Influence of Low Yield Point Reinforcement and High Toughness Concrete on Seismic Performance of the Frame

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Abstract: Currently, ordinary concrete and steel bars are mostly used in frames, but there is no systematic research on the impact of high toughness concrete and low yield point steel bars on the seismic performance of frames. The low-cycle repeated load test of three 1/3 ratio two-story and two-span cast-in-situ concrete frame specimens is carried out. The resilience model hysteresis characteristics, displacement ductility, and energy dissipation capacity are studied and analyzed by experiment. In addition, a comparative analysis is made between the ordinary frame and the frame with low yield point reinforcement. The restoring force model of reinforced concrete (RC) frame with low yield point reinforcement and high toughness concrete is given. The results show that the frame with low yield point reinforcement has better ductility and lateral stiffness than the ordinary reinforced concrete frame. The ductility of the frame will be improved by using high toughness concrete. The deformation ability of the members will be improved by using low yield point steel and high toughness concrete in the frame.

Keywords: low yield point steel; RC frame; seismic performance; energy dissipation capacity

1. Introduction

Among the forms of multi-story and high-rise buildings in various countries, reinforced concrete structure is the most widely used. Reinforced concrete frame structure is the most commonly used structural form. Reinforced concrete frame structure has sufficient strength, good ductility, and strong integrity, and is widely used in seismic fortification areas. Due to the limitations of traditional reinforced concrete structures, especially the low strain cracking of concrete materials, the material function of steel bars cannot be effectively exerted in the linear elastic stage of structures. Low yield point reinforcement is to provide energy consumption capacity for the structure, with lower strain yield point and less damage accumulation. High toughness concrete is to provide effective protection for steel bars, and there is no cracking in the effective working section of the structure. Its damage accumulation is small, and it has the load-bearing function needed by the structure.

When the frame structure is under the action of large earthquake force, the working state of the structure will change from elastic stage to plastic stage [1–3]. Therefore, the frame structure will produce great deformation. The structure must have a certain plastic deformation capacity, so that it can absorb energy by generating plastic deformation under the condition of maintaining a certain carrying capacity, thus improving its seismic capacity [4]. The frame structure is mainly composed of bending members, vertical frame columns, and bending shear members, horizontal frame beams. Kobori et al. [5] first proposed an AVS system (active variable stiffness system). G. Shi et al. [6] proposed a variable damping system. L. Yang [7] carried out a deep theoretical analysis of the variable stiffness semi-active control structure and simulated a shaking table experiment of the variable stiffness semi-active control of a five-layer steel frame structure model. Low yield point steels usually have excellent mechanical properties, including low yield strength, good ductility, excellent energy dissipation capacity, and fatigue properties [8,9]. Compared
with traditional steel, the inherent mechanical properties of low yield point steel make it an ideal material for energy dissipation components [10]. At present, typical applications of low yield point steel include buckling restrained brace (BRB), steel plate shear wall, coupling beam, and various damper types [11,12]. The research combines the latest research results of low yield point metal energy dissipation technology and high toughness concrete. A new reinforced concrete frame structure with different yield points is developed. It has a strong energy dissipation ability and a large range of elastic working sections. Frame structural members are composed of ordinary steel bars, low yield point steel bars, and high toughness concrete materials.

The energy consumption mechanism was expounded, and a reasonable mechanical model was established in this paper. In order to provide a reference for engineering design, the seismic damping capacity of RC frame with low yield point reinforcement is explored by means of a test. The arrangement of different low yield steel bars in the frame structure is equivalent to a built-in damper [13]. Its working principle under the action of an earthquake is that when the ordinary reinforcement and high toughness concrete are still in the linear elastic state, the low yield point reinforcement enters into the plastic deformation earlier because of the lower yield point so that the whole structure can consume the seismic energy in the online elastic working state.

2. Experimental Design

2.1. Specimens Design and Fabrication

The bearing capacity, axial compression ratio, reinforcement ratio of stirrups, the cross-sectional area of each member, and concrete strength grade of the frame are the same. The reinforcement of the test piece is shown in Figure 1 (Unit: mm). The design strength grade of three 1/3 double-layer double-span reinforced concrete frames is C35. The specimen numbers are KJ-1, KJ-2, and KJ-3, respectively. Tables 1–4 show the information and section parameters of specimens.

![Figure 1. Dimension and steel details of frame specimens (Unit: mm). (a) KJ-1. (b) KJ-2.](image-url)
Table 1. Details of frame specimens.

| Test Piece No. | KJ-1 | KJ-2 | KJ-3 |
|----------------|------|------|------|
| Concrete type  | Ordinary concrete C35 | Ordinary concrete C35 | High toughness concrete C35 |
| Types of longitudinal bars of beams | Ordinary grade III steel | Low yield point reinforcement | Low yield point reinforcement |

Table 2. Mechanical properties of concrete.

| Concrete Type | Cube Compressive Strength (N/mm$^2$) | Axial Compressive Strength (N/mm$^2$) | Elastic Modulus (N/mm$^2$) |
|---------------|------------------------------------|-------------------------------------|---------------------------|
| Ordinary concrete C35 | 38.15 | 25.51 | $3.22 \times 10^4$ |
| High toughness concrete C35 | 35.15 | 23.51 | $3.03 \times 10^4$ |

Table 3. Mechanical properties of steel.

| Steel Type            | Diameter (mm) | Yield Strength (N/mm$^2$) | Ultimate Strength (N/mm$^2$) | Elastic Modulus (N/mm$^2$) |
|-----------------------|---------------|---------------------------|-----------------------------|---------------------------|
| grade III steel bars  | 8             | 367.6                     | 496.8                       | $2.23 \times 10^5$        |
|                       | 10            | 362.1                     | 392.2                       | $2.23 \times 10^5$        |
|                       | 12            | 353.7                     | 488.2                       | $2.23 \times 10^5$        |
| Low yield point steel bars | 10 | 225.2                     | 245.7                       | $0.17 \times 10^5$       |

Table 4. Section parameters of the test frames.

| ENo. | Beam Reinforcement Ratio | Column Reinforcement Ratio | Hoop Ratio Spacing |
|------|--------------------------|----------------------------|--------------------|
|      | Common Reinforcement Ratio | Low Yield Steel Ratio | 2.26% | φ8@50/100 |
| KJ-1 | 1.13% | / | 2.26% | φ8@50/100 |
| KJ-2 | 1.13% | 1.5% | 2.26% | φ8@50/100 |
| KJ-3 | 1.13% | 1.5% | 2.26% | φ8@50/100 |

The test piece was made on-site and maintained naturally (Figures 2 and 3). Both specimens KJ-1 and KJ-2 were made of C35 commercial concrete, and KJ-3 was made of C35 commercial fiber concrete. The time of watering and curing after the fabrication of the test piece was more than 7 days. After the specimens reached the design strength after 28 days of curing, they were hoisted back to the structure laboratory. The surfaces of the specimens were brushed with white lime water and the upper grid marked so that the specimens could be used to observe the direction of the crack when they were stressed.

The strain gauges of the frame were distributed at the bottom end of the column and the beam-column joint. Four strain gauges were pasted symmetrically at each position, and a total of 56 strain gauges were pasted on each specimen. The resistance strain gauge was type BX120-10AA with a sensitivity coefficient of 2.18, and the specific arrangement position of the strain gauge is shown in Figure 4.
Figure 2. Production plans of the frameworks.

Figure 3. Accomplished test frame.
2.2. Test Process

The hysteretic curve of the frame under earthquake was obtained by low cycle repeated loading. By measuring the stiffness, strength, ductility, and energy dissipation capacity of the structure, the basic seismic performance of the specimens were judged and identified.

2.2.1. Loading Device

The vertical load was applied to the pressure design value in stages through three hydraulic jacks fixed on the top of the frame column. The horizontal load was realized by MTS multi-channel coordinated loading test system. The rated loading capacity of the actuator was 250 kN and the maximum stroke was ±100 mm. The loading device is shown in Figure 5. A schematic diagram of the frame and panorama of framework are shown in Figures 6 and 7, respectively.
2.2.2. Loading System

Vertical loading: A vertical load was applied on the top of the column to simulate the load from the floor slab in the actual project. First, it was loaded to 30% of the scheduled load. We checked whether the equipment was normal before loading it to the scheduled load. During the vertical loading test, the specified load was applied on the top of the frame column first and remained unchanged during the test.

Horizontal loading: The horizontal load was applied to simulate the horizontal force of an earthquake. The load-displacement hybrid control method was adopted. Before the specimens yielding, increment control was adopted. Here, 10 kN was a one-stage increment, and each stage was cycled once. After yielding, the specimens were controlled by equal amplitude displacement increment. The horizontal displacement $\Delta y$ at yield was taken as the first-order increment, and each displacement loading cycle was done 3 times until the horizontal load dropped to 85% of the maximum value.

Loading process:

1. Apply vertical load and keep the load constant.
2. Preload two levels and check whether each instrument works normally.
3. Apply horizontal load every 10 kN until the test piece yields.
4. Whether the specimens yield or not is determined by the apparent inflection point of the $P-\Delta$ curve and the strain reading.
5. After yielding, the frame is loaded by displacement control until the specimens are damaged. The loading system is shown in Figure 8.

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**Figure 6.** Schematic diagram of the frame.

**Figure 7.** Panorama of framework.
2.3. Yield Point of the Frame

During the test, the specimens experienced four stages: cracking, yield, ultimate bearing capacity, and failure. The initial crack point of the specimens could be determined according to the initial crack load and deformation in the test process. For flexural members, it was generally possible to judge the yield of members by controlling whether the reinforcement in the section reaches the yield strain. However, for the frame model, it is difficult to accurately determine the yield of reinforcement in the control section of the frame. Therefore, an assumed yield point was determined for the skeleton curve. In this test, the combination of the inflection point method and monitoring whether most tensile reinforcement in the control section reaches yield was used to judge the yield load and yield displacement of the structure. When the horizontal load dropped to 85% of the peak load, it was considered that the specimen was damaged, and the test was stopped.

3. Results and Analysis
3.1. Failure Mode Analysis of Frame Test
3.1.1. Specimen KJ-1
1. Load control phase

After applying the vertical load to KJ-1, the vertical load was kept stable. When the positive load was 20 kN, the crack began to appear from above the left end of L1, with a crack width of about 0.01 mm and a crack length of 7 cm. and the crack was closed after unloading. As shown in Figure 9a, some new cracks were generated at both ends of L1, L2, and L4 during continuous loading, and the cracks continued to develop with the increase of load. When the load was −40 kN, the cracks above the right end of L1 and L2 and below the left end of L4 were penetrated. With the increase of load, some new cracks continued to appear. The generation position moved to the thinned-span direction, and the original crack continued to develop into a through the crack. When the load was +80 kN, cracks began to appear at the bottom of the left side of columns Z2 and Z3 (Figure 9b).

When the load was −130 kN, the existing cracks at the column bottom began to penetrate, and new cracks at the column bottom were generated upward. When loaded to +150 kN, micro cracks began to appear at L3 and Z2 nodes and L4 and Z3 nodes. When loaded to 165 kN, the crack width at the upper ends of Z1 and Z2 reached 0.2 mm, and the crack at the end of L4 reached 0.5 mm. When the load was −190 kN, it can be seen from the measured value of reinforcement strain that the plastic hinges at both ends of L1 and L2 were basically formed. All cracks in the beam and column occurred within 30 cm from the node and are relatively concentrated.
the measured value of reinforcement strain that the plastic hinge at the bottom of the column was basically formed. When the positive load was 62 mm, the crack width of the beam column exceeded 2 mm, and a large number of vertical cracks appear in the column. The concrete at the bottom of Z1, Z2, and Z3 columns was crushed, and the damage at the bottom of Z3 was the most serious (Figure 9c). The concrete was crushed, bulging, and peeling. When the reverse load was $-62$ mm, the concrete in the plastic hinge area of the beam was crushed. Among them, L1 and L2 ends are most seriously damaged, the concrete was crushed and peeled, and the longitudinal reinforcement of the stirrup was exposed. When the positive load was 93 mm, the concrete at the column bottom was seriously crushed and separated, and the cracks at the beam column joints continued to develop. When the reverse load was $-93$ mm, the concrete at the bottom of the column fell off. Z1, Z2, and Z3 stirrups were exposed, the longitudinal bars on both sides were also exposed, and the specimen was damaged.

**Figure 9.** Experimental phenomenon diagram of KJ-1. (a) Cracks in the beams of KJ-1. (b) Cracks in the bottom of columns of KJ-1. (c) Failure modes of KJ-1.

2. Displacement control phase

When the reverse load was $-31$ mm, the Z2 second-floor column produced transverse through cracks and the cracks at the column bottom were widened. It can be seen from the measured value of reinforcement strain that the plastic hinge at the bottom of the column was basically formed. When the positive load was 62 mm, the crack width of the beam column exceeded 2 mm, and a large number of vertical cracks appear in the column. The concrete at the bottom of Z1, Z2, and Z3 columns was crushed, and the damage at the bottom of Z3 was the most serious (Figure 9c). The concrete was crushed, bulging, and peeling. When the reverse load was $-62$ mm, the concrete in the plastic hinge area of the beam was crushed. Among them, L1 and L2 ends are most seriously damaged, the concrete was crushed and peeled, and the longitudinal reinforcement of the stirrup was exposed. When the positive load was 93 mm, the concrete at the column bottom was seriously crushed and separated, and the cracks at the beam column joints continued to develop. When the reverse load was $-93$ mm, the concrete at the bottom of the column fell off. Z1, Z2, and Z3 stirrups were exposed, the longitudinal bars on both sides were also exposed, and the specimen was damaged.

3.1.2. Specimen KJ-2

1. Load control phase

When the positive load was 20 kN, the crack began to appear from above the left end of L1, with a crack width of about 0.01 mm and a crack length of 8 cm. and the crack was closed after unloading. As shown in Figure 10a, when the load was continued, some new cracks were generated at both ends of L1 and the right end of L2, and the cracks continued to develop with the increase of load. When the load was +30 KN, cracks began to appear at the bottom of the right side of column Z1, with a crack length of 15 cm. When the load reached +60 kN, the cracks at L3 and L4 beam ends began to extend to the beam side, and the width reached 0.2 mm.
With the increase of load, some new cracks continued to appear. The generation position moved to the mid-span direction, and the original crack continued to develop through the crack. As shown in Figure 10b, when the load was $-105 \text{kN}$, the crack at the column bottom began to penetrate and develop to the side. When it is loaded to $+125 \text{kN}$, new cracks at the column bottom were generated upward. When loaded to $+145 \text{kN}$, cracks began to appear at L3 and Z2 nodes and L1 and Z2 nodes. When the load was $-150 \text{kN}$, the crack width at the upper end of Z1 and Z2 reached 0.2 mm, and the crack at the left end of L3 reached 1 mm. At this time, the plastic hinges at both ends of L1 and L2 were basically formed.

2. Displacement control phase

When the positive load was 32 mm, no new cracks were generated, and the existing cracks were widened. When the reverse load was $-32 \text{mm}$, the Z3 column base concrete was crushed. It can be seen from the measured value of reinforcement strain that the plastic hinge at the bottom of the column were basically formed. When the positive load was 48 mm, the crack width of beam and column exceeded 2 mm, and a large number of vertical cracks appeared in the column. The concrete at the bottom of Z1, Z2, and Z3 columns was crushed, of which the bottom of Z3 was the most seriously damaged, and the concrete was crushed, bulging, and spalling.

When the reverse load was $-64 \text{mm}$, the concrete in the plastic hinge area of the beam was crushed (Figure 10c), in which the ends of L1 and L2 were damaged most seriously, the concrete was crushed and peeled off, and the longitudinal reinforcement of the stirrup was exposed. When the positive load was 80 mm, the concrete at the column bottom was crushed and separated seriously, and the cracks at the beam-column joints continued to develop. When the reverse load was $-80 \text{mm}$, Z1, Z2, and Z3 stirrups were exposed, the longitudinal bars on both sides were also exposed, and the specimen was damaged.

**Figure 10.** Experimental phenomenon diagram of KJ-2. (a) Cracks in the beams of KJ-2. (b) Cracks in the bottom of columns of KJ-2. (c) Failure modes of KJ-2.
3.1.3. Specimen KJ-3

1. Load control phase

When the positive load was 30 KN, the crack began to appear from above the left end of L1 with a crack width of about 0.01 mm and a crack length of 3 cm. When the load was continued at both ends of L2, some new cracks were generated at the right end of L3, and the cracks continued to develop with the increase of load. When the load was +30 kN, cracks began to appear at the bottom of the right side of column Z1, with a crack length of 14 cm. After unloading, the cracks were closed. When the load was +60 kN, the crack at the left end of the L3 beam began to extend to the beam side.

As shown in Figure 11a, new cracks continued to appear with the increase of load, the position of generation moved to the middle of the span, and the original cracks continued to develop into through cracks. When the load was −105 kN, the crack at the column bottom began to penetrate and develop to the side. When it was loaded to +125 kN, new cracks at the column bottom were generated upward (Figure 11b). When loaded to +145 kN, cracks began to appear at L3 and Z2 nodes and L1 and Z2 nodes. When the load was −150 kN, the crack width at the upper end of Z1 and Z2 reached 0.2 mm, and the crack at the left end of L3 reached 1 mm. At this time, the plastic hinges at both ends of L1 and L2 were basically formed.

![Figure 11. Experimental phenomenon diagram of KJ-1. (a) Cracks in the beams of KJ-3. (b) Cracks in the bottom of columns of KJ-3. (c) Failure modes of KJ-3.](image)

2. Displacement control phase

When the reverse load was −32 mm, the Z3 column base concrete was crushed and the existing cracks were widened. It can be seen from the measured value of reinforcement strain that the plastic hinge at the bottom of the column was basically formed. When the positive load was 48 mm, the crack width of beam and column exceeded 2 mm, and a large
number of vertical cracks appeared in the column. The concrete at the bottom of Z1, Z2, and Z3 columns was crushed, of which the bottom of Z3 was the most seriously damaged, and the concrete was crushed, bulging, and spalling (Figure 11c).

When the reverse load was −64 mm, the concrete in the plastic hinge area of the beam was crushed, in which the ends of L1 and L2 were damaged most seriously, the concrete was crushed and peeled off, and the longitudinal reinforcement of the stirrup was exposed. When the positive load was 80 mm, the concrete at the column bottom was crushed and separated, but it was not serious, and the cracks at the beam column joints continued to develop. When the reverse load was −80 mm, Z1, Z2, and Z3 stirrups were exposed, the longitudinal bars on both sides were also exposed, and the specimen was damaged.

3.2. Analysis of the Mechanical Performance of RC Frame

3.2.1. Hysteresis Curve Analysis

The hysteresis curve of each component in this test is shown in Figure 12. Comparing the hysteresis loop of KJ-2 and KJ-1, it can be seen that before yielding, the hysteresis loop area of KJ-2 was larger than that of KJ-1, which indicates that KJ-2 had good energy dissipation capacity at that time. Under the same displacement increment, the hysteresis loop of KJ-2 was fuller and its energy dissipation capacity was stronger. Similarly, the energy consumption capacity of KJ-3 was better than that of KJ-1, and the hysteresis curve of frame KJ-3 with low yield point reinforcement and high toughness concrete was fuller and stronger.

![Load-displacement hysteresis loop](image)

**Figure 12.** Load–displacement hysteresis loop.

3.2.2. Structural Ductility Analysis

It can be seen from Table 5 that the displacement ductility coefficient of both low yield point reinforcement and high toughness concrete frame KJ-3 was 31.7% higher than that of ordinary reinforced concrete frame KJ-1. The displacement ductility coefficient of frame KJ-2 with low yield point reinforcement was 15.7% higher than that of KJ-1. The displacement ductility coefficient of KJ-3 was 13.8% higher than that of KJ-2.

**Table 5.** Displacement versus equivalent he.

| Specimen Number | Parameter Symbols | $\Delta y$ | $2\Delta y$ | $3\Delta y$ | $4\Delta y$ | $5\Delta y$ |
|-----------------|-------------------|-----------|------------|------------|------------|------------|
| KJ-1            | S hysteresis loop / kN·mm $h_e$ | 24.88    | 65.37      | 100.05     |            |            |
|                 |                   | 0.13      | 0.17       | 0.22       |            |            |
| KJ-2            | S hysteresis loop / kN·mm $h_e$ | 12.06    | 23.87      | 50.41      | 68.50      | 90.33      |
|                 |                   | 0.14      | 0.17       | 0.17       | 0.19       | 0.23       |
| KJ-3            | S hysteresis loop / kN·mm $h_e$ | 31.68    | 72.55      | 120.63     |            |            |
|                 |                   | 0.18      | 0.20       | 0.28       |            |            |

In the table, $\Delta y$ of KJ-2 is 16 mm, and $\Delta y$ of KJ-1 and KJ-3 are both 30 mm. This shows that the frame with low yield point reinforcement had better ductility than the ordinary reinforced concrete frame. At the same time, the ductility of the frame was improved by adding high-toughness concrete.
3.2.3. Analysis of Energy Consumption Capacity of the Structure

The energy dissipation capacity of a structure or component is usually considered as the energy expression of its ductility. Energy dissipation capacity is an important index to measure the seismic performance of a structure and a direct expression of the seismic energy absorbed and dissipated by the structure. The energy dissipation performance of structural members is an important basis for the evaluation of their seismic performance. The equivalent viscosity coefficient of an ordinary reinforced concrete frame is generally between 0.1 and 0.2. According to Jacobson’s equivalent viscous damping theory, the equivalent viscous damping coefficient of the frame was calculated. The calculation is shown in Figure 13 and the calculation formula is as follows:

$$h_e = \frac{1}{2\pi} \frac{S_{ABC} + S_{ACD}}{S_{OBE} + S_{ODF}}$$  \hspace{1cm} (1)

The equivalent viscosity coefficient of the frame in this test was basically within this range, which shows that the test results were basically in line with the actual requirements.

![Figure 13. The equivalent viscous damping coefficient.](image)

3.3. Analysis of the Seismic Performance of the Frame

3.3.1. Factors Affecting Ductility

1. Steels with different yield points

   The test piece KJ-1 was only equipped with an ordinary skeleton, while the test piece KJ-2 was equipped with a low yield point skeleton. All test conditions were the same except for the choice of steel. According to the analysis in Table 6, the ductility coefficient of specimen KJ-2 was 15.7% higher than that of specimen KJ-1. When the bearing capacity was not significantly reduced, the deformation of the structure was significantly improved. It showed that the use of low yield point reinforcement had a good effect on improving the ductility of the reinforced concrete frame.

2. Nature of concrete

   General concrete was used for test piece KJ-2 and high toughness concrete was used for test piece KJ-3. All the test conditions were the same except for the concrete material selection. It was found that the ductility coefficient of KJ-3 was 13.8% higher than that of KJ-2. The bearing capacity of the test piece KJ-3 was slightly lower than that of KJ-2, but the ductility of the structure was greatly improved. This shows that the use of high-toughness concrete also had a significant impact on the mechanical properties of the frame.

| Specimen Number | $\Delta_{y}/mm$ | $\Delta_{u}/mm$ | $\Delta_{uy}/mm$ | $\Delta_{uy}/mm$ | $\mu_\Delta$ |
|-----------------|-----------------|-----------------|-----------------|-----------------|-------------|
| KJ-1            | 52.2            | -38.6           | 14.3            | -13.5           | 3.06        |
| KJ-2            | 53.6            | -52.1           | 15.1            | -14.8           | 3.54        |
| KJ-3            | 64.1            | -63.9           | 16.4            | -15.5           | 4.03        |
3.3.2. Comparative Analysis of Bearing Capacity

According to the analysis in Table 7, the initial crack load of test piece KJ-3 was significantly higher than that of test piece KJ-1, reaching 50%. The yield displacement was increased by 14.4% and the bearing capacity was not decreased. Therefore, adding low yield point reinforcement and high toughness concrete to the specimen can greatly improve its deformation ability. According to the failure characteristics, the concrete crushing area at the beam end and the column root of the specimen (KJ-3) with high toughness concrete were smaller than that of the specimen (KJ-2) without high toughness concrete. The horizontal displacement at the top of the frame and the overall deformation of the frame was larger than that of the specimen (KJ-1) without reinforcement when the specimen (KJ-2) with low yield point reinforcement was yielding.

Table 7. Strength index of various stages.

| Specimen Number | Cracking Load/kN | Yield Load/kN | Ultimate Load/kN | Yield Displacement/mm |
|-----------------|------------------|---------------|------------------|-----------------------|
|                 | Forward | Reverse | Forward | Reverse | Forward | Reverse | Forward | Reverse |
| KJ-1            | 20      | -20    | 148.7   | -147.4  | 213.3   | -202.9  | 14.3    | -13.5   |
| KJ-2            | 20      | -20    | 155.8   | -153.6  | 224.9   | -198.9  | 15.1    | -14.8   |
| KJ-3            | 30      | -30    | 156.6   | -148.2  | 209.3   | -219.7  | 16.4    | -15.4   |

3.3.3. Deformation Capability Analysis

The story drift is the ratio of the horizontal relative displacement value between stories to the height of stories. It is mainly used to measure the deformation capacity of frame structures. In structural design specifications and the limit values of story drift are also commonly used to limit the horizontal displacement of structures.

Under the action of the frequent earthquakes, the maximum story drift of frame structure calculated by the elastic method shall meet the requirements of the following formula:

$$\Delta u_e \leq [\theta_e]h$$

$$\Delta u_e$$—The maximum elastic story drift of floors due to the standard value of frequent earthquake action; $$[\theta_e]$$—Limit value of displacement angle between elastic stories. It is used according to Table 8; $$h$$—Height of calculating floor.

Table 8. Elastic story drift limit for the frameworks.

| Structural System                | $$[\theta_e]$$ |
|----------------------------------|----------------|
| Frame structure                  | 1/550          |
| Frame–shear wall structure       | 1/800          |

Under the action of the rare earthquake, the elastic-plastic story drift of the frame structure should meet the following requirements:

$$\Delta u_p \leq [\theta_p]h$$

$$\Delta u_p$$—Elastoplastic interlaminar displacement caused by standard value of rare earthquake action; $$[\theta_p]$$—Limit value of elastic-plastic story drift, according to Table 9.

Table 9. Elastoplastic story drift limits for the frameworks.

| Structural System                | $$[\theta_p]$$ |
|----------------------------------|----------------|
| Frame structure                  | 1/50           |
| Frame–shear wall structure       | 1/100          |
The structural elastic displacement angle and elastoplastic displacement angle of the tested three frames are shown in Table 10. It can be seen from the data that the elastic displacement angle of the test frame were larger than the limit value specified in the code, which indicates that the lateral rigidity of the frame was more flexible. Compared with the three specimens, the elastoplastic story drift of specimen KJ-3 was greater than KJ-2, and KJ-2 was greater than KJ-1. This shows that under the action of the earthquake, the deformation ability of reinforced concrete frames with a low yield point was strong. The displacement angle of the test was larger than the limit value specified in the code. This was mainly because the code stipulates the displacement angle limit of the whole structure, which needs to consider the spatial effect between the components of the frame structure. The test was carried out on a single frame, ignoring the influence of other components.

Table 10. Yield displacement angle of displacement for the frames.

| Specimen Number | Loading Direction | Elastic Displacement Angle $\theta_e$ | Elastoplastic Displacement Angle $\theta_p$ |
|-----------------|-------------------|---------------------------------------|---------------------------------------------|
| KJ-1            | Forward           | 1/129.4                               | 1/43.1                                      |
|                 | Reverse           | 1/137.1                               | 1/45.7                                      |
| KJ-2            | Forward           | 1/122.5                               | 1/40.8                                      |
|                 | Reverse           | 1/125.0                               | 1/41.7                                      |
| KJ-3            | Forward           | 1/112.8                               | 1/37.6                                      |
|                 | Reverse           | 1/120.1                               | 1/40.0                                      |

From the analysis in Table 10, it can be seen that the deformation capacity of the component was improved by using low yield point steel and high toughness concrete in the frame. In the design, the limit values of the story drift with low yield point reinforcement and high toughness concrete frame can be increased appropriately.

3.3.4. Restoring Force Model of Frame

The restoring force model of a structure or member needs to be established for the non-linear seismic response analysis. The restoring force characteristic curve shows the relationship between the resistance and the deformation in an attempt to restore the original state when the structure is deformed by the external force. At the same time, it also reflects the strength, stiffness, stiffness degradation, deformation capacity, and energy dissipation capacity of the structure and components in the process of repeated loads.

The restoring force model is based on the low cycle repeated load test. According to the obtained restoring force skeleton curve and the law of each cycle curve, the skeleton curve determines each characteristic point of the model. It is the connecting line of each characteristic point in the 1/4 hysteresis cycle. The hysteresis loop indicates the walking path and stiffness degradation of the model in the process of positive and negative loading and unloading, then it is simplified according to certain conditions (such as the equivalence of intensity and energy dissipation capacity) [14].

In this paper, based on the analysis of the hysteretic curve of the RC frame with a low yield point under low cyclic loading, research on the restoring force characteristics of the RC frame was carried out. The restoring force models of reinforced concrete frames with different yield points were established. According to the hysteretic curve of a reinforced concrete frame with different yield points and the skeleton curve connecting each vertex of the hysteretic curve, this paper adopted the degenerate three-line restoring force model (Figure 14), and its stiffness in each stage was:
Figure 14. The restoring force model of the framework.

Stiffness of OA section: the elastic loading rule, the frames had no stiffness degradation and residual deformation before the restoring force reached $P_y$, and the specimens were loaded according to the walking route of O-A. The stiffness formula is expressed as:

$$K_1 = \frac{P_y}{\Delta_y} \quad (4)$$

Stiffness of AB section: the elastic-plastic loading rule. When the restoring force was between the yield load and the peak load, the incremental stiffness of the specimens after yield was taken during loading. The stiffness formula is expressed as:

$$K_2 = \frac{P_{\text{max}} - P_y}{\Delta_{\text{max}} - \Delta_y} \quad (5)$$

Stiffness of BC section: When the restoring force exceeded the peak load $P_u$, it began to enter the descending section. The stiffness formula is expressed as:

$$K_3 = \frac{P_u - P_{\text{max}}}{\Delta_u - \Delta_{\text{max}}} \quad (6)$$

Unloading stiffness before yield and maximum load according to the loading rules of the elastic stage. The stiffness formula is expressed as:

$$K_4 = K_1 \quad (7)$$

Unloading stiffness after maximum load. Stiffness formula is expressed as:

$$K_5 = \alpha K_4, \alpha = \left(\frac{\Delta_y}{\Delta_i}\right)^{0.35} \quad (8)$$

$\Delta_i$—Displacement value of the unloading point of the $i$ time cycle after the highest load. The definition of reverse stiffness is the same as that of forwarding stiffness.

3.4. Nonlinear Finite Element Analysis
3.4.1. Establishment of Finite Element Model

The grid diagram of model is shown in Figure 15. The force damage simulation of the test process was carried out by the finite element software ANSYS15.0 and compared and analyzed with the test results to check and optimize the test results to some extent. Based on the characteristics of the test components and the need to simplify the model and improve operational efficiency, the following basic assumptions were followed in the course of this analysis: (1) The deformation of the member cross-section after the test frame has been stressed still satisfies the flat section assumption; (2) good bonding between reinforcement
and concrete without relative slippage; (3) disregarding the axial tensile strength of the concrete and ignoring the mechanical bite between the aggregates after its cracking; (4) the reinforcement had no transverse shear effect, ignoring the transverse shear deformation of the frame or the torsional effect generated during the test.

Figure 15. Meshing of the model.

A combined model was selected for the reinforced concrete structure. The Solid65 element provided by ANSYS was adopted in concrete. This model can consider many nonlinear properties of concrete and other nonlinear materials, such as cracking, crushing, plastic deformation, and creep of concrete in three orthogonal directions. Beam188 element provided by ANSYS was used for longitudinal and circumferential reinforcement. This element can be applied to plastic, spiral, linear, large rotation, nonlinear large strain, and other problems. Formula (9) was adopted for the stress-strain relationship of concrete under uniaxial compression. A bilinear follow-up hardening steel bar model was adopted in the model, with the same direction of tension and compression. Since the material composition of the test frame members had a strong nonlinear intrinsic relationship between ordinary or low yield point reinforcement and concrete, the ultimate load of the entire frame member was not determined by whether the ultimate load was reached in a single part of the material. In the process of increasing load, the material would inevitably enter the elastic-plastic stage, and the stress-strain relationship for some of the materials may also enter a decreasing segment after the peak. Therefore, when performing finite element analysis, the Multilinear Isotropic Hardening model was used for the concrete principal structure relationship. The steel intrinsic structure relationship is based on the Bilinear Kinematic Hardening model. This test analysis uses the five-parameter William Warnke strength criterion, which basically reflects the damage characteristics of concrete.

The reinforced concrete frame model was built in a bottom-up manner from points to lines to surfaces. In order to prevent the influence of secondary parts on the computational convergence, the main body of the test member was taken for the simulation. In order to prevent localized damage of concrete, a 150 mm thick steel mat was added at each end of the frame column, and various restraints and loads were added to the steel mat. The completed concrete and steel skeleton models are shown in Figure 16. During the loading process, the loading step simulated the loading regime in the test as much as possible, including the load control phase and the displacement control phase. The simulation convergence error was controlled to be within 2% to 3%, not exceeding 5%.

\[
\begin{align*}
&\left\{ \begin{array}{l}
y = a_x x + (3 - 2a_x) x^2 + (a_d - x) x^3 \\
y = x / [a_d (x - 1) + x]
\end{array} \right. \\
&x \geq 1
\end{align*}
\]

\[(9)\]

\[
x = \frac{\varepsilon}{\varepsilon_c} \quad y = \frac{\sigma}{f_c}
\]

\[(10)\]

in which, \(a_x\) and \(a_d\) are the parameter values of the ascending and descending sections of the uniaxial compressive stress-strain curve, \(f_c\) is the uniaxial compressive strength of concrete, and \(\varepsilon_c\) is the peak compressive strain of concrete corresponding to \(f_c\). The ascending parameter \(a_x = 1.985\), the descending parameter \(a_d = 1.549\), and the peak compressive strain \(\varepsilon_c = 1.7 \times 10^{-3}\).
3.4.2. Results of Finite Element Analysis

We performed finite element simulations on three-bay frame specimens. The analysis results of KJ-3 are shown here. The stresses in the frame at the ultimate load are shown in Figure 17a, the displacements of the frame in the x-direction are shown in Figure 17b, and the cracks are shown in Figure 17c.
Figure 17. The diagram of KJ-3 result analysis. (a) Stress contour of frame under the ultimate load. (b) The X displacement contour of frame under the ultimate load. (c) Cracks of frame under the ultimate load.

3.4.3. Comparative Analysis of Load-Displacement Curve Results

The comparison of the load-displacement curves obtained from the finite element analysis of the three specimens and those obtained from the tests are shown in Figures 18–20, respectively. From the analysis of the figure, it can be seen that from the load-displacement curve as a whole, the two trends were generally consistent. Due to the discrete nature of the material itself and the different test methods, as well as the error during the test and the different values of the parameters during the finite element analysis, there are still some differences between the experimental results and the finite element analysis results. However, the overall trend can still be consistent with the experimental results, it indicates that the finite element simulation analysis can be used to better guide the test in the pre-test of future frames.

Figure 18. The comparison between the KJ-1 curve and the curve of the finite element analysis.
1. The main factors influencing the ductility of a reinforced concrete frame are the use of low yield point steel and high toughness concrete. The displacement ductility coefficient of frame KJ-2 with low yield point reinforcement was 15.7% higher than that of common frame KJ-1. The displacement ductility coefficient of high toughness concrete frame with low yield point reinforcement KJ-3 was 13.8% higher than that of KJ-2. This shows that the frame with low yield point reinforcement had better ductility than the ordinary reinforced concrete frame. At the same time, the ductility of the frame will be improved by adding high toughness concrete.

2. The hysteretic curves of high toughness concrete frame KJ-3 with low yield point reinforcement and frame KJ-2 with low yield point reinforcement were full, and both had good energy dissipation capacity. In contrast, the hysteretic curve of high toughness concrete frame KJ-3 with low yield point reinforcement was fuller and the energy dissipation capacity was larger. The elastic displacement angle of the test frame was larger than the limit value specified in the code, which indicates that the lateral rigidity of the frame was more flexible.

3. Conclusions

Based on the test, the influence of the yield point of steel bar and the toughness of concrete on the seismic performance of reinforced concrete frame was analyzed. The common frame, the frame with low yield point reinforcement, and high toughness concrete frame with low yield point reinforcement were compared and analyzed. The restoring force model of the RC frame with low yield point reinforcement and high toughness concrete was given. The main conclusions are as follows:

1. The main factors influencing the ductility of a reinforced concrete frame are the use of low yield point steel and high toughness concrete. The displacement ductility coefficient of frame KJ-2 with low yield point reinforcement was 15.7% higher than that of common frame KJ-1. The displacement ductility coefficient of high toughness concrete frame with low yield point reinforcement KJ-3 was 13.8% higher than that of KJ-2. This shows that the frame with low yield point reinforcement had better ductility than the ordinary reinforced concrete frame. At the same time, the ductility of the frame will be improved by adding high toughness concrete.

2. The hysteretic curves of high toughness concrete frame KJ-3 with low yield point reinforcement and frame KJ-2 with low yield point reinforcement were full, and both had good energy dissipation capacity. In contrast, the hysteretic curve of high toughness concrete frame KJ-3 with low yield point reinforcement was fuller and the energy dissipation capacity was larger. The elastic displacement angle of the test frame was larger than the limit value specified in the code, which indicates that the lateral rigidity of the frame was more flexible.
3. After using low yield point steel and high toughness concrete in the frame, the deformation capacity of the members was improved. In the design, the limit value of story displacement angle of the frame with low yield point reinforcement and high toughness concrete can be increased properly.

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