Study on Seismic Performance of Connecting Prefabricated Coupling Beam and Shear Wall with Equivalent Bars

Bo Zheng, Ming Li, Xuyi Liu, Zhen Wang, and Chunyan Wang

School of Civil Engineering, Shenyang Jianzhu University, Shenyang 110168, Liaoning, China

Correspondence should be addressed to Ming Li; 297531635@qq.com

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This study proposes a new method for the connection of the prefabricated coupling beam and shear wall to solve the problems in the construction of assembled frame-shear wall structures. To investigate the seismic properties of this new type of assembled coupling beam, experiments with 2 groups of test pieces under low-period cycling loads were completed. Furthermore, this study studied some important influence factors. Results show that this novel coupling beam has a similar performance to the cast-in-place coupling beam. The bearing capacity and energy dissipation capacity of cast-in-place structures increase with the increase in end block thickness and the ductility inversely, yet the bearing capacity, seismic energy dissipation capacity, and ductility of assembled coupling beam structure decrease with the increase in end block thickness. Numerical simulation shows that increasing the area of equivalent steel bars will increase its yield load but will also reduce the seismic energy dissipation capacity and ductility. The thicker the shear wall thickness, the higher specimen’s bearing capacity, the stronger the seismic energy dissipation capacity, and the worse the ductility is.

1. Introduction

Assembled structure system is strongly advocated due to its advantages such as fast construction speed, high degree of standardization, energy saving, and environmental protection [1–4]. The current prefabricated structural system has been widely used in housing construction [5, 6], integrated pipe gallery [7], and bridges [8].

The connection and structure between prefabricated components play an important role in prefabricated concrete structures. The connection structure between prefabricated components not only affects the mechanical properties of the structures but also has a direct impact on the construction and cost, so it has been widely studied [9, 10]. Paulay et al. [11] first studied the seismic performance of beam-column connections. Breccolotti et al. [12] studied the seismic performance of prefabricated beam and column connection connections. The prefabricated beams and columns are connected by reserved steel bars and steel fibers. The test results show that the beam-column components connected by this technology have very similar structural properties to the cast-in-place beam-column components. Zhang et al. [13] proposed a new type of bolted web friction device for prefabricated post-tensioned self-centering beam-column connection. The test results indicate that the connection has the same satisfactory self-centering capacity compared with that of the common bolted web friction device connection. Liu et al. [14] proposed a new type of energy-consuming fabricated beam-column connection nodes, which can not only provide good structural integrity but also ensure that the plastic hinge moves away from the column edge. The test results show that the new type of joint can achieve a beam-hinge mechanism and prevent joint shear failure.

However, the study performed on the assembled concrete frame-shear structures is not much, especially the research aimed at the assembly method of connecting the shear wall and coupling beam of it is lacking. Most of the studies mainly focus on the problems of how to connect prefabricated shear wall with column shear wall with beam or beam with column [15–22]. The thickness of the coupling beam in the frame-shear structure is generally the same as...
the shear wall, resulting in a smaller thickness and a smaller cross-sectional area of the coupling beam. At the same time, the arrangement of frame beams, columns, and shear walls also greatly limits the assembly space for assembling the coupling beam. For the above two reasons, it is not suitable for placing sleeves in the coupling beam and shear wall to connect the bars in them like the method of connecting beam with column [23]. At present, most of the coupling beams are connected with shear walls by the cast-in-place construction method in the assembled structure [24], which leads to the low assembly rate and slow construction speed.

For the above problems, this research proposes a new method to solve them. To realize the connection between the precast shear wall and the precast coupling beam, this study reserves the shear wall holes and the coupling beam holes in the precast shear wall and the precast coupling beam, respectively, and puts equivalent steel bars in the reserved holes of the coupling beam. After lifting the prefabricated beams to the assembly position, the shear wall holes and the beam holes are corresponding one by one, and then, equivalent steel bars are inserted into the prefabricated shear wall and microexpansion mortar is injected, and then, the grout is poured into the assembly area using U-shaped template to form a whole structure. As a result, the construction process becomes more efficient and convenient.

This study aims to investigate the performance of this novel structure and find out the mechanism of influence factors. In the first part of this study, the seismic performance of this new type of connection was experimentally studied, and 2 sets of 4 coupling beam specimens were completed with low-cycle repeated loading tests. Then, in the second part, the finite element software WCOMD was used to simulate the load-bearing process of the components and analyze its mechanical influencing factors.

2. Experimental Method

2.1. Test Specimens. Four specimens of 2 groups were introduced for the experiment, each group including 1 cast-in-place specimen and 1 fabricated specimen. The specimens were designed with reference to the literature [25, 26]. The research was aimed at the properties and failure forms of the fabricated coupling beams. In order to facilitate the experiment, only part of the shear wall area (end block) was involved in this study, as shown in Figure 1. The specimens were composed of end blocks and connecting beams, as shown in Figure 2. The first group includes XJ1 (cast-in-place) and ZP1 (assembled). The thickness of the end block of them is greater than the width of the connecting beams, which is used to study the failure of the end of the connecting beam without developing to the end block. The second group includes XJ2 (cast-in-place) and ZP2 (assembled). The coupling beam and the end block of them have the same width, which is used to study the damage from the end of the coupling beam to the end block. The dimensions and drawings of each specimen are shown in Table 1 and Figure 2, respectively. The vent hole is used to discharge the gas from the hole during grouting.

All the section height and thickness of the coupling beam are 420 mm and 200 mm separately, and all the length of the end blocks is 1,700 mm.

In order to realize the connection between the precast shear wall and the precast coupling beam, this study reserves the shear wall holes and the coupling beam holes in them, respectively, and first put equivalent bars in the reserved holes of the coupling beam. The end of the equivalent bar is exposed. The prefabricated coupling beam is hoisted to the assembly position until the reserved holes in the shear wall correspond to the reserved holes in the coupling beam. Then, the equivalent bars are pulled out and inserted into the prefabricated shear wall and U-shaped template between the coupling beam and the shear wall. At last, grout is poured into the template until the holes and the connection area are filled in. The area of equal-strength steel bars shall not be less than the area of the longitudinally stressed steel bars of the connecting beams to ensure the transmission of the bending moment between the prefabricated connecting beams and the prefabricated shear wall. Among them, the grouting strength was 80 MPa, the model of steel bars was HRB400, and C25 concrete was used for casting.

2.2. Instruments and Loading Scheme. The test loading device was designed according to reference [25]. This loading method was able to simulate the actual force of the connecting beam in the shear wall limbs. In the test, the electro-hydraulic servo program-controlled structure test system was used to repeatedly load the specimens at a low cycle, and the lower end block was used as the ground beam and fixed with screws and jacks. The specific design test loading device during the test is shown in Figure 3(a), and the actual loading device is shown in Figure 3(b).

The loading method was according to the “Building Seismic Test Method Regulations JGJ/T101-2015.” Before the specimen reaches the yield stage, the load controlling method was used. The load value of each level was adopted as 1/10 of the estimated bearing capacity, and each load only completes one loading cycle (including forward loading and reverse loading). After the specimens reached the yield load, displacement-controlled loading was used instead, which was graded according to 1Δ, 2Δ, 3Δ, and 4Δ stages (Δ is the yield displacement), and each level of displacement completes two loading cycles. When the concrete protective layer was severely compressed and peeled off, and the bearing capacity of specimens dropped below 85% of the ultimate bearing capacity, specimens were considered to be broken or invalid, and tests were terminated.

3. Results and Discussion

3.1. Form of Destruction. Figure 4(a) shows the destruction of XJ1 and ZP1, and Figure 4(b) shows the destruction of XJ2 and ZP2. It can be seen that the cracks of the cast-in-place specimens (XJ1 and XJ2) were mainly oblique cracks, and the fabricated specimens (ZP1 and ZP2) were mainly vertical cracks, which appear at the junction of the grouting area and
Решение 1

Решение 2: составление

Решение 3: заполнение и отвержение

Объединяемый балка

Илигнальные балки

Вентиляционное отверстие

Плита Кузьки

Диаграмма 1: Схематическое изображение объединяемой балки.

Решение 2: продолжение.

Решение 3: продолжение.
the connecting beam. This is because the strength of the grouting material used in the fabricated specimens was higher than the strength of the concrete on the specimen, resulting in stress concentration at the conjunction, causing vertical cracks along with the interface.

The test results showed that the failure process of the cast-in-place specimens and the assembled specimens were basically the same, and the end block thickness had little effect on this (Table 2). The cracking load of the cast-in-place specimen and the assembled specimen was roughly the same. When the displacement was loaded to 18 mm, the concrete of all specimens began to fall off, and the bearing capacity of the specimens gradually decreased. When the displacement was loaded to 20–36 mm, the maximum crack of the test piece developed to 5 mm, a large amount of concrete fell off, and the bearing capacity of the test piece sharply dropped. Generally speaking, the ductility of cast-in-place specimens was better.

### 3.2. Load-Strain Curve

The load-strain curve of the reinforced steel bar of each specimen is shown in Figure 5. XJ1 reached yield earlier than ZP1, and XJ2 reached yield earlier than the ZP2. In terms of strain, the ductility of...
cast-in-place specimens (XJ1 and XJ2) was better than that of assembled specimens (ZP1 and ZP2). This is because the cross-sectional area of the steel bars of the assembly specimens was slightly larger than the cross-sectional area of the longitudinally stressed bars of the cast-in-situ specimens, which made the concrete be crushed with only narrow flaws, resulting in relatively lower ductility of the assembled specimens.

3.3. Hysteresis Curves. As shown in Figure 6, the shapes of the hysteresis curve of the cast-in-place test piece and the assembled test piece are approximately the same. With the increase in the loading displacement, compared with the assembled specimen, the slope of the hysteresis loop of the cast-in-place specimen slightly decreases, and the stiffness degrades faster. The energy dissipation capacities of the cast-in-place test piece and the assembled test piece are similar. In addition, by comparing Figures 6(a) and 6(b), it can be seen that the stiffness degradation rate of the assembled test piece with the thickness of the end block greater than the thickness of the connecting beam has a faster rate of stiffness degredation, smaller ultimate displacement, and poorer ductility. This is because the thickness of the end block for assembly specimens is greater than the thickness of the coupling beam, which is subjected to a stronger restraint effect.

3.4. Load-Displacement Curve. As shown in Figure 7, compared with the cast-in-place specimens (XJ1 and XJ2), the assembly specimens (ZP1 and ZP2) have smaller yield loads and peak loads. Among them, the yield load and peak load of XJ1 are 14.49% and 19.20% larger than those of ZP1, and the yield load and peak load of XJ2 are 11.14% and 3.02% larger than those of ZP2, respectively. The peak load of the assembly test piece (ZP1) whose end block thickness is greater than the thickness of the connecting beam is slightly smaller than that of the cast-in-situ test piece, and the peak load of the assembly test piece (ZP2) with the end block thickness equal to the thickness of the connecting beam is approximately the same as that of the cast-in-situ test piece.

4. Finite Element Simulation and Parameter Research

Numerical simulation methods are usually used to investigate some factors’ influence [27–32]. Shishegaran et al. [33, 34] used the finite element method to analyze the influence factors of the load-bearing capacity and the stiffness of one kind of novel strengthened RC beams. Shishegaran et al. [35] also applied the finite element method in finding the most effective parameters relating to the resistance of reinforced concrete connections. Bigdeli et al. [36] used nonlinear finite element analysis and surrogate models to
study the effects of several parameters, including the compressive and tensile strength of concrete, the size of the longitudinal reinforcement bar, the transverse bar diameter, and the internal water pressure on the performance of reinforced concrete tunnel under internal water pressure.

In this research, the finite element analysis software WCOMD developed by the University of Tokyo Concrete Laboratory was introduced for modeling. The software has a high accuracy for the two-dimensional nonlinear dynamic and static analyses of reinforced concrete structures with
cracks by using high-precision constitution rules. WCOMD software also integrates the constitutive relations of many materials and a large number of solving modules, which can solve various aspects including structural engineering. The convergence algorithm is a combination of the Newton-Raphson algorithm and the modified Newton-Raphson method. WCOMD uses a two-dimensional plane strain fiber model and a yield criterion for stress deflection under multiaxial action.

4.1. Description of Models. This research adopted the concrete uniaxial skeleton curve constitutive model of the “Road and Bridge Instruction Book” (Figure 8). Among them, \( \varepsilon'_{\text{peak}} \) was calculated by

\[
\varepsilon'_{\text{peak}} = 447.2 \sqrt{f'_{c}} \times 10^{-6}.
\]  

Here, \( \varepsilon'_{\text{peak}} \) represents the peak strain, \( f'_{c} \) represents the ultimate strength of concrete under uniaxial compression, and its value can be obtained through material tests.

It is generally believed that drawing forces will be completely borne by the steel bars rather than concrete. However, the concrete and steel bars between adjacent cracks will continue resisting tension, and the tensioned concrete cannot immediately withdraw, and there will still be some residual strength. In order to consider the tension strengthening of concrete and the interaction relationship with steel bars, the tensile stiffness strengthening factor C was introduced to express the tensile softening of concrete (Figure 9), and the tensile strength of concrete can be calculated by

\[
\sigma_t = f_t \varepsilon_{tu} \frac{\varepsilon_t}{\varepsilon_{tu}}
\]

The constitutive model of the steel bar material is shown in Figure 10. In order to reflect the interaction of the concrete attached to the surface of the steel bar, the hardening model was used to consider the stress of the steel bar where the crack developed. Before the concrete starts cracking, the steel bars always remained in the elastic stage. After the steel bar yielded, the hardening model was adopted. This hardening model took into account the average stress of the section at yielding, the strength of the concrete, the reinforcement ratio, the angle between the steel bar, and the crack and other parameters (Figure 10).

The interaction between two precast elements in the assembled specimens was coupled, which means all points...
on the surface share the same displacement. The compression constitutive model of grout materials is as follows:

\[
\begin{align*}
\sigma_c &= 2.17 \frac{\varepsilon_c}{\varepsilon_0} - 1.34 \left( \frac{\varepsilon_c}{\varepsilon_0} \right)^2 + 0.17 \left( \frac{\varepsilon_c}{\varepsilon_0} \right)^3, \quad \frac{\varepsilon_c}{\varepsilon_0} \leq 1.0, \\
\frac{\sigma_c}{f_c} &= \frac{\varepsilon_c}{\varepsilon_0} - 1.56 \left( \frac{\varepsilon_c}{\varepsilon_0} - 1 \right)^2 + \frac{\varepsilon_c}{\varepsilon_0}, \quad \frac{\varepsilon_c}{\varepsilon_0} > 1.0.
\end{align*}
\]

Equation (3) considers that the elastic modulus of grouting material is smaller than that of concrete of the same strength, and the strength of grouting material decreases faster after the peak point. The tensile constitutive relation references the curve of concrete materials.

For visually reflecting the development of cracks in the specimen, this study uses an analysis method that combines the dispersive crack model and the dispersive crack model. The dispersive crack model was introduced to consider the generation of cracks. When the maximum standard stress of the element reached the cracking critical force, the crack generated and a discontinuous displacement field appeared in the element. A microscopic diffuse crack model was used for the boundary where the crack was locally discontinuous and had a greater impact on the development of the crack, and the average stress-strain relationship of the element was introduced to consider the plastic behavior of the interaction zone between the steel bar and the concrete outside of the crack.

The numerical simulation uses rectangular grid elements, and the grid division of the specimen is shown in Figure 11. The finite element model includes shear wall at both ends of the connecting beam, frame beams, shear wall connecting beams, and spacers. A total of 14 finite element models have been established in this research. Among them, M-XJ1, M-XJ2, M-ZP1, and M-ZP2 were used for the comparison between experiments and finite element simulations, and the remaining models were used to affect the mechanical properties of fabricated coupling beams. Parameters are analyzed. The influencing parameters include the following: shear wall thickness \( t \) at both ends of the connecting beam, grouting strength \( f_c \), the ratio of the cross-sectional area of the equivalent steel to the original steel, and the length of the grouting zone \( L_0 \). The specific parameters of the model size are shown in Table 3.

DCC1, DPC2, DCC9, and DPC10 were used to study the effect of the end block thickness \( t \) on the mechanical properties of fabricated coupling beams; DPC2, DPC3, and DPC4 were used to study the effect of equivalent steel bar section area ratio \( y \) on the mechanical properties of fabricated coupling beams; DPC2, DPC5, and DPC6 were used to study the influence of grouting strength \( f_c \) on the mechanical properties of fabricated coupling beams; DPC2, DPC7, and DPC8 were used to study the effect of grouting zone length \( L_0 \) on the mechanical properties of fabricated coupling beams. To accurately and quickly calculate, the mesh was divided into different sizes according to the importance of different components. The mesh was subdivided into the important parts, while the number of seeds was reduced in the unimportant parts. To ensure the uniform structure of the specimen model, the rectangular mesh was generally used. The mesh’s size affects the calculation accuracy, so this study studied the mesh sensitivity by comparing the error between test results and calculation results of models with different divisions. When the grid size increased by one time, the error increases by about 30%; when the grid size decreased by one time, the calculation process did not converge, and the accuracy was almost not improved with a maximum difference of less than 10%.

4.2 Results and Discussion

4.2.1 Comparison of Numerical Simulation and Experimental Results

(1) Load-Displacement Curves. As shown in Figure 12, the trends of the load-strain curves of the test and the numerical simulation were basically the same. The load-strain curve of the XJ1 had an insignificant descending section, while the declining section of the curve in the simulated skeleton curve was obvious and relatively gentle. From the simulation results, the maximum error of the yield load between the test and the numerical simulation of the specimen was 17.83%, the maximum error of the peak load was 12.73%, and the maximum error of the ductility coefficient was 36.2%.
The errors were within a reasonable error range, and the numerical simulation was capable of simulating the performance of the test piece better.

(2) **Structure Cracking.** Due to the crack development process of each specimen in the experiment was roughly the same, this section took specimens XJ1 and ZP1 as examples.

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Table 3: Detailed information on finite models.

| Code  | Height $H$ (mm) | Thickness $T$ (mm) | Span $L/H$ | End block thickness $t$ (mm) | Grouting strength $f_c$ (MPa) | Equivalent area ratio ($\gamma$) | Length of assembly area $L_a$ (mm) |
|-------|----------------|-------------------|------------|-----------------------------|-------------------------------|---------------------------------|----------------------------------|
| DCC1  | 420            | 200               | 3          | 400                         |                               |                                 | 40                               |
| DPC2  | 420            | 200               | 3          | 400                         | 60                            | 1                               | 40                               |
| DPC3  | 420            | 200               | 3          | 400                         | 60                            | 0.85                            | 40                               |
| DPC4  | 420            | 200               | 3          | 400                         | 60                            | 1.15                            | 40                               |
| DPC5  | 420            | 200               | 3          | 400                         | 80                            | 1                               | 40                               |
| DPC6  | 420            | 200               | 3          | 400                         | 100                           | 1                               | 40                               |
| DPC7  | 420            | 200               | 3          | 400                         | 60                            | 1                               | 20                               |
| DPC8  | 420            | 200               | 3          | 400                         | 60                            | 1                               | 60                               |
| DCC9  | 420            | 200               | 3          | 200                         |                               |                                 | 40                               |
| DPC10 | 420            | 200               | 3          | 200                         | 60                            | 1                               | 40                               |

Figure 12: Comparison of experimental and numerical simulation results. (a) Specimen XJ1. (b) Specimen ZP2. (c) Specimen XJ2. (d) Specimen ZP2.
to compare the experimental and numerical simulation results of the cast-in-place and assembled specimens.

As shown in Figure 13, the crack development of the specimen in the test and the numerical simulations was basically the same. In the test specimens XJ1 and ZP1, the first shear crack that appeared at the beam end position was 60 kN. In the simulation, the corresponding cracking loads were 65 kN and 60 kN, respectively; during the test, when the displacement was loaded to 32 mm and 20 mm, the concrete of XJ1 and ZP1 specimens began to massively fall off, and the bearing capacity of the specimens sharply dropped; likewise, the loading displacements corresponding to the massive fall of concrete of the M-XJ1 and M-ZP1 specimens were 23.7 mm and 28 mm, respectively. M-XJ1 reached the specimen failure earlier than XJ1, and compared to ZP1, M-ZP1 failure was later.

Therefore, it can be considered that the numerical simulation was capable of simulating the stress and failure process of the specimens. In the next part, this research further investigates the parameters that affected the mechanical properties of the prefabricated coupling beam structure.

4.2.2. Analysis of Major Parameters

(1) End Block Thickness. As shown in Figures 14(a) and 14(b) and Tables 5 and 6, the yield load, maximum load, and energy dissipation coefficient of DCC9 were 19.16%, 4.85%, and 26.77% lower than those of DCC1, respectively. The ductility coefficient of DCC9 was 43.76% lower than that of DCC1; compared with DPC2, the yield load of DPC10 was 11.94% greater, the maximum load was 10.18%, the ductility coefficient was 21.45%, and the energy dissipation coefficient was 11.39%. Among them, the thickness of the shear walls at both ends of the specimens DCC1 and DPC2 were 400 mm, the thicknesses of the shear walls at both ends of DCC9 and DPC10 were 200 mm, DCC1 and DCC9 were cast-in-place specimens, and DPC2 and DPC10 were fabricated specimens. The equivalent damping coefficients in Table 6 are be calculated by

$$h_c = \frac{1}{2\pi} \frac{S_{(ABC+CDA)}}{S_{(OBF+ODE)}}$$

Here, $S_{(ABC+CDA)}$ represents the energy consumed by the component (i.e., single-cycle hysteretic energy consumption), and $S_{(OBF+ODE)}$ represents the energy absorbed by the component in the elastic phase during cyclic loading.

Therefore, the thickness of the end block had greater impacts on the mechanical properties of the cast-in-place and assembled single-link beams. The thicker the beam end shear wall was, the higher the bearing capacity of the cast-in-place single-link beams, the stronger the seismic energy consumption, and the worse the ductility; the smaller the bearing capacity of the assembled single-link beams, the worse the seismic energy consumption and ductility.

The cast-in-place and assembly specimens had obvious stiffness degradation during the entire simulation process and concentrated in the early stage of loading. The stiffness degradation from cracking to yield was more obvious, but the stiffness degradation slowed down when the rebars began to yield. As shown in Figure 15(a), the initial stiffness of DCC9 was 56.48% larger than that of DCC1. After reaching the yield phrase, the stiffness of the two were almost the same, and the stiffness of DCC9 decreased faster than DCC1.

Therefore, for the cast-in-place specimens, the thinner the end block, the greater the initial stiffness of the connecting beam, and the faster the stiffness decreased. For fabricated specimens, the thinner the end block, the smaller the initial stiffness of the connecting beams, and the slower the stiffness decreased.

(2) Equivalent Area Ratio. Load-displacement curves of DPC2, DPC3, and DPC4 are shown in Figure 14(c), in which the diameters of the equivalent steel bar of DPC2, DPC3, and DPC4 were 16 mm, 14 mm, and 18 mm respectively, and the cross-sectional area ratio of the equivalent steel bars was 1 : 0.85 : 1.15.

With the increase in the diameter of the equivalent steel bars of the coupling beams, the yield and maximum load of the specimens increased by 10.99% and 9.11%, respectively; the ductility coefficient and energy dissipation coefficient of the specimen decreased by 37.95% and 9.05%, respectively. Therefore, the larger the diameter of the equivalent steel bar of the coupling beams, the larger the yield and maximum

| Table 4: Comparison of experimental and numerical simulation results. |
|---------------------------------------------------------------|
| Yield load (kN) | Peak load (kN) | Yield displacement (mm) | Peak displacement (mm) |
|-----------------|---------------|------------------------|------------------------|
| XJ1             | 55.82         | 76.03                  | 2.90                   | 31.86                  |
| M-XJ1           | 54.29         | 66.35                  | 3.94                   | 30.96                  |
| Error (%)       | 2.74          | 12.73                  | 26.4                   | 2.82                   |
| ZP1             | 47.73         | 61.43                  | 1.67                   | 28.25                  |
| M-ZP1           | 46.01         | 58.24                  | 2.41                   | 27.37                  |
| Error (%)       | 3.6           | 5.19                   | 30.71                  | 3.12                   |
| XJ2             | 49.03         | 60.51                  | 1.93                   | 24.09                  |
| M-XJ2           | 46.41         | 62.96                  | 2.95                   | 23.47                  |
| Error (%)       | 5.34          | 3.89                   | 34.58                  | 2.57                   |
| ZP2             | 43.57         | 58.68                  | 3.28                   | 36.44                  |
| M-ZP2           | 53.09         | 66.54                  | 3.64                   | 30.53                  |
| Error (%)       | 17.83         | 11.81                  | 9.89                   | 16.22                  |

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Figure 13: Cracking of the specimen. (a) XJ1. (b) ZP1.

Figure 14: Continued.
load, and the weaker the seismic energy dissipation capacity and ductility.

In addition, as shown in Figure 15(c), DPC2, DPC3, and DPC4 all have obvious stiffness degradation during the loading process, and the stiffness degradation was concentrated in the early loading stage. The initial stiffness of DPC3 was 28.54% greater than that of DPC2. The initial stiffness of DPC4 was 22.9% greater than that of DPC2. The stiffness of DPC4 decreased the fastest. The rate of decrease in the stiffness of DPC2 was slightly greater than that of DPC3.

Table 5: Simulation results of the specimens.

| Specimen code | Yield load (kN) | Maximum load (kN) | Failure load (kN) | Yield displacement (mm) | Ultimate displacement (mm) | Ductility coefficient |
|---------------|-----------------|-------------------|-------------------|--------------------------|----------------------------|-----------------------|
| DCC1          | 54.29           | 66.35             | 56.40             | 3.94                     | 30.96                      | 5.99                  |
| DPC2          | 46.01           | 58.24             | 49.50             | 2.41                     | 27.38                      | 7.58                  |
| DPC3          | 45.84           | 57.21             | 48.63             | 1.85                     | 24.26                      | 11.54                 |
| DPC4          | 50.88           | 62.42             | 53.06             | 2.47                     | 22.87                      | 7.13                  |
| DPC5          | 48.76           | 59.41             | 50.50             | 2.78                     | 23.02                      | 7.83                  |
| DPC6          | 46.35           | 57.07             | 48.51             | 2.49                     | 23.72                      | 7.85                  |
| DPC7          | 43.94           | 56.95             | 48.41             | 2.39                     | 24.06                      | 10.08                 |
| DPC8          | 46.10           | 56.03             | 47.63             | 1.98                     | 18.07                      | 8.44                  |
| DCC9          | 43.89           | 63.13             | 53.66             | 1.83                     | 23.68                      | 10.65                 |
| DPC10         | 52.25           | 64.84             | 55.11             | 3.11                     | 30.31                      | 9.65                  |

Table 6: Energy dissipation capacity of the specimen.

| Specimen code | Equivalent viscous damping coefficient | Energy dissipation coefficient |
|---------------|----------------------------------------|------------------------------|
| DCC1          | 0.265238                               | 1.666539                     |
| DPC2          | 0.254105                               | 1.596587                     |
| DPC3          | 0.236544                               | 1.486253                     |
| DPC4          | 0.231109                               | 1.452103                     |
| DPC5          | 0.142186                               | 0.893381                     |
| DPC6          | 0.258646                               | 1.625122                     |
| DPC7          | 0.26481                               | 1.663851                     |
| DPC8          | 0.25882                               | 1.626212                     |
| DCC9          | 0.19137                               | 1.202412                     |
| DPC10         | 0.286758                               | 1.801756                     |
Figure 15: Stiffness degradation curves. (a) DCC1 and DCC9. (b) DCC2 and DCC10. (c) Equivalent area ratio as a variable. (d) Grouting strength as a variable. (e) Assembly area length as a variable.
After reaching the yielding stage, three stiffness of the test piece were approximately the same. Therefore, the larger the cross-sectional area of the steel bar, the faster the stiffness degraded.

(3) The Influence of Grout Strength. Figure 14(d) depicts the load-displacement curves of DPC2, DPC5, and DPC6, where the grout strength of DPC2 was 60 MPa, the grout strength of DPC5 was 80 MPa, and the grout strength of DPC6 was 100 MPa. As shown in Tables 5 and 6 and Figure 14(d), the yield load, maximum load, ductility coefficient, and energy dissipation coefficient of the specimen were very small. The change in the strength of the grouting material in the assembly area had no significant effect on the mechanical properties of the assembled coupling beam specimens.

As shown in Figure 15(d), the initial stiffness of DPC2 was the largest, the initial stiffness of DPC6 was the smallest, the stiffness degradation rate of DPC6 was the slowest, and the stiffness decline rate of DPC2 was the fastest. The stiffness of the specimen after reaching the yield stage was basically the same. The higher the strength, the lower the rate of decrease in the stiffness of the coupling beams.

(4) The Influence of the Assembly Area Length. Figure 14(e) shows the load-displacement curves of DPC2, DPC7, and DPC8 with the length of the assembly area as a variable. Among them, the length of the assembly area of DPC2 was 40 mm, the length of the assembly area of DPC7 was 20 mm, and the length of the assembly area of DPC8 was 60 mm. As shown in Figures 14(e) and 15(e), the length of the assembly area has no significant influence on the bearing capacity, ductility, seismic energy dissipation capacity, and stiffness degradation of the specimen.

5. Conclusions

To solve the problems in the construction of the fabricated frame-shear wall structure and improve the construction efficiency, this research introduced a new type of fabricated coupling beam for connecting the fabricated shear walls. To investigate the seismic performance of a new type of coupling beam and its influential factors, numerical simulation analysis and low-cycle repeated loading tests were conducted. The following conclusions can be drawn as follows:

(1) This new coupling beam had an excellent seismic performance. The failure processes of the cast-in-place specimens and assembled specimens were similar, and their cracking loads. Compared with the cast-in-place specimens, the ductility, yield load, and peak load of the assembled specimens were smaller, but the difference was not distinct. The yield load and peak load of XJ1 were 14.49% and 19.20% larger than those of ZP1, respectively. The yield load and peak load of XJ2 were 11.14% and 3.02% larger than those of ZP2, respectively.

(2) The change in the end block thickness had a great impact on the mechanical properties of all specimens. The thicker the beam end shear wall, the higher the bearing capacity, the stronger the seismic energy consumption, and the worse the ductility. Numerical simulation results are also shown in the same pattern.

(3) Other factors also have significant effects. With the increase in the cross-sectional area ratio of equivalent steel bars, the yield load and maximum load increased by 10.99% and 9.11%, respectively; the ductility coefficient and energy dissipation coefficient were reduced by 37.95% and 9.05%, respectively. In addition, the larger the diameter of the equivalent steel bars, the larger the yield load and maximum load, the worse its seismic energy dissipation capacity and ductility, and the faster the stiffness degradation.

(4) Changes in the strength of the grouting material in the assembly area and the length of the assembly area had no significant effect on specimens' yield load and peak load. Increasing the strength of the grouting material can only slightly reduce the rate of stiffness degradation of the coupling beam.

Therefore, the connection method proposed in this study was demonstrated to have similar performance to the cast-in-place structures, and the main factors' influences were discussed in this study. Future research will study a whole structure's performance, which includes the performance of walls and floors in multistory structures. Finally, studies aim to propose these kinds of structures' specific design methods.

Data Availability

All data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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