Research on the influence of the distance between the ribs of the energy dissipating beam section on the seismic performance of the eccentrically supported frame

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Abstract: A reciprocating force loading test was performed of a single-layer shear-yield eccentric brace frame specimen, and the finite element model of stiffeners of several energy-dissipating beam segments with different spacing is established. On the basis of verifying the validity of the finite element analysis, a non-linear numerical analysis of its hysteretic performance is performed, and the bearing capacity, initial stiffness, stiffness degradation and energy dissipation capacity of different model structures are analyzed. Studies have shown that appropriately reducing the spacing of the stiffeners in the energy dissipating beam segments can increase the load-bearing capacity, initial stiffness and energy dissipation capacity of the component, but the residual stiffness of the structure can be reduced.

1. Introduction

The characteristic of eccentric support is that at least one of the two ends of the support diagonal rod does not intersect at the beam-column node, so that an energy-dissipating beam section will be formed between the column and the diagonal rod, or between the diagonal rod and the diagonal rod$^{[1-2]}$. The eccentrically braced frame has both the strength and rigidity of the central support frame$^{[3]}$ and the ductility of the pure frame. Under the action of strong earthquakes, energy is dissipated through the inelastic deformation of the energy dissipating beam section to make the energy dissipating beam section shear Yield first, so as to protect the support diagonal bar from yielding or yielding later$^{[4-6]}$, it is a good seismic structure system.

Zhang Guangwei et al$^{[7]}$ conducted a nonlinear finite element analysis on six eccentric supports with different lengths and found that the energy dissipation capacity of the specimen first increased and then decreased with the increase of the length of the energy-dissipating beam segment. Su Mingzhou et al$^{[8]}$ conducted a nonlinear finite element analysis on the web thickness of five energy dissipating beams with different thicknesses, and found that as the web thickness of the energy dissipating beam section increased,
the bearing capacity and energy dissipation performance of the structure gradually increased, but when the thickness increases to a certain extent, the energy consuming capacity decreased drastically. Duan Liusheng et al\cite{9} conducted a numerical analysis on eccentrically supported steel frames with different steel combinations and found that when the strength of the energy dissipating beam section steel is the same, the load bearing capacity of specimens will increase and the stiffness degradation rate will be slow if the steel strength of the frame segment is improved.

In this paper, on the basis of experiments, the hysteretic properties of several eccentrically braced steel frames with stiffeners of different spacing are analyzed by nonlinear finite element method under the same design conditions. The load-bearing capacity, initial stiffness, stiffness degradation, and energy consumption capacity of each model are compared.

2. Test overview

The test specimen is a 1:2 reduced-scale single-layer single-span K-type eccentric support frame of shear-yielding type for the energy dissipating beam section, with a floor height of 1800mm, a span of 3600mm, and a beam section length of 600mm. The frame beam and the frame columns are supported by Q345C, and Q235C steel is used for the energy-dissipating beam section. The cross-sectional dimensions of each member and the results of the steel material properties test\cite{10-11} are shown in Table 1 and Table 2 respectively.

| Component          | Section size               | Component               | Section size               |
|--------------------|---------------------------|-------------------------|---------------------------|
| Frame beam         | H250×125×6.5×9            | Support                 | H125×125×6.5×9            |
| Frame column       | H200×200×8×12             | Energy dissipating beam section | H250×125×6.5×9            |

| Steel  | Thickness/mm | Yield Strength/MPa | Ultimate strength/MPa | Elastic Modulus/GPa | Elongation/% |
|--------|--------------|--------------------|-----------------------|--------------------|--------------|
| Q235C  | 6.5          | 273.5              | 448.5                 | 230                | 37           |
| Q235C  | 9            | 257                | 465                   | 233                | 30.7         |
| Q345C  | 6.5          | 360.27             | 537.7                 | 221.37             | 29.83        |
| Q345C  | 9            | 327.5              | 530.6                 | 224.3              | 33.83        |

The test was carried out in the Structural Laboratory of Luoyang Institute of Technology. A vertical load of 200KN was applied on the top of the column through a jack, and the horizontal load was applied by a 100T actuator. Test loading device such as 1. It is stipulated that the west direction is positive (push), and the load-displacement hybrid control method is adopted. The vertical load is first applied to the top of the column. After the stability is reached, the specimen is loaded horizontally under load control. Before yielding, it is increased by \( P_y/4 \), and each level is cycled 3 times; After yielding, it is increased by \( N\Delta y \) with 3 cycles per level. \( P_y \) is the estimated yield load, \( \Delta y \) is the estimated yield displacement. When the load reaches 516.81KN in the pulling direction and the displacement is 48.32mm in the first circle of the eighth level of displacement loading, the web of the first zone from west to east torn from the lower left corner to the middle of the web, the lower flange of the second zone was severely buckled, and other energy-dissipating beam sections also have varying degrees of buckling. The energy-dissipating beam sections basically withdrew from work and the experiment ended. The damage of the specimen is shown in Figure 2.
3. Establishment of finite element model

3.1 Model design

In order to study the influence of the number of stiffeners in the energy dissipating beam section on the seismic performance of semi-rigid eccentrically braced frames, a set of 4 finite element models were designed for nonlinear finite element analysis according to the current "Code for Seismic Design of Buildings"[12]. And the combined methods are shown in Table 3. In order to facilitate comparison, all models are completely consistent except for the number of stiffeners.

| Specimen number | Stiffener spacing | Energy dissipating beam section | Frame column | Support | Frame beam | Bolt |
|-----------------|-------------------|---------------------------------|--------------|---------|------------|------|
| S-0             | Noting            | Q235                            | Q345         | Q345    | Q345       | M20  |
| S-1             | 300               | Q235                            | Q345         | Q345    | Q345       | M20  |
| S-2             | 200               | Q235                            | Q345         | Q345    | Q345       | M20  |
| S-3             | 150               | Q235                            | Q345         | Q345    | Q345       | M20  |
3.2 Grid type and model establishment

The finite element size is consistent with the experiment, and ABAQUS solid modeling is adopted. In this paper, the eccentric support model bolts, energy-dissipating beams, frame beams, and frame columns all adopt 8-node solid elements (C3D8R), and 6-node female triangular prism elements (C3D6) are used at the corners of diagonal bracing. All nodes are densely meshed with edge cloth.

3.3 Model material constitutive relationship

The steel adopts the multilinear kinematic hardening (KINH) constitutive model, the elastic modulus E of the steel and the bolt are both 2.06×105MPa, and the Poisson's ratio is 0.3. The Mises yield criterion is adopted, and the combined hardening rule suitable for cyclic loading is adopted. According to the simplification, the welding seam is as strong as the base metal, the TIE constraint is adopted to bind the beam-column contact part, and the influence of the residual stress of the welding seam and the initial defects of the component is ignored.

3.4 Loading system and boundary conditions

In order to eliminate the influence of different loading systems on the energy consumption of the specimens[13], finite element adopts displacement control method for loading uniformly. The cyclic loading method is carried out in the manner of $\Delta y$, $2\Delta y$, $3\Delta y$, ..., where $\Delta y$ represents the yield displacement, as shown in Figure 3. Each level of displacement cycles once. When there is no decline in the bearing capacity before the model is damaged, it is considered that the structure is destroyed when the interlayer displacement angle of the structure reaches 1/50, and the displacement at this time is the limit displacement of the structure $\Delta m$.

All the degrees of freedom of the column bottom are constrained to simulate rigid joints, and the out-of-plane degrees of freedom are constrained. The vertical load of 200KN is applied to the top of the column, and the bolt pre-tightening force is 166KN, as shown in Figure 4. The model does not consider initial geometric defects and welding residual stresses.

![Fig.3 Loading system](image)

![Fig.4 Boundary conditions](image)

3.5 Finite element reliability verification

In order to ensure the correctness of the finite element model used in this paper, the skeleton curve and failure patterns of the S-2 model are compared with the test results. It can be seen from Table 4 that the yield load $P_y$ of the finite element is slightly higher than the test value, and the ultimate load $P_m$ is equivalent to the test value, which is in good agreement.

It can be seen from the analysis that the overall finite element skeleton curve is slightly higher than the test curve. The reason is that the finite element considers the influence of initial geometric defects and welding residual stress; The finite element does not consider the metal damage and the damage of the test in the loading process. Since the finite element model does not consider weld fracture and metal damage, the bearing capacity does not decrease.
Table 4  Comparison of finite element and experiment

| Result | Yield load | (Finite Element-Test Value)/Test Value | Ultimate load | (Finite Element-Test Value)/Test Value |
|--------|------------|---------------------------------------|---------------|---------------------------------------|
| Fem    | 302.21     | 22.6%                                  | 688.5         | 2.80%                                 |
| Exp    | 246.44     |                                       | 671.25        |                                       |

4. Finite element analysis

4.1 Hysteresis curve

Figure 6 shows the hysteresis curve of each model under cyclic loading. It can be seen from the figure that before yielding, the hysteretic curve is basically a straight line, the residual deformation is very small, and the specimen is in the elastic working stage. After the specimen yields, as the horizontal displacement increases, the slope of the ascending curve gradually decreases, and the hysteretic curve begins to incline to the displacement axis, the residual deformation after unloading becomes larger and larger, and the area enclosed by the hysteresis loop gradually increases, and energy consumption begins. The hysteretic curves of all eccentric support frames are fusiform, full and stable, and show good energy consumption. A cycle of $6\Delta y$ can be completed before the displacement angle between layers reaches 2%.
4.2 Skeleton curve

Figure 7 shows the skeleton curve of each model under cyclic load. It can be seen from the figure that the four skeleton curves are all straight lines before the specimen yields, and the initial stiffness is relatively large. After the specimen yields, the curve stiffness begins to decrease, the skeleton curve gradually bend to the displacement axis, the increase rate of the load slows down, and the specimen enters the elastic-plastic working stage. Each curve reaches the specified limit of the displacement angle between layers, i.e. there is no descending section at 1/50, so the bearing capacity at 1/50 of the displacement angle between layers is taken as the ultimate bearing capacity.

4.3 Carrying capacity

It can be seen from Figure 7 that the bearing capacity of each model shows an upward trend with the increase of horizontal displacement. Therefore, the bearing capacity corresponding to when the displacement between layers reaches 1/50 of the layer height is the ultimate bearing capacity. The bearing capacity of each model is
shown in Table 5.

Table 5  Carrying capacity of each model

| Model   | S-0      | S-1      | S-2      | S-3      |
|---------|----------|----------|----------|----------|
| Ultimate bearing capacity/KN | 701.18   | 709.27   | 729.61   | 690.43   |

It can be seen from Table 5 that appropriately reducing the spacing of the stiffeners in the energy dissipating beam section can increase the bearing capacity of the eccentric braced frame. The bearing capacity of S-2 is increased by 4.1% and 2.87% for S-0 and S-1 respectively. But the stiffening the spacing of the ribs should not be too small. The bearing capacity of S-3 is 1.53%, 2.66% and 5.37% lower than S-0, S-1 and S-2 respectively. It can be concluded that the spacing of stiffeners in the energy dissipating beam section has more or less influence on the bearing capacity of the structure. It can be seen from Table 5 that the spacing of the stiffeners in the energy dissipating beam section is not as small as possible but should be controlled within an appropriate range.

4.4 Stiffness degradation

The stiffness of the structure corresponds to the tangent stiffness of the load curve of the structure. It is defined as the ratio of the sum of the absolute value of the forward and reverse load peak value and the corresponding absolute value of the displacement peak value when the load is at the same level.

The stiffness of the test piece is expressed by the secant stiffness, and the secant stiffness $K_i$ is calculated according to formula (1):

$$K_i = \frac{\left| +F_i \right| + \left| -F_i \right|}{\left| +X_i \right| + \left| -X_i \right|}$$

In the formula, $+F_i$, $-F_i$—Load value of the i-th positive and negative peak point(KN);

$+X_i$, $-X_i$—The displacement value of the i-th positive and negative peak point(mm).

Table 6  Comparison of stiffness of each model

| Model   | S-0      | S-1      | S-2      | S-3      |
|---------|----------|----------|----------|----------|
| Initial stiffness | 61.87   | 63.98   | 67.16   | 64.89   |
| Failure stiffness | 19.4    | 19.52   | 20.07   | 18.94   |
| Residual stiffness ratio | 31.36%  | 30.51%  | 29.88%  | 29.19%  |

It can be seen from Table 6 that when the spacing of stiffeners in the energy dissipating beam is larger, the initial stiffness of the structure is smaller. As the spacing of stiffeners decreases, the initial stiffness of the model gradually increases. The initial stiffness of S-2 is 8.55% and 4.97% higher than S-0 and S-1. However, when the number is greater than 2, the initial stiffness tends to decrease, and the initial stiffness of S-3 is 3.38% lower than that of S-2.

The stiffness degradation law is shown in Figure 8. It can be seen from the figure that the stiffness degradation law of the four models is basically the same. The stiffness degradation curve attenuates relatively uniformly, and there is no obvious sudden change in stiffness. It still has a certain stiffness until the destruction, indicating that the specimens are all good seismic performance. After analysis, it can also be known that increasing the number of stiffeners will increase the stiffness degradation rate ($K/K_0$) of the structure and reduce the residual stiffness. The residual stiffness of S-3 at 1/50 of the displacement angle between layers has been degraded to 29.19%, which is 6.92%, 4.32% and 2.31% lower than S-0, S-1 and S-2 respectively. The above shows that appropriately reducing the spacing of the stiffeners in the energy dissipating beam section can increase the initial stiffness of the structure, but it will also lead to an increase in the stiffness degradation rate.
4.5 Energy consumption capacity

Energy consumption capacity is the ability to consume energy. Since the product of displacement and force is energy, that is, the area enclosed by the hysteresis curve is the energy consumed. The energy dissipation capacity of the test structure is measured by the area enclosed by the load-displacement hysteresis curve, that is, energy dissipation coefficient \(^{[14]}\). S-2 energy dissipation coefficient \(\varepsilon_2\) is calculated according to formula (2):
Table 7  Energy consumption coefficient of each model

| Model | S-0  | S-1  | S-2  | S-3  |
|-------|------|------|------|------|
| Energy consumption coefficient | 1.724 | 1.784 | 1.858 | 1.923 |

It can be seen from Table 7 that the energy dissipation capacity of different specimens is quite different, and the energy dissipation coefficient $E_3$ is 11.5%, 7.79% and 3.5% higher than $E_0$, $E_1$, and $E_2$ respectively. The results show that reducing the spacing of the stiffeners of the energy dissipating beam section can significantly improve the energy dissipation capacity of the component.

5. Conclusion
Through the nonlinear analysis of the hysteretic performance of the finite element model of the eccentrically braced frame with four different stiffener spacings, the following conclusions are obtained:

1. The maximum load-bearing capacity and failure mode of the specimen obtained from the test and the finite element are basically the same, and the maximum load-bearing capacity error does not exceed 5%. It shows that the finite element modeling method in this paper is effective and can better simulate the seismic performance of the eccentric support frame.

2. Under the same design conditions, reducing the spacing of the stiffeners in the energy dissipating beam section can improve the bearing capacity of the member, but when the number is greater than 2, the bearing capacity of S-3 is 5.37% lower than that of S-2.

3. Under the same design conditions, reducing the spacing of the stiffeners in the energy dissipating beam section can increase the initial stiffness of the member, but when the number of stiffeners is greater than 2, the initial stiffness of S-3 is 3.38% lower than that of S-2.

4. Under the same design conditions, reducing the spacing of the stiffeners in the energy dissipating beam section will increase the stiffness degradation rate of the structure and reduce the residual stiffness of the structure.

5. Under the same design conditions, appropriately reducing the spacing of the stiffeners of the energy-dissipating beam section can significantly improve the energy-dissipating capacity of the component.

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