths of the components are shown in Table 2. The member sections were built-up sections, and H-sections were used for the members, where “H” refers to the welded H-shaped section, and the following numbers are section depth, flange width, web thickness, and flange thickness. A diagram of the replaceable link is shown in Figure 14. The length ratio $\rho$ is determined by the plastic shear capacity $V_p$ and the plastic bending capacity $M_p$ of the link, which is calculated by Equations (1)–(3).

$$V_p = 0.58 f_y A_w$$  \hspace{1cm} (1)

$$M_p = f_y W_y$$  \hspace{1cm} (2)

$$\rho = e / \left( M_p / V_p \right)$$  \hspace{1cm} (3)

where $f_y$ is the yield strength of the replaceable link; $A_w$ is the cross-sectional area of the web in the link; $W_y$ is the plastic modulus of the full section in the link; $e$ is the length of the energy dissipation region.

Table 2. Dimensions of elements.

| Specimen             | Cross-Section | Length/(mm) | H-Shaped Section |
|----------------------|---------------|-------------|------------------|
| Beam                 | H620 × 180 × 10 × 16 | 2480        |                  |
| Column               | H400 × 400 × 12 × 12 | 3600        |                  |
| Brace                | H260 × 180 × 10 × 16 | 4275        |                  |
| Energy consuming region link | H500 × 180 × 10 × 16 | 840 ($e'$) |                  |
|                      | H620 × 180 × 10 × 16 | 1840 ($e'$) |                  |

![Figure 14. Section size of the link.](image)

The length ratio of the replaceable link is calculated according to the length of the energy-consuming region $e$. The length ratio $\rho = 1.17 < 1.6$ calculated according to Equations (1)–(3) indicates that the designed replaceable link of the variable section is
of shear yielding type. From the section size and section form of the link, the end moments and shear forces of the link are calculated according to Equations (4) and (5) when the ultimate bearing capacity is reached.

\[ V_u = \phi V_p \]  
\[ M_u = \eta \left( e V_p \right) / 2 \]  

where \( \phi \) is the shear capacity strengthening factor for the link after consideration of the material over strengthening and strain hardening, usually \( \phi = 1.5 \); \( \eta \) is the seismic action adjustment factor, determined according to the seismic intensity of the frame.

When designing the bolted connection of the variable-section replaceable link to the frame beam, it is assumed that the end moments are all carried by the upper and lower flange bolts and \( V_{uf} \) is the shear force at the flange caused by the end moments \( V_{uf} = M_u / h \). The web splice bolts only carry the end shear force; see Figure 15 for a schematic calculation.

![Figure 15. Bolt connection design of link.](image)

The finite element model (FEM) was established using ABAQUS software, and the C3D8R solid element was selected for the model. In the FEM model, the parts were bonded together using a tie restraint, and the residual deformation and stress concentration of the steel plate caused by the high welding temperature were ignored. In mesh generation, hexahedral elements are used to improve the calculation accuracy, and the mesh is refined appropriately in the link area. The bolt pretension force is considered in multiple steps to converge the calculation. A reduced bolt pretension force is applied first to establish a smooth contact relationship between the bolt and the surrounding, and then the full pretension is applied according to the design value. The bilinear kinematic constitutive model was adopted for the stress–strain curve of the steel plate, defining the stress strengthening of 0.01 E. According to the material property tests, the material properties were determined. The von Mises yield criterion was adopted for the calculation of the model, and Poisson’s ratio was 0.3. The high-strength bolt in the test was used in the model.

The schematic diagram of the base model is shown in Figure 16. The dimensional design of the parametric model was based on the prototype structure regarding the relevant codes and regulations in China, and three sets of parametric models were considered, with a total of nine different models. The various parameters are the length of the energy-consuming region \( e \), the different steel strength combinations, and the length of the replaceable link section \( e' \). The models for each series are shown in Table 3.
The web splice bolts only carry the end shear force; see Figure 15 for a schematic calculation of the consumption region.

Table 3. Dimensions of parametric analysis models.

| Series | Number | LA-1 | LA-2 | LA-3 | LA-4 |
|--------|--------|------|------|------|------|
| LA Series | e/mm | 924 | 840 | 756 | 672 |
| | ρ | 1.28 | 1.17 | 1.04 | 0.92 |
| | Steel | Q345/Q460 | Q345/Q460 | Q345/Q460 | Q345/Q460 |

| Series | Number | LB-1 | LB-2 | LB-3 |
|--------|--------|------|------|------|
| LB Series | e/mm | 840 | 840 | 840 |
| | ρ | 1.17 | 1.17 | 1.17 |
| | Steel | Q345/Q690 | Q345/Q460 | Q235/Q460 |

| Series | Number | LC-1 | LC-2 | LC-3 | LC-4 |
|--------|--------|------|------|------|------|
| LC Series | e/mm | 840 | 840 | 840 | 840 |
| | ε | 1740 | 1840 | 1940 | 2040 |
| | ρ | 1.17 | 1.17 | 1.17 | 1.17 |

In the table, e and e′ are the length of the energy-consuming region and the length of the replaceable link section, respectively, and ρ is the length ratio of the link.

As the frame column and beam do not require high ductility, Q460- and Q690-grade steel were used. To reduce the cross-sectional area of the members, the bolts were selected from M22, 10.9-grade high-strength bolts, and the other members were made of Q345-grade steel. In this paper, solid units were selected to establish the bolt model. The preload was applied according to China’s steel design standard, M22 with grade 10.9 should be used with a preload of 190 kN, and it was applied in steps to the center of the bolt rod. The horizontal loading method was displacement-controlled loading, the Z-axis positive load was applied at the beam end by the coupled action surface until the structure was damaged, and the cyclic loading was applied to study the seismic performance of the structure. The model boundary conditions are shown in Figure 17. The loading protocol was referred to China’s seismic test regulations [21] and the loading protocol is shown in Figure 18.
Figure 18. Loading protocol.

The high-strength eccentrically braced steel frame structure with replaceable link consumed seismic energy through plastic deformation, while the non-energy-consuming elements such as frame columns, beams, and braces were still in the elastic phase. For the numerical analysis in this paper, the structural damage was determined by the following phenomena that occurred: (1) Stress in the link reaching ultimate strength. (2) The maximum story drift of the structure greater than 1/20. (3) Large plastic deformation of members other than links, bolts, and end plates due to local buckling.

4. Discussion Results for the Base Model

4.1. Hysteresis Performance

The horizontal displacement is applied at the center line of the beam to simulate cyclic loading. Figure 19 reflects the hysteresis curve and backbone curve of the base model structure in terms of base shear force versus horizontal displacement. The hysteresis curve of the structure is relatively full. In the loading process, the link enters the yielding stage, the plastic deformation increases, the bearing capacity increases slowly, the area enclosed by the hysteresis curve gradually increases, and finally, after experiencing three times $5\Delta_y$ loading, the replaceable link reaches the ultimate load, the column base enters plasticity, and the structure is declared to be failed. The equivalent plastic strain and the stress distribution of the replaceable link at the time of damage to the base model are shown in Figure 20. From the equivalent plastic strain diagram, it can be seen that under the cyclic loading, the replaceable link as the energy-consuming element of the structure can absorb the energy generated by the reciprocating load through plastic deformation, thus ensuring that the frame beam, column, and brace are in an elastic state. The stress distribution at the time of damage of the replaceable link reflects that the energy-dissipating region in the middle of the beam is more stressful due to its smaller cross-sectional area,
thus enabling a more effective concentration of energy dissipation and ensuring that no permanent deformation occurs at the bolt connection.

Figure 19. Loading–displacement curve of base model.

Figure 20. Failure mode of the base mode.

4.2. Plastic Rotation

The plastic rotation of the link refers to the ratio of the vertical displacement difference between the two ends of the link to the length of the link. Figure 21 shows the shear–rotation curve of the base model for the link under cyclic loading. The shear–rotation hysteresis curve of the base model is full in shape and the elastic rotation of the link is 0.01 rad, with a maximum rotation of 0.17 rad. The plastic rotation far exceeds the code requirements, indicating that the link has good plastic rotation capability.

Figure 21. Shear–rotation curve of link.
5. Finite Element Parametric Analysis

5.1. Analysis of LA Series Results

The LA series is a study of the effect of the length (e) on the seismic performance of the structure. The individual hysteresis curves and backbone curves of the model BA and LA series under cyclic loading are shown in Figures 22 and 23, respectively. In Figure 22, the loading–displacement curves of the LA series models under cyclic loading are all shuttle-shaped, with LA-4 having the fullest hysteresis curve. The final damage of LA-4 occurs at stage $5\Delta y$, when shear yielding has already occurred at the web of the column base and LA-3 is damaged at stage $5\Delta y$, with a similar final damage mode to LA-1. The LA-1 specimen reaches the ultimate bearing capacity at the first $5\Delta y$ loading stage, when the energy-consuming region of the replaceable link has already yielded and the column base has entered the plastic phase.

Figure 22. Hysteretic curves of LA models.

From Figure 23, the initial stiffness of each model LA-1, LA-2, LA-3, and LA-4 increases in turn; under cyclic loading, the initial stiffness of the structure becomes smaller and the ultimate loading capacity gradually becomes lower with the length of the energy-consuming region increasing. The performance characteristic points of the LA series, such as yield loading, ultimate loading, and ductility, are shown in Table 4, with each performance characteristic increasing as the length of the intermediate energy-consuming region decreases.
The elastic rotation of all four model structures is around 0.01 rad. After exceeding the performance of the structure, Figures 25 and 26 show the hysteresis curves and backbone curves. The shear–rotation curves of the LA series under cyclic loading are shown in Figure 24. The elastic rotation of all four model structures is around 0.01 rad. After exceeding the elastic rotation, the plastic deformation in the energy-consuming region of the replaceable link gradually increases and the energy dissipation loop gradually opens, with the ultimate plastic rotation exceeding 0.15 rad.

5.2. Analysis of LB Series

The LB series is a study of different combinations of steel strengths on the seismic performance of the structure. Figures 25 and 26 show the hysteresis curves and backbone curves.
The LB series hysteresis curves in Figure 25 are all shuttle-shaped and have the same shape, of which LB-3 encloses the smallest area. LB-1 enters plasticity after completing 2 times $5\Delta_y$ displacements under cyclic loading, at which time the link segment has reached
the ultimate bearing capacity, the ultimate structural displacement reaches 202.5 mm, and the corresponding ultimate bearing capacity is 5863.5 kN, which is 16.8% higher than that of the LB-2 model (base model). The ultimate load capacity of the structure reaches 3177.4 kN. As can be seen from the backbone curves, different steel combinations have little effect on the initial stiffness of the high-strength eccentrically braced steel frame structure with the replaceable link. From the performance characteristics data in Table 4, it is found that changing the steel strength combination has a significant effect on the structural performance, with the loading capacity decreasing sharply as the steel strength decreases, but the ductility performance increases.

The shear–rotation curve of the LB series under cyclic loading is shown in Figure 27. The series model hysteresis curve is full in shape, in which the LB-1 hysteresis performance is best, and the maximum plastic rotation reaches 0.19 rad, which is 17.6% higher than that of the base model structure. The plastic rotation of the three model structures all complies with the limits specified in code 1.

![Graph showing shear-rotation curves of LB models](image)

*Figure 27. Shear–rotation curves of LB models.*

### 5.3. Analysis of LC Series Finite Element Result

The LC series is a study of the length of the replaceable link on the mechanical and seismic performance of the structure. The results are compared and the pattern is analyzed to arrive at a reasonable length value. Figures 28 and 29 show the hysteresis curves of the LC series under cyclic loading and the backbone curve comparison, respectively. The final damage of the base model occurs at the third $5\Delta_y$ cyclic load stage when the web in the energy-consuming region yields to the full section; the final damage of the LC-2 model occurs at the first $5\Delta_y$ reciprocating load stage when the joint between the energy dissipation region and the non-energy dissipation region as well as the column base yields to the full section; the final damage of the LC-1 model occurs at the first $5\Delta_y$ reciprocating load stage. The final damage of the LC-2 model occurs at the first $5\Delta_y$ reciprocating loading phase when the energy-consuming zone and the non-energy dissipation zone joints and the column base have entered plasticity.

From the comparison of the backbone curves in Figure 29, it is found that the initial stiffness of LC-4, LC-3, LC-2, and LC-1 models gradually increases. Combined with the data in Table 4, the ultimate bearing capacity and displacement of the LC-1 model under reciprocating load reach 5240.2 kN and 184.1 mm, respectively, while the ultimate bearing capacity and displacement of the LC-4 model are 4792.7 kN and 150.1 mm, respectively. The ultimate loading capacity increases by about 5.15% compared to the LC-1 (base model) and decreases by about 4% compared to the LC-4 model. It can be seen that the length $e$ of the replaceable link has a greater influence on the stiffness of the structure, with the stiffness and ultimate loading capacity decreasing as $e$ increases.
Figure 28. Hysteretic curves of LC models.

Figure 29. Backbone curves of LA models.

Figure 30 shows the shear–rotation curves for the LC series under cyclic loading, the hysteresis curves for each model of the LC series under cyclic loading are full, and the plastic rotation increases to varying degrees as the length of the replaceable link decreases, with a slight increase in the corresponding shear force, but the change is not significant. Because the shear force is mainly provided by the energy-consuming region of the link. In the LC-series, the length of the energy-consuming region is not changed; therefore, the
change in shear force in the LC-series is not obvious. The LC-1 model has the best hysteresis performance, with a maximum plastic rotation of 0.19 rad and a corresponding shear force of 990.4 kN, an increase of about 11.8% compared to the base model. The plastic angle of rotation of all three models meets the limits specified in the code.

Figure 30. Shear–rotation curves of LC models.

6. Conclusions

This paper proposes a new type of variable-cross-section replaceable link performing in an energy consumption region and applies it in a high-strength eccentrically braced steel frame structure. The existing test models of the replaceable link are simulated by ABAQUS software, and the effects of the variable-section replaceable link on the loading characteristics and seismic performance of the structure are compared and verified. The loading capacity, stiffness, plastic rotation, force mechanism, and other seismic performances of the structure are discussed under cyclic loading. This paper mainly focuses on the effects of length of the energy-consuming region, the steel strength, and the length of the replaceable link on the seismic performance of the structure. The conclusions are shown as follows.

(1) Regarding the replaceable link causing shear-yielding, reducing the length of the energy-consuming region will clearly improve the loading capacity, ductility, and rotational performance of replaceable link, but neither will enhance the stiffness of the structure. The replaceable link should be limited to \((0.92 - 1.17) M_p / V_p\) when designing high-strength eccentrically braced steel frame structures with the replaceable link; excessively long energy-consuming regions will not provide better performance.

(2) The steel strength of the link and other components presents a significant impact on the mechanical and seismic properties of the structure. Under cyclic loading, with the
increasing strength of steel, the ductility coefficient of the structure decreases while the loading capacity of the structure and the rotation of the replaceable link significantly improve; meanwhile, the initial stiffness of various structures is not obvious to the strength of steel. This suggests the application of Q690 for the frame, while Q345 for the replaceable link can present the best seismic performance of the structure.

(3) The length of the replaceable link also presents a significant impact on the seismic performance of the structure. The reduction in length will clearly improve the loading capacity, ductility, the plastic rotation of the replaceable link, and the energy dissipation capacity of the structure. Especially, reducing the length of the replaceable link can lead to the increase in the initial stiffness and the decrease in stiffness degradation of the structure. The initial stiffness, bearing capacity, and ductility factor are decreased about 6%, 2.3%~4.9%, and 6%~7%, respectively, when the length of the link increases by 100 mm.

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