Experimental study on seismic performance of prefabricated viscoelastic damping bolted joints

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ARTICLE INFO

Keywords:
Prefabricated concrete
Beam-column joint
Seismic performance
Bolted connection
Viscoelastic damper

ABSTRACT

Aiming at the defect of insufficient bearing capacity of prefabricated concrete beam-column joint with bolted connection, a prefabricated viscoelastic damping bolted joint (VDBJ) is proposed with good seismic performance. The damping principle is based on that viscoelastic (VE) dampers can provide energy dissipation and additional stiffness for the structures. Three full-scale beam-column joint specimens were designed, including one cast-in-place joint specimen, one prefabricated bolted joint specimen without VE dampers and one prefabricated bolted joint specimen with VE dampers. Since the viscoelastic damper is a passive energy dissipation device with velocity correlation, comparison and analysis of three specimens under high-frequency cyclic loading are performed on the failure modes and seismic performance indexes such as hysteretic energy dissipation, displacement ductility, stiffness degradation and ultimate bearing capacity. The analysis of the test results indicate that the proposed prefabricated viscoelastic damping bolted joint has stronger ultimate bearing capacity and displacement ductility than the cast-in-place joint. The arrangement of VE dampers effectively controls the appearance and expansion of concrete cracks, which not only achieves the expected effect of shock absorption and energy dissipation, but also can realizes the rapid replacement of components after the earthquakes.

1. Introduction

Compared with the cast-in-place concrete structure, the precast concrete structure has the advantages of saving energy, improving the construction conditions in the production process, and accelerating the construction progress [1]. However, the previous earthquake disaster shows that the serious damage at the beam-column joint is an important reason to limit the popularization and application of prefabricated buildings. Therefore, how to improve the seismic performance of the joint connection position through reasonable design has become a hot issue for scholars [2-5].

At present, the connection methods of prefabricated building beam-column joint can be divided into two categories: wet connection and dry connection. The wet connection method is mainly to splice the precast beam and column first, and then pour concrete in the post-cast-region reserved in advance. The beam-column joints with wet connection have been proved to emulate the seismic performance of cast-in-place joint [6,9]. Different from the former, dry connection avoids unnecessary wet operation on-site, and the precast beams and columns are spliced together mainly through prestressed tendons, bolts and welding. Compared with the two methods, dry connection is considered to be more in line with the trend of building industrialization [7-9].

In the above dry connection modes, bolted connection has been widely studied because of its rapid installation and convenient construction. Ertas et al. [10] conducted comparative experiments on precast concrete beam-column joints with different connection modes, and the results showed that hysteretic curve of the improved bolted joint was plump, but its maximum bearing capacity and displacement ductility were weaker than cast-in-place joint. Vidjeapriya et al. [11] proposed a precast concrete beam-column joint connected by bolts and angle steel with stiffener. The test results showed that its bearing capacity was 25% lower than that of cast-in-place joint, but it performed satisfactory ductility and energy dissipation capacity. Nzabonimpa et al. [12]...
proposed a new type of steel–concrete composite beam-column joint connected by high-strength bolts, and proved that the joint has good seismic performance through low cyclic loading experiments. However, the construction process of this connection method is complex. Ghayeb et al. [13] proposed a composite beam-column joint bolted with steel plate and steel tube. The test results indicated that the joint has good ductility, but low bearing capacity, and needs secondary pouring of concrete. These studies show that the prefabricated bolted connection joints have the defect of insufficient bearing capacity, but they can satisfy the seismic requirements through reasonable design.

As a passive energy dissipation device, viscoelastic dampers have been proved to effectively reduce wind-induced vibration response and explosion response [14–17]. In recent 20 years, scholars have increased the research on using viscoelastic dampers for structural earthquake resistance, and have repeatedly confirmed that they can effectively reduce the displacement response of structures in earthquakes. Xu et al. conducted long-term research on typical plate-type VE dampers [18], and proved that the VE dampers can effectively reduce the displacement response of building structures under earthquake by shaking table tests and hybrid shaking table test system [19–21]. Tubaldi [22] analyzed the dynamic characteristics of two adjacent buildings with different heights connected by VE dampers located at the top of the shortest building, and the preliminary design method of damper properties was proposed. Gong et al. [23] carried out shaking table tests on a three-story steel frame structure with VE dampers, and the effectiveness of the VE damper was proved. Mehrabi et al. [24] proposed a rotary rubber braced damper (RRBD), and the shaking table tests showed that RRBD can effectively reduce the displacement and acceleration response of steel frame. Li et al. [25] proposed a gear-driven rotation-amplified rubber viscoelastic damper (GRRVD) placed at the beam-column joint, and the test results showed that the control effect of the damper is remarkable. However, the existing viscoelastic dampers are relatively complicated in construction, production and installation. In most cases, they need to be effectively connected to the frame structure with the help of diagonal braces or herringbone steel supports [15,26], which may affect part of the building functions. Therefore, a prefabricated viscoelastic damping bolted joint (VDBJ) which connects the viscoelastic dampers with the prefabricated bolted beam-column joint by a built-in method is proposed in this paper. Because the viscoelastic dampers have the characteristics of both dissipating energy and providing additional stiffness for the structure, the defect of insufficient seismic performance of prefabricated bolted joint may be improved.

To verify the feasibility of this designing, three full-scale concrete beam-column joint specimens are manufactured, including one cast-in-place joint specimen and two prefabricated bolted joint specimens with and without dampers. Then, the seismic performance of the proposed beam-column joint, such as hysteresis characteristics, energy dissipation, displacement ductility, and stiffness degradation are analyzed and compared through high-frequency cyclic loading tests.

2. Proposed beam-column joint: Conformation and design theory

2.1. Conformation and energy dissipation mechanism of VDBJ

The specific connection method and detail structure of the prefabricated viscoelastic damping bolted joint (VDBJ) proposed in this paper is shown in Fig. 1. The precast beam and column are connected together with high-strength bolts after the embedded steel plates are butted. Four notches on the side of the beam end close to the embedded steel plate are used to place viscoelastic dampers. Four high-strength screws are reserved in advance on the embedded steel plate of the column, and their positions are corresponding to the four corner longitudinal reinforcements of the beam. The bolt hole at the bottom of the external restraint steel sleeve of the damper is connected with the high-strength screw, and then the pull rod of the damper is connected with the corner longitudinal reinforcement in the beam through the reinforcement connector.

As a prefabricated frame substructure, “VDBJ” is a semi-rigid joint with full dry connection. Under the action of seismic force, there will be a relative rotation angle between beam and column in the substructure, which means that the structure will be displaced in both horizontal and vertical directions. In this process, the longitudinal reinforcement in the beam will drive the pull rod of viscoelastic damper to slide, so as to squeeze the viscoelastic cushion to dissipate energy. Viscoelastic materials have hyperelastic properties, that is, their volume remains unchanged during compression. Therefore, when the viscoelastic cushion is compressed to fully contact with the inner wall of the externally restraint steel sleeve, as the relative rotation angle of the beam and column increases, the damper will become a rigid connection between beam and column to strengthen the joint after exceeding the design compressible displacement.

2.2. Mathematical model of initial rotational stiffness

For the overall lateral stiffness of a frame structure, the initial rotational stiffness of the substructure plays a decisive factor. Therefore, the design theory of the VDBJ is also realized based on the calculation theory of initial rotational stiffness. The component method is used to establish the initial rotational stiffness calculation model of the
proposed joint, and the following assumptions are made:

(1) The column is assumed to be a fixed rigid body;
(2) All components are in an elastic state with small deformation;
(3) Only the deformation in the tension zone of the joint is considered, and the deformation in the compression zone is ignored;
(4) The rotation center of the joint is located at the outermost edge of the embedded steel plate;
(5) The stiffness of viscoelastic damper in compressible state is ignored;
(6) Comply with the plane-section assumption.

The bending moment $M$ generated at the beam end can be equivalent to a pair of force couples, as shown in Fig. 2. The connection form of the proposed joint is similar to the flush end-plate bolted connection in steel structure. According to the European code for design of steel structures [27], the initial rotational stiffness $K$ of this joint can be expressed according to Eq. (1).

$$K = \frac{M}{\theta} = \frac{F_d d}{2 \sum K_i} = \frac{d^2}{\sum K_i}$$  \hspace{1cm} (1)

It is preliminarily considered that when the members are in the elastic stage, the total deformation $\delta$ in the tensile zone of the joint is mainly composed of the total deformation $\delta_s$ of the embedded steel plate at the beam end and the tensile deformation $\delta_b$ of the bolts, and it satisfies $\delta = \delta_s + \delta_b$. The deformation of bolted joint after stress can be equivalent to a simplified spring model, as shown in Fig. 3, where $K_s$ and $K_b$ are the equivalent spring stiffness of the embedded steel plate at the beam end and bolts in the core area of the joint, respectively. Because of the assumption of a fixed rigid body for the column, the shear deformation of its core area is not considered.

According to Hooke’s Law, the calculation equation of $K_s$ and $K_b$ can

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Fig. 1. Fabricated viscoelastic damping bolted joint (VDBJ).

Fig. 2. Rotational deformation principle diagram of VDBJ.

Fig. 3. Simplified spring model.
be obtained as follows:

$$K_1 = \frac{F_s}{\delta_1}$$  \hspace{1cm} (2)

$$K_2 = \frac{F_s}{\delta_2}$$  \hspace{1cm} (3)

Since the embedded steel plate and precast concrete beam are a whole, according to the force transfer mechanism, the tensile force $F_s$ of the embedded steel plate is equal to $F$. The total deformation $\delta_1$ of the embedded steel plate consists of bending deformation $\delta_{1b}$ and shear deformation $\delta_{1s}$. The embedded steel plate and adjacent bolts are equivalent to a simply supported beam element shown in Fig. 2. According to the knowledge of material mechanics, $\delta_{1b}$ and $\delta_{1s}$ can be obtained as follows:

$$\delta_{1b} = \frac{Fb(3S^2 - 4h^2)}{48E_bI}$$  \hspace{1cm} (4)

$$\delta_{1s} = \frac{FS}{4K_{eff}} = \frac{FS}{4G_bA_b} = \frac{0.65FS}{E_bb_t}$$  \hspace{1cm} (5)

where $S$ is the distance between adjacent bolts in the tensile area; $E_b$ is the elastic modulus of steel; $I$ is the section moment of inertia of embedded steel plate; $K_{eff}$ is the equivalent shear stiffness; $G_b$ is the shear modulus of steel, and $G_b = E_b / (1 + \nu), \nu = 0.3; A_b$ is the sectional area of embedded steel plate, and $A_b = b_t t_s$; $b_s$ and $t_s$ are the width and thickness of the embedded steel plate.

In this paper, it is considered that the concentrated force $F$ acts on the center of two adjacent bolts, i.e. $a = b$. The total deformation $\delta_1$ and bending stiffness $K_1$ of the embedded steel plate can be expressed by Eqs. (6) and (7).

$$\delta_1 = \delta_{1b} + \delta_{1s} = \frac{FS}{4E_bb_t} + \frac{0.65FS}{E_bb_t}$$  \hspace{1cm} (6)

$$K_1 = \frac{F}{\delta_1} = \frac{1}{4\frac{S^2}{E_bb_t^2} + \frac{0.65S^2}{E_bb_t^2}}$$  \hspace{1cm} (7)

Axial deformation of bolts is an important part of joint rotational deformation and the tensile stiffness $K_b$ of bolts can be calculated by the following Eq. (8) [27].

$$K_b = nE_bA_b \frac{L_b}{L_o}$$  \hspace{1cm} (8)

where $E_b$ is the elastic modulus of the bolts; $n$, $A_b$, and $L_b$ represent the number of bolts in tension area, the effective cross-sectional area of the bolts, and the effective length of the bolted connection, respectively. The value of $A_b$ can be obtained by checking the table in the specification. $L_o$ can be calculated by Eq. (9) as follows:

$$L_o = t_e + t_s + 2t_p + t_h$$  \hspace{1cm} (9)

where $t_e$ is the thickness of embedded steel plate of column; $t_s$ is the thickness of embedded steel plate of beam; $t_p$ is the thickness of washer; $t_h$ is the thickness of bolt head.

In addition, assuming the horizontal displacement of the beam end under the action of bending moment $M$ is $s$, and the linear distance between the force application point and the embedded steel plate is $H$, then

$$\theta = \frac{\delta}{d} = \frac{s}{H}$$  \hspace{1cm} (10)

The value of drift ratio when the viscoelastic cushion reaches the incompressible state can be determined by Eq. (10).
force to the column head with hydraulic jack according to the design axial pressure ratio of 0.15. Finally, adjusted the horizontal position of the actuator to basically fit the surface of the beam end, and then used the splint to fix it.

Due to the velocity dependence of viscoelastic damper [15], in order to give full play to the function of damper, the tests adopted high-frequency loading under displacement control. However, the concrete specimens were limited, and the multi-condition test under different frequencies cannot be completed. According to the previous test, it was found that setting the loading frequency as 2 Hz can minimize the displacement lag of the actuator during the cyclic loading under the condition of exerting the function of the VE damper. Therefore, the loading frequency was 2 Hz for research. The specific loading system is shown in Table 4. Stop loading when the bearing capacity dropped below 85% of the ultimate load. The loading system was tentatively set to 13 levels. If the bearing capacity does not drop below 85% of the ultimate load after the last level of loading, continue to increase the number of cycles according to the displacement increment of 10 mm.

### 4. Test results and analysis

#### 4.1. Failure mode

Since the loading rate was fast, observed the test phenomenon after each level of loading. The failure modes of the three beam-column joint specimens are shown in Fig. 8 (a)–(c). For two prefabricated specimens, there was no fracture damage at the weld between longitudinal reinforcements connected with the viscoelastic dampers in the beam. In the subsequent loading process, in addition to generated new cracks, the existing cracks were connected to each other, finally, a plastic hinge was

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**Table 1**

<table>
<thead>
<tr>
<th>Concrete strength</th>
<th>Average value of three groups of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cube compressive strength</td>
<td>Elastic modulus</td>
</tr>
<tr>
<td>$f_{cu}$/MPa</td>
<td>$E_c$/MPa</td>
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<tr>
<td>C40</td>
<td>47.80</td>
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</tbody>
</table>

**Table 2**

<table>
<thead>
<tr>
<th>Steel type (Specifications)</th>
<th>Average value of three groups of specimens</th>
<th>Tensile strength (MPa)</th>
<th>Elastic modulus (MPa)</th>
<th>Elongation after fracture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter/Thickness (mm)</td>
<td>Yield strength (MPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HRB400</td>
<td>8</td>
<td>419.47</td>
<td>565.04</td>
<td>$2.01 \times 10^5$</td>
</tr>
<tr>
<td>16</td>
<td>441.92</td>
<td>602.55</td>
<td>$1.93 \times 10^5$</td>
<td>26.6</td>
</tr>
<tr>
<td>18</td>
<td>500.16</td>
<td>623.17</td>
<td>$2.03 \times 10^5$</td>
<td>25.8</td>
</tr>
<tr>
<td>Q345B</td>
<td>20, 30</td>
<td>377.45</td>
<td>551.72</td>
<td>$2.07 \times 10^5$</td>
</tr>
<tr>
<td>Grade 8.8M30 high-strength bolt</td>
<td>Height of bolt head (mm)</td>
<td>Screw length (mm)</td>
<td>Washer thickness (mm)</td>
<td>Effective cross-sectional area (mm$^2$)</td>
</tr>
<tr>
<td>20</td>
<td>50</td>
<td>6.85</td>
<td>$2.07 \times 10^7$</td>
<td>561</td>
</tr>
</tbody>
</table>

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“Dynamic and static strain acquisition and analysis system” UT7816 was used for data acquisition in the tests. This paper mainly studied the strain of the longitudinal reinforcements connected with the viscoelastic dampers in the beam. The displacement meters were used to measure the curvature change of beam section and the shear deformation of joint area during loading. The specific layout of measuring points is shown in Fig. 7.
formed at the beam end. The ultimate bearing capacity of specimen SJ were 80.81 kN and 81.21 kN when the drift ratio reached +2.8% and −2.7%. When the drift ratio increased to 4.5%, the bearing capacity of the joint dropped below 85% of the ultimate bearing capacity. After peeled off the broken concrete, it was found that the longitudinal reinforcements at the beam end yielded seriously, and the damage state at this time is shown in Fig. 8 (a). During the whole loading process, there was no crack on the concrete surface of the column, which meets the design standard of “strong column and weak beam”.

(2) Specimen BJ: When the drift ratio was 1%, a gap of about 1 mm was formed between the embedded steel plate of the beam and the surface of the column. When the drift ratio increased to 2.2%, the gap reached 2 mm, and the concrete between the two notches on the side of the beam fell off a little. When the drift ratio increased to 3.5%, there was a separation of less than 1 mm at the connection between the beam and the embedded end plate. When the last level of loading was completed, the concrete at the

<table>
<thead>
<tr>
<th>Displacement amplitude (mm)</th>
<th>Rigidity (kN/mm)</th>
<th>Maximum damping force (kN)</th>
<th>Equivalent viscous damping ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.40</td>
<td>3.9679</td>
<td>3.815</td>
</tr>
<tr>
<td>11</td>
<td>1.42</td>
<td>15.6332</td>
<td>1.492</td>
</tr>
<tr>
<td>11.2</td>
<td>2.35</td>
<td>26.2969</td>
<td>1.038</td>
</tr>
</tbody>
</table>

Fig. 5. Mechanical performance test of viscoelastic dampers.

Table 3
Performance parameters of viscoelastic dampers.

Fig. 6. Schematic diagram of loading device.
damaged part of the beam end fell off obviously. However, the bearing capacity did not drop below 85% of the ultimate bearing capacity until two additional levels of loading were added according to 10 mm displacement increment. After stop loading, no crack was observed on the surface of the precast beam and the four high-strength bolts were basically intact. In the whole loading process, there was no damage to the precast column, and the excessive yield of embedded steel plate of the precast beam was the main reason of joint failure, as shown in Fig. 8 (b).

(3) Specimen VDBJ: When the drift ratio was 1.75%, a gap of less than 1 mm appeared between the embedded steel plate of the beam and the surface of the column. When the drift ratio increased to 2.2%, the concrete between the two notches on the side of the beam fell off obviously. When the drift ratio increased to 4.5%, the embedded steel plate of the beam in contact with the damper yielded obviously, and the dampers were jacked up by 1 mm. When the drift ratio increased to 5.2%, the bearing capacity has decreased to less than 85% of the ultimate bearing capacity. After stopping loading, it was observed that the viscoelastic damper was basically intact, except for a small amount of yield of the pull rod. The failure mode of specimen VDBJ was the same as that of specimen BJ, which was also the excessive yield of the embedded steel plate, as shown in Fig. 8 (c).

4.2. Hysteresis curve and energy dissipation capacity analysis

According to the test results, the force–displacement hysteresis curves of the three specimens are drawn as shown in Fig. 9 (a)–(c). The specific results are analyzed as follows:

(1) Under loading with high-frequency of 2 Hz, the hysteresis curve of specimen SJ was in a sharp shuttle shape before yielding. Pinching occurred in the later stage, and the curve shape tended to be bow. After stop loading, there was a large residual deformation, and the hysteresis loop was relatively plump.

(2) The hysteresis curve of specimen BJ was very plump, which showed the hysteresis characteristics of a typical semi-rigid joint. The unrecoverable deformation of the embedded steel plate at beam end led to great residual deformation in the later stage.

(3) Before the drift ratio reached 3.5%, the residual deformation of the specimen VDBJ was very small, which showed the characteristics of self-centering beam-to-column connection [4]. It was proved that the arrangement of viscoelastic dampers plays a role. At this time, the hysteretic curve was shuttle shaped. When the drift ratio was greater than 3.5%, the viscoelastic dampers formed multiple rigid connection after reached the design compressible displacement, resulting in increased stress on the embedded steel plate. It can be seen from the hysteretic curve that the residual deformation increased in the later stage and the curve gradually turned into a bow shape. These results was basically consistent with the effect of viscoelastic dampers changed from flexible connection to rigid connection after the design drift ratio reached 3.77%.

The envelope area of hysteresis curve represents the energy value absorbed by the specimen under cyclic loading, which can reflect the energy dissipation capacity of the specimen. The hysteresis loop area of three specimens under different drift ratios is shown in Fig. 9 (d).

It can be seen from the figure that the energy dissipation capacity of specimen SJ was better than that of specimen BJ and VDBJ under small displacement amplitude, because the rigid joint has higher rotational stiffness than the semi-rigid joint in elastic and elastoplastic stages. With the increased of the drift ratio to 3.5%, the energy dissipation capacity of the specimen VDBJ was gradually better than that of specimen SJ, which was mainly because the viscoelastic dampers have a strengthening effect on the rotational stiffness of the joint after changed from flexible connection to rigid connection. In addition, the energy dissipation capacity of specimen VDBJ was better than that of specimen BJ in the whole loading process for three main reasons as follows:

(1) The arrangement of viscoelastic dampers can effectively restrain the development of yielding of embedded steel plate around high-strength bolts.

(2) The viscoelastic cushion dissipated part of the energy in the compressible stage.

(3) Viscoelastic dampers strengthen the bearing capacity of the joint in later stage.
4.3. Skeleton curve and displacement ductility analysis

After each cyclic loading, the maximum load in the positive and negative directions was recorded by MTS computer. The skeleton curve of three specimens is drawn as shown in Fig. 10. The characteristic values of main loads and displacement ductility factors are summarized in Table 5, where $P_y$, $P_m$ and $P_u$ are yield load, peak load and ultimate load, respectively. The displacements corresponding to these three characteristic loads were $\Delta_y$, $\Delta_m$ and $\Delta_u$, respectively. In this paper, the point on the curve farthest from the connecting line between the origin and the peak point is defined as the yield point [31]. The ultimate displacement is defined as the displacement corresponding to the bearing capacity decreasing to 85% of the peak load.

The ratio of ultimate displacement to yield displacement is called the displacement ductility factor $\mu$.

$$
\mu = \frac{|+\Delta_u| + |-\Delta_u|}{|+\Delta_y| + |-\Delta_y|}
$$

Combining Fig. 10 and Table 5, it can be seen that the maximum bearing capacity of the specimen BJ was 31.0% and 26.6% lower than that of specimen SJ in the positive and negative directions, respectively. However, the maximum bearing capacity of specimen VDBJ was 33.5% and 29.9% higher than that of specimen SJ in the positive and negative directions, respectively. It showed that the arrangement of the visco-elastic dampers not only compensated for the negative effect of the four notches at the beam end on the bearing capacity of the specimen, but also greatly improved the bearing capacity of the bolted joint. Among the three beam-column joint specimens, the displacement ductility factor of specimen VDBJ was lower than specimen BJ, but higher than specimen SJ, which proved that the proposed VDBJ in this paper can be used in seismic regions.

Fig. 8. Failure modes of three specimens.

(a) Specimen SJ

(Failure modes: Plastic hinge at beam end and yield of longitudinal reinforcements in beam)

(b) Specimen BJ

(Failure modes: Yield of embedded steel plates on both sides of beam end)

(c) Specimen VDBJ

(Failure modes: Yield of embedded steel plates on both sides of beam end)
4.4. Stiffness analysis

The theoretical values of initial rotational stiffness can be calculated by Eqs. (1)-(9). The secant stiffness when the joint bending moment is 2/3 of the full plastic bending capacity of the beam section is taken as the experimental value of the initial rotational stiffness. The comparison results between theoretical values and experimental values are shown in Table 6.

For specimen BJ, the theoretical value of initial rotational stiffness was larger than the experimental value, and the ratio was 1.28. This was because the four notches at the beam end have a weakening effect on the initial rotational stiffness of the joint. In order to obtain accurate results, the theoretical value needs to be multiplied by a reduction coefficient, which can be obtained only by multiple groups of tests, and this paper will not study it for the time being.

However, the ratio of theoretical value to experimental value for specimen VDBJ was 0.94, which was very close to 1. This proved the arrangement of viscoelastic dampers makes up for the weakening effect caused by the notches at the beam end, and the calculation model can be used to calculate the initial rotational stiffness of the proposed joint.

Stiffness degradation is a parameter characterizing secant stiffness attenuation. In this paper, the secant stiffness is defined by the slope of the connecting line between the peak points at both ends of the hysteretic loop. The variation curve of secant stiffness of three specimens under each cycle loading as shown in Fig. 11. The stiffness of specimen

![Fig. 9. Hysteresis curves and energy dissipation.](image1)

![Fig. 10. Comparison of skeleton curves.](image2)
longitudinal reinforcements in the beam have yielded, and the strain frequency was set to 0.001 s. When the drift ratio reached 1%, the strain value remained constant at 35,413.974 με. When the drift ratio increased to 2.75%, the strain deformation of the joint core of precast column was no significant change, and the strain values of the reinforcements in the precast column also had no obvious fluctuation. It is proved that under this connection mode, the shear deformation of the column can be ignored, and the relevant measuring points used to measure this part can be cancelled. In the case of high-frequency loading, the data acquisition of strain is difficult to be effectively controlled, so it is suggested to adopt a higher acquisition frequency.

4.5. Summary and suggestions on strain and displacement measurement

The base specimen SJ was loaded first, and the data acquisition frequency was set to 0.001 s. When the drift ratio reached 1%, the longitudinal reinforcements in the beam have yielded, and the strain value was 22,225.29 με. When the drift ratio increased to 2.75%, the strain gauges have failed, and the strain value remained constant at 35,413.974 με during subsequent loading. The variation curve of strain gauge before failure as shown in Fig. 12.

After the loading of specimen SJ was completed, it was found that the amount of data was too large, so the acquisition frequency was adjusted to 0.1 s. It can be seen from the longitudinal reinforcement strain variation curve of specimen VDBJ in Fig. 12, when the loading frequency was fixed at 2 Hz, the loading rate accelerated with the increased of drift ratio, and lower acquisition frequency caused data acquisition lag. Therefore, only positive loading had normal strain value. The longitudinal reinforcement in the beam connected with viscoelastic damper did not reach the yield strain value during the whole loading process.

The readings of displacement meters for measuring shear deformation of the joint core of precast column were no significant change, and the strain values of the reinforcements in the precast column also had no obvious fluctuation. It is proved that under this connection mode, the shear deformation of the column can be ignored, and the relevant measuring points used to measure this part can be cancelled. In the case of high-frequency loading, the data acquisition of strain is difficult to be effectively controlled, so it is suggested to adopt a higher acquisition frequency.

5. Conclusions

To solve the problem of insufficient bearing capacity of precast concrete bolted connection joint, the prefabricated viscoelastic damping bolted joint (VDBJ) is proposed in this paper. Through the comparative tests of three full-scale beam-column joint specimens under the frequency of 2 Hz cyclic loading, the following conclusions are obtained:

(1) Specimen BJ exhibited the best ductility, but its energy dissipation capacity and bearing capacity are weaker than the other two specimens. It is consistent with the test results in the references, and the excessive yielding of the embedded steel plate at the beam end is the main failure mode of specimen BJ and VDBJ.

(2) Before the drift ratio reaches 3.5%, the energy dissipation capacity of specimen VDBJ is slightly weaker than specimen SJ, but after that, specimen VDBJ shows stronger energy dissipation capacity. The displacement ductility and maximum bearing capacity of specimen VDBJ are better than specimen SJ, and the maximum bearing capacity of specimen VDBJ is 33.5% and 29.9% higher than specimen SJ in the positive and negative directions, respectively. It proves the feasibility and superiority of the design.

(3) The arrangement of viscoelastic dampers effectively restrains the yield deformation of embedded steel plate around high-strength bolts, and makes up for the weakening effect of four notches at the beam end on the bearing capacity of the specimen. The viscoelastic dampers not only participate in energy dissipation, but also provide additional stiffness for the beam-column joint. These are the main reasons for the improvement of the beam-column joint performance.

(4) Using this built-in method to connect the viscoelastic dampers with the beam-column joint can avoid affecting the building functions. The longitudinal reinforcements connected with viscoelastic dampers are always in elastic state, which effectively controls the appearance and expansion of cracks on the surface of concrete beam. The concrete beam is basically not damaged in the whole test process, and can be replaced quickly after earthquakes.
(5) The simplified calculation model of initial rotational stiffness can be used to calculate the initial rotational stiffness of the proposed joint.

(6) Although the prefabricated concrete structure with pure bolted connection has many advantages, its application in mid and high-rise buildings is almost non-existent. However, the test results prove that the vibration control effect by means of VE dampers is of great significance to increase the application of this type of structure in engineering.

Declaration of Competing Interest
The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgments
This work was supported by the National Natural Science of China (Grant Number), the National Key R&D Programs of China (Grant Number), the Program of Chang Jiang Scholars of Ministry of Education, the National Natural Science of China (Grant Number), the Science Discovery Award. These supports are acknowledged gratefully.

References