Research Article

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Mechanical behaviour of precast prestressed reinforced concrete beam–column joints in elevated station platforms subjected to vertical cyclic loading

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Abstract: Precast-reinforced concrete (RC) structures in urban rail transit projects can provide many advantages over their cast-in-place counterparts. However, lessons learned from past earthquakes show that beam-column joints may be a critical point of these structures and can overestimate the mechanical performance under vertical seismic loadings if not properly understood. This paper presents unbonded and bonded prestressed precast RC beam-column joints for elevated station platforms. Prestressed steel strands are used to provide joints with self-centring capacity. The performance of the proposed joints under vertical cyclic loadings is experimentally investigated and compared to that of monolithic joints in this study. The obtained results demonstrate the good properties of the proposed precast joints in terms of bearing capacity, energy dissipation capacity and ductility control. A comparison with a conventional monolithic beam-column joint indicates the better performance against earthquakes of the proposed precast prestressed joints, and the precast joint with symmetric prestressed steel strands in the top and bottom of the beam exhibits better flexural stiffness and energy dissipation capacity.

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1 Introduction

At present, the mainstream urban rail transit construction method is cast-in-place in developing countries [1]. However, this construction method of elevated stations has many disadvantages [2]. For example, cast-in-place construction must set up full framing and have a large range of enclosures in the scope of the station, which will greatly impact ground traffic. In addition, environmental problems such as noise and dust during construction must be solved. Therefore, it is urgent to improve the traditional design and construction method of elevated stations and seek a safer, more efficient and more environmentally friendly construction mode [3].

Currently, precast-reinforced concrete (PC) structures are considered a more environmentally friendly alternative to conventional (cast-in-place) structures in elevated stations [3,4]. PC structures are an assembly of monolithic elements (columns, beams, slabs, etc.), i.e., PC members are fabricated in the factory, transported to the construction site, and assembled together [4]. In addition to environmental benefits, PC structures have less wet work on site and better control of component quality than cast-in-place concrete structures [5]. Furthermore, precast concrete technology can facilitate the speed of construction, which has great potential for massive urban rail projects [4].

For PC systems, connections appear to be the most critical challenge, since they play a significant role in controlling the behaviour of structures [3–10]. The performance of PC structures is greatly influenced by the connection modes. As such, precast structures often collapse due to the failure of the connection systems [2,11–14]. PC

structures are distinguished from cast-in-place structures by the connection method of elements, especially by the beam–column joints [5]. Lessons learned from past earthquakes demonstrate the need for beam–column joints that are specifically conceived to make PC structures safe and permit the full exploitation of their potential [4]. There have been numerous studies on the connection mode of beam–column joints, mechanical properties of joints, and the seismic behaviour of prefabricated structures with different connection modes [3–6,15–27]. It is widely believed that precast prestressed beam–column joints are a promising solution to develop highly resilient seismic-resistant structural systems [4,5,28–33]. In this technology, beam–column joints are assembled by prestressed steel strands in the connections that are conceived to dissipate energy and to be easily repaired after earthquakes [28]. Yan et al. [5] also noted that the introduction of prestressing on the PC beam–column joint reduced the deformation and ensured good crack closure and deformation recovery performance of the structure.

To date, many studies have been conducted to investigate the seismic behaviour of prestressed PC beam–column joints [4,28–38]. Priestley and Tao [34] reported that PC beam–column joints with partially unbonded prestressing tendons had the strong self-recovery ability. The effect of energy-dissipation steel bars on the seismic performance of prestressed PC beam–column joints was experimentally investigated by Cheok et al. [35], and the self-recovering capacity was shown to be provided by the prestressing strands and energy consumed by the dissipation steel bars. To prevent concrete spalling from beam ends, Wang et al. [4] presented a type of prestressed PC beam–column joint reinforced by a steel jacket and found that it had superior resilience against earthquakes. These studies show that prestressing enhances the self-recovery ability of PC beam–column joints under seismic loadings, but some post-tensioned prestressed joints have poor energy dissipation capacity.

These studies focused on the mechanical behaviour of prestressed PC beam–column joints under horizontal seismic forces. Very limited information is available to assess the reliability of this type of joint under vertical seismic forces, which is of primary importance in strong earthquake cases such as the 1995 Hanshin Earthquake and 2008 Wenchuan Earthquake. In fact, many buildings, especially urban rail transit structures, were damaged due to the vertical earthquake shock in the 1995 Hanshin Earthquake [39], and similar cases occurred in the 2008 Wenchuan Earthquake. Moreover, the majority of previous studies focused on the application of prestressed PC joints in building structures with few elevated urban rail transit

stations. The platform of an elevated station as a secondary structure may be designed as a type of frame composed of beams with large cross sections to satisfy the requirements of bearing capacity and deformation capacity under crowd loads. Hence, it does not necessarily follow the design principle of a “strong column weak beam” in the main structures. Thus, it is necessary to examine the bearing capacity and seismic response of an elevated station platform fabricated with prestressed precast technology to facilitate designers in selecting a suitable prestressed PC beam–column joint.

As mentioned earlier, the PC system has been considered a more promising alternative to cast-in-place construction, and the introduction of prestressing on the PC system can provide many advantages. It is noted that many studies on PC systems have been conducted on high-rise structures, but limited literature focused on the prestressed PC system of elevated station platforms. In addition, very limited information is available for the mechanical performance of elevated station platforms fabricated with prestressed precast technology, especially under vertical cyclic loading, which is of dominant importance in cases such as emergency evacuation (massive crowd load) and earthquake.

This study aims to develop a prestressed precast beam–column joint for elevated station platforms. For this purpose, two types of prestressed PC beam–column joints for platforms are proposed based on the prototype of an elevated station in Guangzhou city of China, which are connected by unbonded prestressed strands and dissipation steel bars and by bonded prestressed strands. This paper focuses on the mechanical behaviours of these two prestressed PC beam–column joints at elevated station platforms under vertical cyclic loadings. A series of quasi-static tests were performed on the proposed joints and a conventional cast-in-place beam–column joint. The failure mode, stiffness, bearing capacity, ductility, and energy dissipation capacity of the prestressed PC joints were analysed. The reliability of the prestressed PC joints under vertical seismic loadings was determined by comparison with the conventional cast-in-place joint.

2 Experimental programme

2.1 Design of the experimental specimens

Based on the prototype of the beam–column joint in the elevated station platform in Guangzhou city of China, as shown in Figures 1 and 2, two prestressed PC precast

Figure 1: Schematic diagram: (a) elevated station and (b) simplified joint model of the platform.

beam-column joints (specimens J_{P-UBP} and J_{P-BP}) and one monolithic beam-column joint (i.e., cast-in-place; specimen J_M) were prepared and tested in the study. The letter “J” denotes the beam-column joint; the letters “p” and “m” denote the precast and monolithic casts, respectively; the letters “UBP” and “BP” denote the connection by unbonded prestressed strands and bonded prestressed strands, respectively. Each subassembly represents the beam-column joint of a plane frame extracted from a platform prototype of an elevated station. The platform structure is 1-story high with a story height of 1.6 m. Figures 1 and 2 show the schematic diagram of the elevated station and plan layout of the platform prototype, respectively. Here, the black dotted rectangle (Figure 2) indicates the position of the beam-column joint to be investigated. Only unidirectional connections are considered in this study. As shown in Figures 1 and 2, there is a π -member adjacent to the beam end in the prototype structure. To simplify the joint model, the π -member was replaced by the column

adjacent to the beam end in the study, so that the beam-column joint could function as a connection. In other words, the tested joint consists of one beam and one column adjacent to the beam end, as shown in Figure 1b.

In this study, all specimens were 1/2 scaled based on the prototype of the beam-column joint. The beam cross-section was scaled from 300 mm $\times$ 600 mm to 150 mm $\times$ 300 mm, and the column cross-section was scaled from 400 mm $\times$ 400 mm to 200 mm $\times$ 200 mm. Details of the monolithic specimen J_M dimensions and rebars are shown in Figure 3. To fix the lower end of the column during the test, a concrete base was designed to connect the column end of each specimen, and it was cast as a whole with the column. Figure 4 shows the section and reinforcement of the concrete base.

The prestressed PC joint connected by unbonded prestressed strands and dissipation steel bars is called specimen J_{P-UBP} , as demonstrated in Figure 5. Before casting, one tube and two tubes were embedded into the

Figure 2: Plan layout of the platform prototype.

Figure 3: Design of monolithic specimen J_M (unit: mm).

specimens to accommodate the post-tensioned strands and mild steel bars, respectively. The post-tensioned strands through the middle part of the precast beam and the upper part of the precast column are mainly responsible for the self-recovery capacity of PC joints. The mild steel bars are responsible for energy dissipation. To avoid local concrete failure, a 20 mm-wide grouting gap was left at the beam-column interface before the

joint assembly, and the grouting gap was filled with ultra-high performance concrete (UHPC) with steel fibre. This treatment was also conducted in specimens J_{P-BP} . Both energy-dissipation rebars had a 100 mm-long unbonded area near the beam end.

The prestressed PC joint connected by bonded pre-stressed strands is called specimen J_{P-BP} , as demonstrated in Figure 6. Before casting, two tubes were embedded

Figure 4: Design of the concrete base (unit: mm).

Figure 5: Design of specimens J_{p-UBP} (unit: mm).

Figure 6: Design of specimens J_{p-BP} (unit: mm).

into the specimens to accommodate the post-tensioned strands. The post-tensioned steel strand ran through the upper and lower parts of the precast beam and connected the upper part of the precast column to form an entire joint. After the tension of the steel strands, the tube was filled with high-strength mortar, which made the steel strands bond with the concrete. The prestressed steel strands of specimen J_{P-BP} are distinguished from those of specimen J_{P-UBP} ; the former plays a role in connection and shares the tensile stress.

2.2 Material properties

The mix proportion of the commercial concrete in this study was 1 (cement):1.98 (sand):3.10 (stone):0.51 (water):0.022 (additive):0.28 (mixed material). The 60-day compressive strength and elastic modulus of the concrete cylinder with a diameter of 150 mm and a height of 300 mm were 69 MPa and 37 GPa, respectively.

Two grades of rebars were used for the specimens. Except for plain bars (HPB 300), the other reinforcements were deformed bars (HRB 400). HRB 400 steel was used as the main rebar with the diameter, yield strength, ultimate strength, and elastic modulus of 16 mm, 427, 605 MPa, and 200 GPa, respectively. Steel strands with the diameter, ultimate strength, and elastic modulus of 15.2 mm, 1,860 MPa, and 195 GPa were used as the post-tensioned steel of the specimens.

A commercial high-strength mortar was used to fill the tubes of energy dissipation steel in specimen J_{P-UBP} and the prestressed strand in specimen J_{P-BP} . The 7-day compressive strength and flexural strength of the mortar with 70.7 mm × 70.7 mm × 70.7 mm cubes and 40 mm × 40 mm × 160 mm prisms were 84 and 18 MPa, respectively.

The UHPC (70.7 mm × 70.7 mm × 70.7 mm cube) to grout the interface gap of J_{P-UBP} and J_{P-BP} had a 3-day compressive strength of 103 MPa. The mix proportions of the UHPC can be found in our previous literature [40].

Noted that the mechanical properties of the above materials were the average value of three specimens made for the same set of test parameters. The compressive strength and elastic modulus of the concrete and mortar specimens were tested under axial compression loading at room temperature. The axial compressive load, controlled by the displacement, was applied at a speed of 0.2 mm·min⁻¹. The flexural strength of the mortar prisms was obtained by flexural test, which was controlled by the load with a speed of 50 N·s⁻¹. The mechanical properties of

rebars and steel strands were measured by the tensile test with a load speed of 6 MPa·s⁻¹.

2.3 Assembly of the precast specimens

The cast-in-place specimen J_M was cast as a whole with the concrete base. The beams and columns of the two precast specimens were individually cast, and the column was cast as a whole with the concrete base. After 28 days of curing, the connection of the beam with the column was conducted to assemble the joint specimen, as illustrated in Figure 7.

2.4 Test setup

The test was conducted in the structure test hall of the Guangdong University of Technology. Figure 8 shows the typical test setup of all specimens. As shown, the concrete base connected with the column was fixed by two steel beams with four anchor bolts. The vertical cyclic loads applied on the beam end were performed by a servohydraulic machine with a 1,000 kN load cell. The reversed cyclic loads were displacement-controlled and imposed displacements with equal amplitudes but opposite signs. Figure 9 demonstrates the loading protocol, and the drift ratio is the ratio of the beam end displacement to the beam length. In the initial stage, the displacement amplitude was set to be 2.5, 5 and 10 mm for one cycle to ensure that all the specimens showed elastic responses. Subsequently, the displacement amplitude was set to be 20, 30, 40, 50, 60, 70, 80 mm, etc. for three cycles until the vertical applied load decreased to 85% of the peak load or joint was severely damaged and could not continue to resist the load.

The instrumentations to monitor the load, displacement and strain during the test are shown in Figure 8. The force sensors installed in the actuator were used to measure the load applied to the specimens. The time-dependent prestress force of the steel strands was monitored by fibre bragg grating (FBG) sensors, which were bonded on the midspan surface of the strands. The displacement of the specimens was recorded by linear variable differential transformers (LVDTs). Moreover, many electric resistance strain gauges were attached to the rebars of the beams and columns around the joint core and attached to the concrete surface of the beam end adjacent to the column to acquire the strain evolution of the rebars and concrete, as shown in Figure 8.

Figure 7: Assembly steps of precast specimens: (a) specimen J_{p-UBP} and (b) specimen J_{p-BP} .

3 Experimental results and discussion

3.1 Failure modes

3.1.1 Specimen J_M

During vertical cyclic loading, monolithic joint J_M experienced four stages: initial crack stage, yielding stage,

ultimate stage and failure stage. Figure 10 shows the distribution (front and rear views) of cracks in specimen J_M under different loading displacements. At the beginning of loading, specimen J_M was in the elastic stage, the load-displacement curve was linear, the displacement amplitude was basically symmetrical, and the loading and unloading curves basically coincided with each other, so there was almost no residual deformation after unloading. When the vertical displacement of the beam end was $\Delta = 5 \text{ mm}$ (1/300 drift ratio), horizontal cracks first began to

Figure 8: Test setup: (a) East view and (b) West view.

Figure 9: Cyclic loading scheme.

appear in the column and gradually developed with increasing displacement, as shown in Figure 10a. After $\Delta = 20$ mm, the joint entered the yielding stage. At $\Delta = 30$ mm (1/50 drift ratio),

many vertical cracks appeared in the column. At $\Delta = 50$ mm (1/30 drift ratio), the cracks in the column ran through the joint core area, and some cross-inclined cracks appeared. Then, the

Figure 10: Distribution of cracks in specimen J_M : (a) $\Delta = 5$ mm; (b) $\Delta = 10$ mm; (c) $\Delta = 20$ mm; (d) $\Delta = 30$ mm; (e) $\Delta = 40$ mm; (f) $\Delta = 50$ mm; (g) $\Delta = 60$ mm; (h) $\Delta = 70$ mm; and (i) $\Delta = 80$ mm.

concrete in the column was crushed and peeled off. At $\Delta = 70$ mm, the core area of the joint was split, and the positive bearing capacity (push-down) dropped to 65% of the peak value. At $\Delta = 80$ mm, the positive (push-down) bearing capacity of the joint continued to decline, while the negative (pull-up) bearing capacity dropped to 95% of the peak value. The concrete in the column at the joint core area was seriously broken; thus, the test was stopped. The beam had insignificant damage during loading. The failure mode of specimen J_M is shown in Figure 11. Specimen J_M failed at the joint core area, which indicates that specimen J_M is a typical joint with a strong beam and a weak core. This failure mode can be attributed to the beam having a larger cross section than the column, which may be required because the platform of the elevated station functions as a secondary structure. However, the failure process shows that although failure occurred in the joint core area, there was no collapse phenomenon after failure, which suggests that the bearing capacity and safety storage of specimen J_M under vertical cyclic loads are not bad.

3.1.2 Specimen J_{P-UBP}

Specimen J_{P-UBP} also experienced four stages: the initial crack stage, yielding stage, ultimate stage and failure stage, but it had a different failure process from specimen J_M . Figure 12 shows the distribution (front and rear views) of cracks in specimen J_{P-UBP} under different loading displacements. At $\Delta = 5$ mm (1/300 drift ratio), the first horizontal crack of specimen J_{P-UBP} appeared in the column. At $\Delta = 20$ mm, the cracks in the column developed from the horizontal direction to the vertical direction. At $\Delta = 40$ mm, some diagonal cracking appeared in the joint core. At $\Delta = 50$ mm, the connection interface between beam and column opened and closed during the reserved loading. At $\Delta = 60$ mm, the concrete in the top of the column was crushed and peeled off, and the gap at the

interface between column and beam end increased during loading. At $\Delta = 70$ mm, the steel strands significantly shrunk, accompanied by a loud noise during the second cycle of push-down loading; then, the bearing capacity of specimen J_{P-UBP} sharply dropped. At $\Delta = 90$ mm, the cone cracks in the joint core area continued to expand, and the concrete at the top of the column was seriously broken, which resulted in the bearing capacity of the joint dropping below 85% of the peak load. Figure 13 demonstrates the failure mode of specimen J_{P-UBP} . As observed, the failure of specimen J_{P-UBP} mainly occurred in the core area. Unlike the failure mode of specimen J_M , the concrete at the top of the joint core area of J_{P-UBP} exhibited local splitting failure, which could be attributed to the stress concentration caused by the local compression in the anchorage zone of prestressed strands. A comparison of Figures 11 and 13 shows that the safe storage of specimen J_{P-UBP} after failure appears better than that of specimen J_M . This finding is consistent with previous analogous investigations [4,5].

3.1.3 Specimen J_{P-BP}

The failure process of specimen J_{P-BP} is partly similar to that of specimen J_{P-UBP} , especially before yielding. Figure 14 shows the distribution (front and rear views) of cracks in specimen J_{P-BP} under different loading displacements. Comparing Figures 10, 12 and 14, we observe that specimen J_{P-BP} had the fewest cracks and smallest crack width, which indicates that specimen J_{P-BP} had the best self-recovering capacity. Moreover, a smaller gap opened at the interface between column and beam in specimen J_{P-BP} than in specimen J_{P-UBP} under identical loads. Figure 15 shows the failure mode of specimen J_{P-BP} , and the main failure occurred at the end of the column, which distinguishes it from the joint core failure of specimen J_{P-UBP} . This result

Figure 11: Failure modes of specimen J_M .

Figure 12: Distribution of cracks in specimen J_{p-UBP} : (a) $\Delta = 5$ mm; (b) $\Delta = 10$ mm; (c) $\Delta = 20$ mm; (d) $\Delta = 30$ mm; (e) $\Delta = 40$ mm; (f) $\Delta = 50$ mm; (g) $\Delta = 60$ mm; (h) $\Delta = 70$ mm; and (i) $\Delta = 80$ mm.

Figure 13: Failure modes of specimen J_{p-UBP} .

is mainly due to the arrangement of prestressed steel strands at the upper and lower parts of the joint core area, and the anchorage plate reinforced the core area, which led to better safety storage for specimen J_{p-BP} .

3.2 Hysteretic performance

Figure 16 illustrates the hysteretic responses of all specimens under vertical cyclic loading. In this study, the

Figure 14: Distribution of cracks in specimen J_{p-BP} : (a) $\Delta = 5$ mm; (b) $\Delta = 10$ mm; (c) $\Delta = 20$ mm; (d) $\Delta = 30$ mm; (e) $\Delta = 40$ mm; (f) $\Delta = 50$ mm; (g) $\Delta = 60$ mm; (h) $\Delta = 70$ mm; and (i) $\Delta = 80$ mm.

push-down displacement is defined as positive, and the corresponding load is also positive. At the initial stage of loading, the hysteresis curves were smooth and full. Although the specimen was in the elastic phase, the downward and reverse loading curves of the hysteresis

loops were almost noncoincident, which is consistent with previous studies [4,5]. This result can be attributed to the geometrical asymmetry of the joints in this study. Because of the asymmetry of boundary conditions, all specimens had a fuller hysteresis curve and a higher

Figure 15: Failure modes of specimen J_{p-BP} .

Figure 16: Hysteretic curves: (a) specimen J_M ; (b) specimen J_{P-UBP} ; (c) specimen J_{P-BP} ; and (d) comparison of three specimens.

bearing capacity under pull-up loading than under push-down loading, especially for specimen J_M (two additional longitudinal reinforcements were arranged in the top compared to those in the bottom of the beam, as shown in Figure 3), as shown in Figure 16.

With increasing loading, the hysteresis responses showed nonlinearity manifested by pinching of the hysteresis loops. Figure 16 shows that both precast specimens had fuller hysteresis loops than the monolithic specimen after yielding until the peak load. However, in the post-peak stage, specimen J_{P-UBP} exhibited an interesting phenomenon under cyclic loading: the load increased and subsequently suddenly dropped with increasing displacement, and this phenomenon was more pronounced under

push-down loading, as shown in Figure 16b). This result is attributed to the large gap at the interface between column and beam after the concrete crush, which caused a rapid drop in the hysteresis loop under large loads; however, due to the resilience of the prestressed strands, the structure can continue to bear the load until failure. Figure 16 shows that specimen J_{P-BP} exhibited the best hysteretic performance in the post-yielding stage.

3.3 Bearing capacity and ductility

The envelope curves of the tested specimens are shown in Figure 17. Before yielding, the envelope curves of

Figure 17: Envelope curves of specimens.

the three specimens almost coincided; in the post-peak stage, the precast specimens exhibited a significantly greater stiffness than the monolithic specimen. Thus, the advantage of the prestressing connection on the vertical load resistance mainly occurs in the post-peak stage of the beam–column joint. In addition, it is interesting to find a hardening stage of specimen J_{P-UBP} after yielding, which may be attributed to the self-recovering capacity of the prestressed strands embedded in the middle part of the beam and the contribution of energy-dissipation rebars.

Table 1 presents the yielding load and peak load of the tested specimens. The yielding loads of the three specimens are similar, except specimen J_{P-BP} under pull-up loading had a significantly greater yielding load than specimen J_{P-UBP} and specimen J_M , which can be due to the prestressed strands in the bottom part of the beam. Compared with specimen J_M , specimen J_{P-UBP} and specimen J_{P-BP} improved the peak load by 16.4 and 5.4% in the push-down direction and improved the peak load by –2.1 and 6.1% in the pull-up direction. Thus, the proposed

Figure 18: Method to determine the yielding point.

prestressed PC joints in this study have good vertical bearing capacity.

In this study, the ductility of beam–column joints is determined by the following equation [41,42]:

$$\mu = \Delta_u / \Delta_y, \quad (1)$$

where Δ_y is the yield displacement at the yielding load and Δ_u is the ultimate displacement at 85% post-peak load (the ultimate displacement of specimen J_M in the pull-up direction was monitored at 95% post-peak load because the test was stopped at this moment). Figure 18 shows the adopted method to define the yield displacement of all tested specimens.

The ductility comparisons of all specimens are presented in Table 1. The two precast specimens had a similar ductility, and specimen J_{P-UBP} showed the highest ductility ($\mu = 3.27$) among the three specimens, while the ductility of specimen J_M was approximately 12% lower than that of specimen J_{P-UBP} and specimen J_{P-BP} . Thus, the ductility comparisons of the monolithic specimen and precast specimens show that the presented PC joints have excellent ductility under vertical cyclic loadings.

Table 1: Bearing capacity and ductility of the specimens

Specimens	Loading direction	Q_y (kN)	Δ_y (mm)	Q_p (kN)	Δ_p (mm)	Q_u (kN)	Δ_u (85%) (mm)	μ	Average (μ)
J_M	Push	19.9	18.9	22.6	40	19.2	52.6	2.79	2.87
	Pull	–21.7	–26.9	–29.3	–70	–27.8 ^a	–80a	2.95	
J_{P-UBP}	Push	20.8	25.3	26.3	70	22.3	82.0	3.25	3.27
	Pull	–20.7	–25.2	–28.7	–40	–24.4	–82.9	3.28	
J_{P-BP}	Push	18.9	20.0	23.8	50	20.2	73.5	3.68	3.25
	Pull	–26.3	–26.7	–30.4	–70	–25.8	–75.3	2.82	

Notes: Q_y = yielding load; Q_p = peak load; Δ_p = displacement at the peak load.

^a At 95% of the post-peak load.

3.4 Degradation of stiffness

The stiffness of the beam–column joints is calculated as follows [18]:

$$K_i = \frac{|+F_i| + |-F_i|}{|+X_i| + |-X_i|}, \quad (2)$$

where $|+F_i|$ and $|-F_i|$ are the peak loads of the i th cycle in the push and pull directions, respectively; $|+X_i|$ and $|-X_i|$ are the peak displacements of the i th cycle in the push and pull directions, respectively.

Figure 19 shows the degradation evolution of the stiffness of all specimens during cyclic loading. Because specimen J_M was damaged during the preliminary preparation, it exhibited a lower stiffness before the 10 mm displacement. With displacements of 10–80 mm, the three specimens have similar stiffness curves, and the stiffness gradually decreases with increasing displacement. A comparison in Figure 19 shows that specimen J_{P-BP} had better stiffness than specimen J_{P-UBP} and specimen J_M after yielding until they reached the peak load, which is attributed to the four prestressed steel strands that were symmetrically arranged in the top and bottom of the beam.

3.5 Energy dissipation capacity

The energy dissipation capacity is one of the crucial characteristics to evaluate the performances of beam–column joints under seismic loads [42]. In this study, the energy dissipation is evaluated by the enclosed area inside the hysteretic loop of each cycle [43]. The energy dissipation

Figure 19: Degradation evolution of the stiffness.

capacities of all specimens are shown in Figure 20. Figure 20(a) presents the energy dissipation of three specimens in each cycle. Figure 20(b) shows the accumulated energy dissipation of specimens during loadings. Figure 20(a) and (b) show that at the elastic stage (approximately $\Delta < 20$ mm), the energy dissipation curves of the three specimens basically coincide; with the increase in joint displacement, the energy dissipation curves gradually separate; after yielding, the energy dissipation capacity of specimen J_{P-BP} was similar to that of specimen J_M but better than that of specimen J_{P-UBP} ; once entered the post-peak stage, the energy dissipation capacity of specimen J_M decreased with increasing the displacement, but those of specimens J_{P-BP} and J_{P-UBP} continued to increase, so the accumulated energy dissipation of the two precast specimens is greater than that of the monolithic specimen. This improvement in accumulated energy dissipation of precast specimens benefited from the self-recovery capacity of the prestressed strands [4,5]. As shown in Figure 20(b), specimen J_{P-BP} had the largest accumulated energy dissipation at failure among the three specimens; when the loading displacement was 80 mm, the accumulated energy dissipation of specimen J_{P-BP} was higher than those of specimens J_M and J_{P-UBP} by 34 and 10%, respectively.

In addition, due to the accumulated damage under cyclic loading, the hysteresis loop area of the subsequent cycle under identical load levels is commonly lower than that of the first cycle, so the energy consumption values of three cycles with identical load levels were in a declining trend. This phenomenon can be found in the curves of specimens J_M and J_{P-BP} , as shown in Figure 20(a). However, the curve of specimen J_{P-UBP} in Figure 20(a) did not follow this evolution. To clarify this distinguishment, Figure 20(c) presents the energy dissipation curve of specimen J_{P-UBP} in each cycle. The energy dissipation capacity of specimen J_{P-UBP} non-linearly varied with the number of cycles, which could be attributed to the post-gapping (at the interface between column and beam) resilience effect of the prestressed strands embedded in the middle part of the beam. This result corresponds to the evolution of its hysteretic curve, as shown in Figure 16(b).

3.6 Displacement and rotation evolution

Figure 21(a–c) presents the evolution of the horizontal and vertical displacements of the three specimens during cyclic loading. The vertical displacement of the beam end adjacent to the column was measured by LVDT-1, and the horizontal displacement of the column end was measured

Figure 20: Energy dissipation capacity: (a) energy dissipation capacity per cycle; (b) accumulated energy dissipation; and (c) energy dissipation capacity of J_{P-UBP} per cycle.

by LVDT-2. As expected, these vertical and horizontal displacements of the three specimens generally increased with increasing vertical displacement of the free beam end, i.e., the loading point. Figure 21(a) shows that the horizontal displacement of the column end rapidly increased after specimen J_M was yielded. This result did not occur on specimens J_{P-UBP} and J_{P-BP} , as shown in Figure 21(b) and (c), which can be attributed to the anchorage of prestressed strands strengthening the core area.

The rotation θ of the joint core area can be calculated by equation (3):

$$\theta = \frac{|\Delta_1| + |\Delta_2|}{h} (\text{rad}), \quad (3)$$

where Δ_1 and Δ_2 are the displacement values measured by LVDT-2 and LVDT-3, respectively, as shown in Figure 21(d), and h is the distance between LVDT-2 and LVDT-3.

The rotation θ of the three specimens during loading is shown in Figure 21(d). Because the local concrete at the position of LVDT-3 was crushed, which interrupted the data acquisition, only the data before 50 mm were intercepted. Figure 21(d) shows that at the beginning of loading (before 10 mm), the specimens were elastic; consequently,

Figure 21: Displacement and rotation of specimens: (a) specimen J_M ; (b) specimen J_{P-UBP} ; (c) specimen J_{P-BP} ; and (d) rotation of the joint core area.

the rotation angles of the specimens were small, and the curves basically coincided. After 10 mm, the rotation of the three specimens continued to increase in a similar trend and generally became gentle due to the yielding of rebars in the column. Figure 21(d) shows that the rotation of specimen J_{P-BP} was higher than those of specimens J_M and J_{P-UBP} . This finding indicates that specimen J_{P-BP} had the greatest stiffness among the three specimens.

3.7 Evolution of rebars strain and strands stress

Figure 22 shows the load-strain curves of the longitudinal reinforcement at the beam end and the stirrup in the joint core of the three specimens. The strains of the longitudinal bars of the beam end of the three specimens were relatively small, and they basically remained in

the elastic stage, which was manifested by the failure mode of the specimens. Figure 22 shows that the strain of the stirrup is significantly greater than that of the beam rebar, and the stirrup of all specimens yields at failure. These findings demonstrate that great damage occurs in the joint core area of the specimens in Section 3.1. Compared with specimens J_M and J_{P-UBP} , specimens J_{P-BP} had a lower strain in the stirrup, which indicates that the prestressed system with symmetrical arrangement can enhance the joint core, as mentioned earlier.

Figure 23 presents the stress evolution of post-tensioned strands. As shown, specimen J_{P-UBP} under loading had a higher strand stress variation than specimen J_{P-BP} , but the stress variation of the prestressed strands of the two precast specimens was slight (varying below 80 MPa), which is approximately 4% of the ultimate strength of the steel strand. Thus, this strand stress variation was almost negligible and did not diminish the

Figure 22: Rebar strain of the specimens: (a) rebar in the beam of J_M ; (b) rebar in the column of J_M ; (c) rebar in the beam of J_{P-UBP} ; (d) rebar in the column of J_{P-UBP} ; (e) rebar in the beam of J_{P-BP} ; and (f) rebar in the column of J_{P-BP} .

effect of the prestressed system. These findings reveal that the proposed joints can efficiently control the prestress loss

of the post-tensioned strands under vertical cyclic loads, which is beneficial to the self-centring capacity of the joint.

Figure 23: Stress evolution of steel strands: (a) steel strand stress of J_{P-UBP} vs cycles; (b) steel strand stress of J_{P-UBP} vs displacement at the loading point; (c) upper strand's stress of J_{P-BP} vs cycles; (d) upper strand's stress of J_{P-BP} vs displacement at the loading point; (e) lower strand's stress of J_{P-BP} vs cycles; and (f) lower strand's stress of J_{P-BP} vs displacement at the loading point. (Note: the strain value was measured by the FBG sensor set on the middle of the steel strand.).

4 Conclusion

Two types of prestressed precast beam–column joints were proposed for the elevated station platform. The prestressed precast joint connected by unbonded post-tension strands embedded in the middle part of the beam and dissipation steel bars is called J_{P-UBP} , and the prestressed precast joint connected by bonded post-tension strands symmetrically arranged in the top and bottom of the beam is called J_{P-BP} . An experimental study was conducted to investigate the structural performance of the two precast prestressed beam–column joints under vertical cyclic loads compared to the monolithic joint. The following conclusions can be made as follows:

1. The monolithic joint based on the modified prototype platform exhibited a typical failure mode of a “weak joint core”. The prestress system of the precast joints could reinforce the joint core and may consequently change the failure mode.
2. Both precast prestressed joints have a good vertical carrying capacity. Compared with the monolithic joint, the precast prestressed joints could improve the ultimate carrying capacity by approximately 5–16% in the push-down direction.
3. The presented precast joints have excellent ductility under vertical cyclic loadings. The two precast specimens had similar ductility (approximately 3.2), which is approximately 12% higher than that of the monolithic joint.
4. Both precast prestressed joints have good energy dissipation capacity, especially after the peak load. The accumulated energy dissipation of the two precast prestressed joints is greatly higher than that of the monolithic joint at failure. Compared with J_{P-UBP} , J_{P-BP} has a better energy dissipation capacity under identical loads.
5. The precast prestressed joints have a similar stiffness degradation trend to the monolithic joint. In general, among the tested joints, J_{P-BP} has the best stiffness before the peak load.
6. The two proposed joints can efficiently keep the prestress system under vertical cyclic loads and consequently provide good safety storage after peak loading.

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References

- [1] Jh, A., B. Gqs, C. Zl, B. Bz, and D. Wz. Barriers to promoting prefabricated construction in China: A cost–benefit analysis – ScienceDirect. *Journal of Cleaner Production*, Vol. 172, 2018, pp. 649–660.
- [2] Ghayeb, H. H., H. A. Razak, and N. Sulong. Performance of dowel beam-to-column connections for precast concrete systems under seismic loads: A review – ScienceDirect. *Construction Building Materials*, 2020, Vol. 237, id. 117582.
- [3] Zhang, Q. and M. S. Alam. State-of-the-art review of seismic resistant precast bridge columns. *Journal of Bridge Engineering*, Vol. 25, No. 10, 2020, id. 03120001.
- [4] Wang, H., E. M. Marino, P. Pan, H. Liu, and X. Nie. Experimental study of a novel precast prestressed reinforced concrete beam-to-column joint. *Engineering Structures*, Vol. 156, 2018, pp. 68–81.
- [5] Yan, X., S. Wang, C. Huang, A. Qi, and C. Hong. Experimental study of a new precast prestressed concrete joint. *Applied ences*, Vol. 8, No. 10, 2018, id. 1871.
- [6] Wu, C., J. Xie, and F. Liu. Research progress and prospects on beam-to-column connections of precast concrete fram. *Concrete*, Vol. 4, 2020, pp. 128–133+137 [In Chinese].
- [7] Mokhtar, R., Z. Ibrahim, M. Z. Jumaat, Z. Abd. Hamid, and A. H. Abdul Rahim. Behaviour of semi-rigid precast beam-to-column connection determined using static and reversible load tests. *Measurement*, Vol. 164, 2020, id. 108007.
- [8] Brunesi, E., S. Peloso, R. Pinho, and R. Nascimbene. Shake-table testing of a full-scale two-story precast wall-slab-wall

- structure. *Earthquake Spectra*, Vol. 35, No. 4, 2019, pp. 1583–1609.
- [9] Brunesi, E., S. Peloso, R. Pinho, and R. Nascimbene. Cyclic tensile testing of a three-way panel connection for precast wall-slab-wall structures. *Structural Concrete*, Vol. 20, No. 4, 2019, pp. 1307–1315.
  - [10] Brunesi, E., R. Nascimbene, and S. Peloso. Evaluation of the Seismic Response of Precast Wall Connections: Experimental Observations and Numerical Modeling. *Journal of Earthquake Engineering*, Vol. 24, No. 7, 2020, pp. 1057–1082.
  - [11] Elsanadedy, H. M. New moment-resisting beam–column joints to increase progressive collapse resistance of precast concrete buildings. *Journal of Building Engineering*, Vol. 44, 2021, id. 102884.
  - [12] Elsanadedy, H. M., Y. A. Al-Salloum, M. A. Alrubaidi, T. H. Almusallam, N. A. Siddiqui, and H. Abbas. Upgrading of precast RC beam–column joints using innovative FRP/steel hybrid technique for progressive collapse prevention. *Construction and Building Materials*, Vol. 268, 2021, id. 121130.
  - [13] Elsanadedy, H. M., Y. A. Al-Salloum, M. A. Alrubaidi, T. H. Almusallam, and H. Abbas. Finite element analysis for progressive collapse potential of precast concrete beam-to-column connections strengthened with steel plates. *Journal of Building Engineering*, Vol. 34, 2021, id. 101875.
  - [14] Quiel, S. E., C. J. Naito, and C. T. Fallon. A non-emulative moment connection for progressive collapse resistance in precast concrete building frames. *Engineering Structures*, Vol. 179, 2019, pp. 174–188.
  - [15] Aninthaneni, P. K., R. P. Dhakal, J. Marshall, and J. Bothara. Nonlinear cyclic behaviour of precast concrete frame sub-assemblies with “dry” end plate connection. *Structures*, Vol. 14, 2018, pp. 124–136.
  - [16] Shufeng, L., L. Qingning, Z. Hao, J. Haotian, Y. Lei, and J. Weishan. Experimental study of a fabricated confined concrete beam-to-column connection with end-plates. *Construction Building Materials*, Vol. 158, 2018, pp. 208–216.
  - [17] Ketiyot, R. and C. Hansapinyo. Seismic performance of interior precast concrete beam–column connections with T-section steel inserts under cyclic loading. *Earthquake Engineering and Engineering Vibration*, Vol. 17, 2018, pp. 355–369.
  - [18] Cai, X., N. N. Gong, C. C. Fu, Y. Zhu, and J. C. Wu. Seismic behavior of self-centering prestressed precast concrete frame subassembly using steel top and seat angles. *Engineering Structures*, Vol. 229, 2020, id. 111646.
  - [19] Khaloo, A. and R. Bakhtiari Doost. Seismic performance of precast RC column to steel beam connections with variable joint configurations. *Engineering Structures*, Vol. 160, 2018, pp. 408–418.
  - [20] Breccolotti, M., S. Gentile, M. Tommasini, A. L. Materazzi, M. F. Bonfigli, B. Pasqualini, et al. Beam–column joints in continuous RC frames: Comparison between cast-in-situ and precast solutions – ScienceDirect. *Engineering Structures*, Vol. 127, 2016, pp. 129–144.
  - [21] Dal Lago, B., P. Negro, and A. Dal Lago. Seismic design and performance of dry-assembled precast structures with adaptable joints. *Soil Dynamics Earthquake Engineering and Engineering Vibration*, Vol. 106, 2018, pp. 182–195.
  - [22] Jbyab, C., Z. Wei, B. Zzta, G. A. Xuan, and B. Sp. Seismic behavior of precast concrete beam–column joints with steel strand inserts under cyclic loading. *Engineering Structures*, Vol. 216, 2020, id. 110766.
  - [23] Li, L. X., H. N. Li, C. Li, Y. B. Yang, and C. Y. Zhang. Modeling of force-displacement behavior of post-tensioned self-centering concrete connections. *Engineering Structures*, Vol. 198, 2019, id. 109538.
  - [24] Wang, C. L., L. Ye, X. Zheng, and W. Jie. Experimental investigation of a precast concrete connection with all-steel bamboo-shaped energy dissipaters. *Engineering Structures*, Vol. 178, 2019, pp. 298–308.
  - [25] Guerrero, H., V. Rodriguez, J. A. Escobar, S. M. Alcocer, F. Bennetts, and M. Suarez. Experimental tests of precast reinforced concrete beam–column connections. *Soil Dynamics Earthquake Engineering and Engineering Vibration*, Vol. 125, id. 105743.
  - [26] Bakhtiari Doost, R. and A. Khaloo. Steel web panel influence on seismic behavior of proposed precast RCS connections. *Structures*, Vol. 32, 2021, pp. 87–95.
  - [27] Sousa, R., N. Batalha, and H. Rodrigues. Numerical simulation of beam-to-column connections in precast reinforced concrete buildings using fibre-based frame models. *Engineering Structures*, Vol. 203, 2020, id. 109845.
  - [28] Wang, H., E. M. Marino, and P. Peng. Design, testing and finite element analysis of an improved precast prestressed beam-to-column joint. *Engineering Structures*, Vol. 199, 2019, id. 109661.
  - [29] Kailai, D., P. Peng, A. Lam, P. Zhenhua, and Y. Lieping. Test and simulation of full-scale self-centering beam-to-column connection. *Earthquake Engineering and Engineering Vibration*, Vol. 12, No. 4, 2013, pp. 599–607.
  - [30] Vasdravellis, G., T. L. Karavasilis, and B. Uy. Finite element models and cyclic behavior of self-centering steel post-tensioned connections with web hourglass pins – ScienceDirect. *Engineering Structures*, Vol. 52, No. 9, 2013, pp. 1–16.
  - [31] Haishan, G., Q. Hu, P. Peng, S. Pengfei, L. Liming, L. Kang, et al. Study on a novel precast unbonded post-tensioned concrete beam–column connection. *Journal of Building Structures*. Doi: 10.14006/j.jzjgxb.2019.052 [In Chinese].
  - [32] Peng, P., W. Haishen, G. Haishan, L. Kang, W. Dongyan, Q. Hu et al. Experimental study of seismic performance of unbonded post-tensioned pre-stressed beam-to-column dry connections. *Journal of Building Structures*, Vol. 39, No. 10, 2018, pp. 46–55 [In Chinese].
  - [33] Kurosawa, R., H. Sakata, Z. Qu, and T. Suyama. Precast prestressed concrete frames for seismically retrofitting existing RC frames. *Engineering Structures*, Vol. 184, 2019, pp. 345–354.
  - [34] Priestley, M. N. and J. R. Tao. Seismic response of precast prestressed concrete frames with partially debonded tendons. *PCI Journal*, Vol. 38, No. 1, 1993, pp. 58–69.
  - [35] Cheok, G., W. C. Stone, and S. K. Kunnath. Seismic response of precast concrete frames with hybrid connections. *ACI Structural Journal*, Vol. 95, 1998, pp. 527–538.
  - [36] Xiong, X., G. Yao, and X. Su. Experimental and numerical studies on seismic behavior of bonded and unbonded pre-stressed steel reinforced concrete frame beam. *Engineering Structures*, Vol. 167, 2018, pp. 567–581.
  - [37] Yao, G. and X. Xiong. Detailed numerical research on the performance of unbonded prestressed SRC frame beam under

- vertical cyclic load. *Engineering Structures*, Vol. 177, 2018, pp. 61–71.
- [38] Koshikawa, T. Moment and energy dissipation capacities of post-tensioned precast concrete connections employing a friction device. *Engineering Structures*, Vol. 138, 2017, pp. 170–180.
- [39] Harada, K. N. I., S. Katsuki, and T. Ohta. On the damage of RC column under vertical earthquake shock. *Proceeding of the 3rd Asia-Pacific Conference on SHOCK&IMPACT Loads On Structures*, CI-Premier Conference Organisation, Singapore, 1999, pp. 155–162.
- [40] Huang, K., J. Xie, R. Wang, Y. Feng, and R. Rao. Effects of the combined usage of nanomaterials and steel fibres on the workability, compressive strength, and microstructure of ultra-high performance concrete. *Nanotechnology Reviews*, Vol. 10, 2021, pp. 304–317.
- [41] Ngo, T. T., T. M. Pham, and H. Hao. Effects of steel fibres and prestress levels on behaviour of newly proposed exterior dry joints using SFRC and CFRP bolts. *Engineering Structures*, Vol. 205, 2020, id. 110083.
- [42] Ngo, T. T., T. T. Tran, T. M. Pham, and H. Hao. Performance of geopolymer concrete in monolithic and non-corrosive dry joints using CFRP bolts under cyclic loading. *Composite Structures*, Vol. 258, 2021, id. 113394.
- [43] Ngo, T. T., T. M. Pham, and H. Hao. Ductile and dry exterior joints using CFRP bolts for moment-resisting frames. *Structures*, Vol. 28, 2020, pp. 668–684.