Experimental study on a new precast post-tensioned concrete beam column connection system

Do Tien THINH[1], Koichi KUSUNOKI [2] and Akira TASAI [3]

[1] *PhD Student, Department of Architecture, Yokohama National University, Yokohama, Japan*
[2] *Associate Professor, Department of Architecture, Yokohama National University, Yokohama, Japan*
[3] *Professor, Department of Architecture, Yokohama National University, Yokohama, Japan*
Email: thinhdt2001@yahoo.com

ABSTRACT :

The objectives of this study were to investigate the hysteresis characteristics, shear transferring capacity, damage state and possibility to disassemble the precast unbonded post-tensioned connections with and without shear bracket under the simultaneous cyclic load and large gravity load. Three half-scale specimens of precast unbonded post-tensioned exterior beam-column connection of large span frame was proposed and tested. The test results of the specimen without shear bracket showed excessive beam slip, resulted in large residual deformation, while the specimens with shear bracket that have the amount of prestressed steel 50% less than that of the specimen without shear bracket expressed very good performance with very small residual deformation, almost no slip, minor damage to the beam and column elements, and easy to disassemble even after experienced the story drift up to 6% drift.

KEYWORDS: Precast concrete, Post-tensioned, Exterior beam column connection, Shear friction.

1. INTRODUCTION

The reinforced concrete (RC) building is generally stiffer and cheaper to construct than steel structures. These advantages motivate to construct high-rise RC apartment buildings in urban areas. Recently, the number of the RC high-rise apartment buildings is drastically increasing in Japan. However, most of office buildings are steel structures, since office building requires much wider space than apartment building. In order to achieve long-span RC structure, special techniques, i.e. prestressed and post-tensioned techniques, should be used.

On the other hand, from the environmental point of view, the structure should be easy to disassemble when the buildings need to be removed. In order to achieve this purpose, the beam-column joint system with un-bonded prestressed bars is applied for the proposed system in this paper. As for the un-bonded prestressed concrete (hereafter referred to as PC) joint, the vertical constant load due to the live and dead loads is carried by the friction at the beam-column interface, and bending moment due to lateral external force is carried by the PC bars. In the office building, because of the long-span frame, the vertical constant load becomes larger than that of apartment buildings. The amount of PC bars for joints at higher stories depends on the vertical load rather than the bending moment, then larger amount of PC bars tends to be required from the view point of the lateral load resistance. In order to reduce the amount of PC bars, new joint system was proposed in this paper, where the column had a steel bracket on the surface of the joint to carry the vertical load of the beam mechanically.

This study investigates the seismic behavior of the un-bonded precast prestressed concrete exterior beam column joint at the upper story of long-span frame building. In this study, the test parameter was the behavior of the shear bracket. A specimen without the shear bracket was also tested to investigate how the shear friction performs at the beam-column interface. In order to obtain the research objectives, large concentrated vertical load was applied on the beam to simulate the effect of the gravity load. This vertical load was maintained throughout the test. The behavior of the joint with bracket and design recommendation would also be discussed in the paper.

2. EXPERIMENTAL PROGRAM

2.1 Specimens Outline

The test specimens were taken to represent exterior beam-column connection at the 10th floor of a twelve-story prototype office building, of which story height is four meters and the span length is eighteen meters. There were three one-half scale specimens in this study. The first, named SP1-A, was designed without shear bracket. The second, named SP2-A, was designed with shear bracket to resist the shear force induced by the gravity load. The third, named SP3-A, was similar with specimen SP2-A, was designed to resist the gravity load 1.5 times that of specimen SP2-A to check the performance of the connection even with longer span. Due to the limitation of loading system, the beam length was shortened from 4.5m to 2.415m. Brief outline of the test specimens are shown in Table 2.1 while dimensions and reinforcement details of the specimens is shown in Figure 2.1

Table 2.1 Specimens outline

Specimens		SP1-A	SP2-A	SP3-A
Beam	Section (mm)	300 x 500		
	F_c (N/mm^2)	60	70	69
	f_y (N/mm^2)	339	339	339
	f_{wy} (N/mm^2)	313	313	313
	PC steel	2-ϕ26 SBPR	2-ϕ15 SBPR Grade C	2-ϕ15 SBPR Grade C
	σ_0 (Mpa)	4.0	1.8	1.8
	P_0/P_y	0.72	0.72	0.72
	PC length (mm)	1500	1500	1500
	Bracket area a_w (mm^2)	none	3036	4950
Column	Section (mm)	400 x 400		
	F_c (N/mm^2)	60	70	69
	f_y (N/mm^2)	534	534	534
	f_{wy} (N/mm^2)	313	313	313

Where: F_c was the concrete compressive strength, f_y was the yield strength of beam and column longitudinal reinforcement, f_{wy} was the yield strength of lateral reinforcement, σ_0 was the initial concrete stress in the beam, P_0 was the initial prestressed force, P_y was the yield load of prestressed steel, a_w was the shear resistance area of the bracket.

The specimens were designed so that the beam and column reinforcement still within elastic range and only the PC bar yields, and the most critical section was the beam-column interface. Two interlock ϕ6D150 steel spirals were arranged at the top of the beam end near the column face to prevent the compressive failure of the concrete at large drift level. The demand moment and shear force at the beam end were 86 kNm and 288 kN, respectively. In the specimen SP1-A, the PC bar was designed to resist all the demand moment and shear force, while in the specimen SP2-A, SP3-A, the shear force induced by the gravity load was carried by the shear bracket, and the PC steel was designed to resisted the demand moment and shear force induced by the seismic load.

In the specimen SP1-A, beam shear force was transferred to the column by the shear friction at the beam-column interface. The amount of PC steel was determined by Eqn. (2.1):

$$P_0 = \frac{Q}{\mu} \tag{2.1}$$

Where: P_0 was the initial prestressed force, Q was the total shear force at the beam end, and μ was the friction coefficient, μ = 0.5 by AIJ guidance. Applied 2ϕ26 SBPR 785/1030 PC bars with yield load of P_y=834 kN which was larger than the required P_y of 823 kN. Corresponded yielded moment strength was M_y = 187.7 kNm, satisfy to resist demand moment of 86 kNm.

In the specimen SP2-A, SP3-A, the amount of PC bars was determined by Eqn. (2.2):

$$M_y = 0.9\frac{P_y.D}{2} \tag{2.2}$$

Where: M_y was the yield moment strength, P_y was the PC yield load, D was the beam height. Applied 2ϕ15 SBPR 1080/1230 PC bars with the yield load of P_y=382 kN. Corresponded moment and shear strength were 86 kNm and 133.7 kN, respectively. It can be seen that with the existence of the shear bracket, the required volume of the PC steel was considerably reduced. The PC bars were arranged at the mid-height of the beam to delay yielding of the PC bars at large drift level.

The shear bracket in the specimen SP2-A and SP3-A was designed for long-term load as a T-shaped corbel welded to a steel plate that anchored to the column by studs. The shear strength of the bracket was calculated by Eqn. (2.3):

$$Q_s = 0.9\frac{F_y}{1.5\sqrt{3}}a_w \geq Q_L \tag{2.3}$$

where: Q_s was the long-term shear strength of the bracket, F_y was the yield strength of the steel material, a_w was the shear resistance area, and Q_L was the shear force at the beam-column interface induced by the gravity load.

In order to prevent failure of the beam by stress concentration at the top face of the bracket touching with the beam, the bracket was designed with T-shaped section to widen the top face area. At the beam end, in order to prevent failure of the concrete resulted from large compressive stress, the inverted U-shaped steel box was used. The beams and columns were cast separately in vertical direction. At the concrete age of 4 days, the gap between the beam and the column was grouted. The ϕ3 @30 x 30 steel meshes was used to reinforce the grout and prevent it from spalling down at large drift level. Six days after grouting, the beam was jointed to the column by the PC bars.

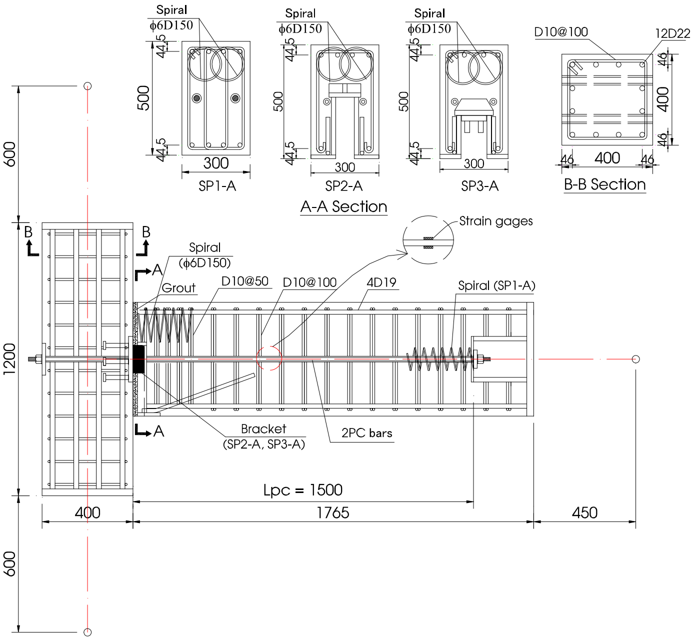


Figure 2.1 Reinforcement details of the specimens

2.2 Test Setup

2.2.1 Test setup

The test setup and measuring system is shown in Figure 2.2. The bottom of the column was connected to the strong floor with the pin while the top was connected to the reaction wall by the horizontal two-end pin brace that equivalent to a vertical roller. The cyclic load was applied to the beam end by the 1000 kN vertical hydraulic oil jack connected to the beam end with the pin. The gravity load was applied to the beam as a concentrated load at the distance of 215 mm from the column face. As mentioned above, the beams of the specimens were shortened from 4.5m to 2.415m.In order to generate the same combination of moment and shear force at the beam column interface as in original condition; the gravity load was controlled as Eqn. (2.4) according to the original gravity load Q_{L1} and the cyclic load Q_{CY}.

$$Q_L = Q_{L1} + \left(\frac{L_2 - L_1}{L_1 - L'}\right) Q_{CY} \tag{2.4}$$

Where: Q_{L1} was the original gravity load, L_1 was the original beam length, L_1 = 4.5m, L_2 was the new beam length, L_2 = 2.415m, L' was the distance from the gravity load to the column face, L' = 0.215 m, Q_{CY} was the cyclic load. Q_{CY} had the same sign with Q_L if they act on the same direction, and vice versa.

2.2.2 Loading history

The loading history is shown in Figure 2.3. It should be noted that the gravity load was applied before the cyclic load and exist throughout of the test. For the cyclic load, the first two cycles was loaded by the force control. The peak values were 0,1Q and 0,5Q, where Q is the story shear force that corresponded with yielding of the PC bars. After that, displacement control was used with the peak displacements of 0.25%, 0.5%, 0.75%, 1%, 1.5%, 2%, 3%, 4% story drift. Two cycles were conducted at each story drift level. Because of the limitation of the transducers, after finished the cycles of 4% drift, the cyclic load was applied as pushover load up to 6% story drift in positive direction.

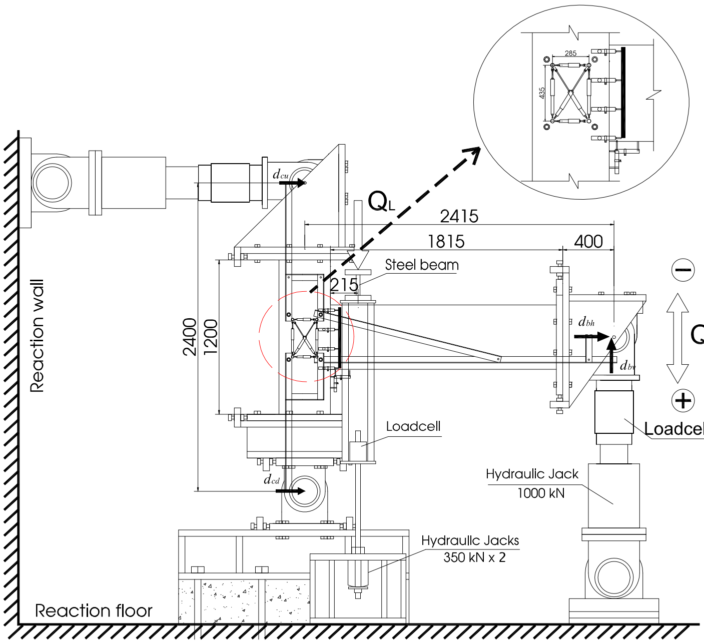


Figure 2.2 Test set up

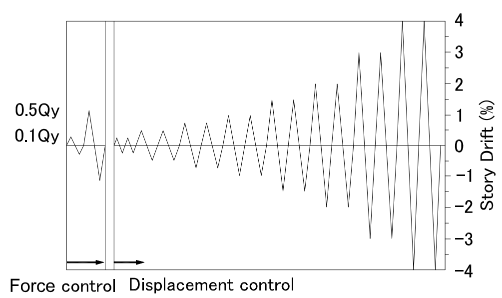


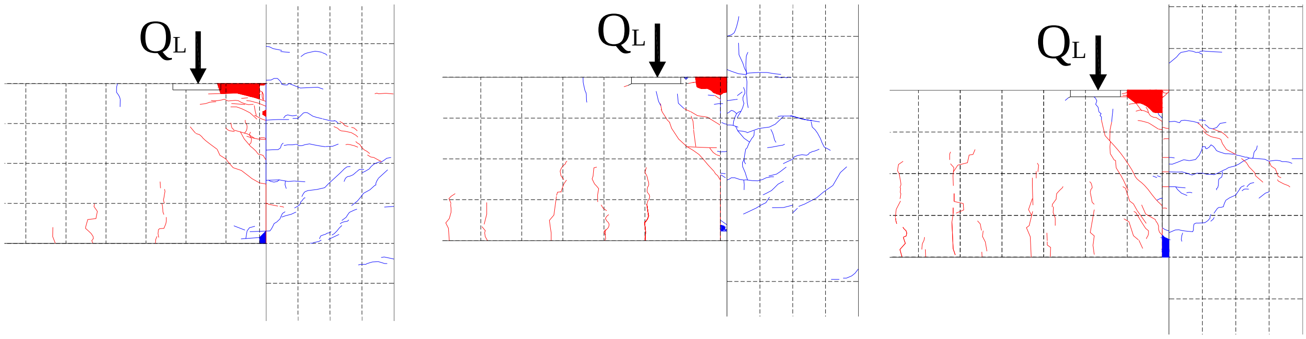
Figure 2.3 Loading history

2.2.3 Data acquisition system

The gravity load, cyclic load, and PC load were measured by the loadcells. The story drift, lateral and vertical deformation of the beam and column were measured by the displacement transducers. The opening of the beam at the column face and the vertical slip of the beam were also monitored. The strain gages were attached at the midpoint of the PC bars to measure its strains. Three beam stirrups closest to the column face; four hoops of the column within the joint, column longitudinal bars were measured using electric resistance strain gauges. The gauges were also used to measure the strain of the inverted U-shaped steel box.

3. TEST RESULTS AND DISCUSSIONS

3.1 Visual Observation



SP1-A SP2-A SP3-A

Figure 3.1 Crack patterns of the specimens after 4% drift

a) SP2-A b) SP3-A

Figure 3.2 Shear bracket and U-shaped steel box after the test

Figure 3.1 shows the crack patterns of the specimens at 4% drift. The bracket and U-shaped steel box after the test were shown in Figure 3.2. It can be seen that very few cracks occurred in all specimens compared to normal RC connection at large story drift of 4%. There was nearly no flexural crack occurred in the columns of all specimens. Some shear cracks occurred in the joint area, but their width also very small, less than 0.15 mm. However, in the specimen SP2-A, some vertical cracks occurred in the joint area, near the position of the shear bracket at the second cycle of 4% drift level.

In the beams of all specimens, flexural cracks were fewest in the specimen SP1-A. As mentioned above, the amount of PC steel of the specimen SP1-A was about twice that of the other specimens, provided two times larger flexural strength, hence fewer cracks occurred. On the other hand, the number of beam flexural cracks was largest in the specimen SP3-A, because of small amount of PC steel and largest gravity load. However, all flexural cracks were very small with the width less than 0.1 mm and absolutely closed when the load was removed. In all specimens, at the story drift of 0.25%, an inclined shear crack occurred from the lower part of the beam to the point of applying gravity load, and stopped to extent at 0.75% drift. At the story drift of 0.09% and 0.2% (SP1-A), 0.07% and 0.12% (SP2-A), 0.09% and 0.15% (SP3-A), decompression occurred in the positive and negative direction, respectively. The crushing of cover concrete at the top of the beam and falling of the grout at the bottom of the beam critical section began at 1.5% drift (SP1-A), and 2% (SP2-A, SP3-A).

As seen in the Figure 3.2, the shear brackets and U-shaped steel boxes were not suffered from any damage or deformation, although they have experienced very large vertical load and high drift level. Especially in specimen SP3-A where the gravity load was 1.5 times larger than that in other specimens. Moreover, in the case of specimen SP2-A and SP3-A, it was very easy to separate the beam out of the column after the test, confirmed one of the purpose of this type of structural that it should be easy to disassemble.

3.2 Hysteresis Characteristics

The beam moment – rotation relationship of the specimens is shown in Figure 3.3, while the beam moment – story drift relationship is shown in Figure 3.4. The test results are summarized in Table 3.1. All the specimens were successfully passed the drift of 4% in negative directions and 6% in positive direction, and no fracture of PC bars was recorded. As shown in Figure 3.3, all specimens show very small residual rotation of the beam. As seen in Figure 3.4, while the self-centering characteristics of the specimens SP2-A and SP3-A was very good, that of specimen SP1-A was poor. As discussed in the next paragraph, the vertical slip of the beam in the specimen SP1-A was considerably large and was the cause of poor behavior of this specimen.

In the specimen SP1-A, the PC steel was yielded at the drift of 1.99% and 1.74%, and maximum moment strength was reached at the drift of 5.21% and 4% in positive and negative direction, respectively. In the specimen SP2-A, the PC steel was yielded at the drift of 3.82% and 2.65%, and maximum moment strength was reached at the drift of 4.97% and 2.82% in positive and negative direction, respectively. In the specimen SP3-A, the PC steel was yielded at the drift of 3.85% and 2.61%, and maximum moment strength was reached at the drift of 5.62% and 1.82% in positive and negative direction, respectively. In the specimen SP3-A, in the negative direction, the maximum strength was reached at 1.82% drift, before the yielding of the PC steel. The reason may be the crush of the concrete at the top of the beam, hence the concrete compressive force could not develop.

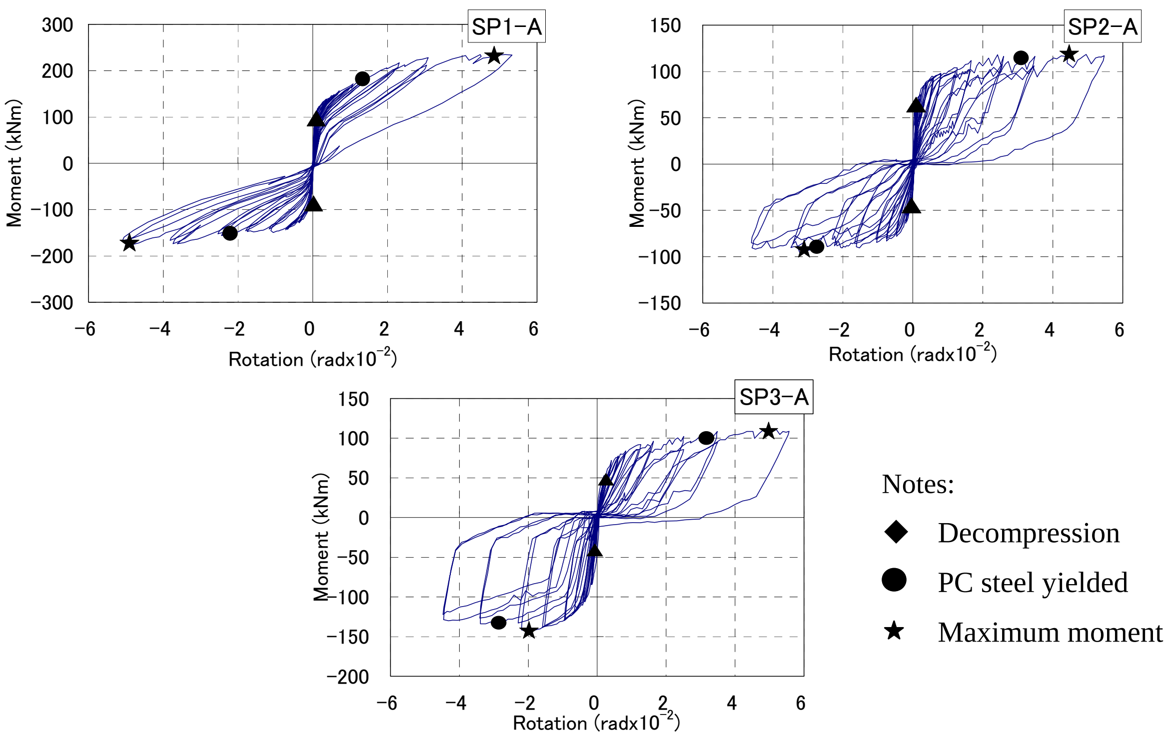


Figure 3.3 Moment-rotation relationship

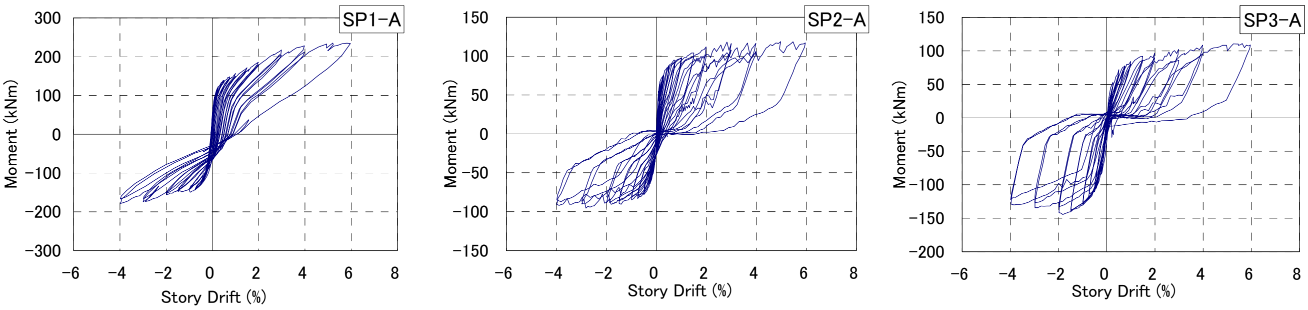


Figure 3.4 Moment-story drift relationship

Table 3.1 Test results: Beam moment strength and story drift

Specimens	Loading Direction	M_d (kNm)	R_d (%)	M_y (kNm)	R_y (%)	M_{max} (kNm)	R_{max} (%)
SP1-A	+	97.1	0.09	185.6	1.99	234.9	5.21
	–	-84.7	-0.20	-152.5	-1.74	-178.7	-4.0
SP2-A	+	52.7	0.09	109.4	3.82	118.7	4.97
	–	-50.3	-0.12	-94.2	-2.65	-95.4	-2.82
SP3-A	+	53.8	0.07	101.9	3.85	110.9	5.62
	–	-43.1	-0.15	-132.0	-2.61	-144.3	-1.82

3.3 Beam slip

The beam slip – story drift relationship of three specimens is shown in Figure 3.5. The beam slip of the specimens at 4% drift were 20.8 mm, 1.01 mm, and 1.04 mm for the specimens SP1-A, SP2-A, and SP3-A, respectively. Figure 3.5 shows that the beam slip of specimen SP1-A was excessive larger than specimen SP2-A and SP3-A. It can be said that the shear bracket successfully prevented the slip of the beam in the specimens SP2-A and SP3-A. Figure 3.6 shows the relationship between beam slip and story drift, while Figure 3.7 shows the relationship between the ratio of the beam shear force to PC force and story drift of the specimen SP1-A. Figure 3.6 shows that the beam slip increased gradually when the story drift become higher, but the slip increased faster when the drift was around +1% of each cycles, and after finished 6% drift, at the drift of 2.49%, the slip increased rapidly. As seen in Figure 3.7, the ratio of beam shear force over PC force (Q/P_{PC}) made a "peak" when the story drift was around +1%, coincide with the slip status in Figure 3.6. (Q/P_{PC}) ratio at the drift of 2.49% at the drift of 2.49% after finished 6% drift was 0.57 and reached maximum value of 0.9. It can be concluded that the shear friction was not a secure mechanism to transfer the shear force at the beam-column interface.

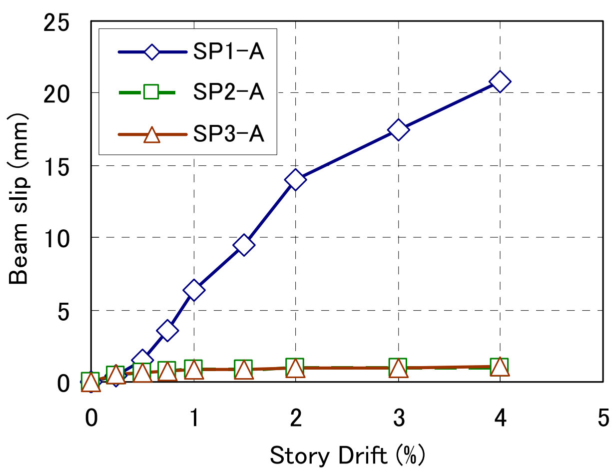


Figure 3.5 Beam slip-story drift relationship of all specimens

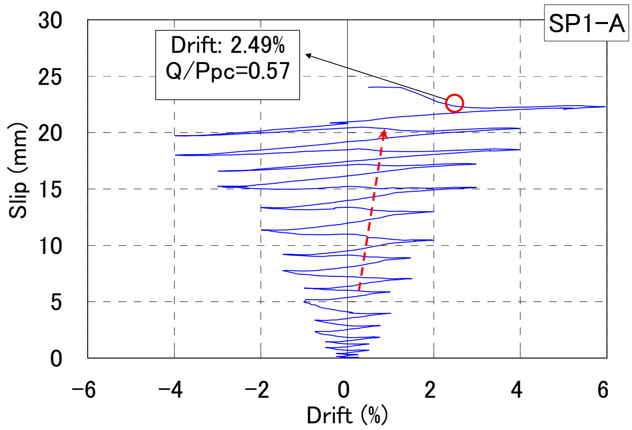


Figure 3.6 Beam slip vs. story drift, SP1-A

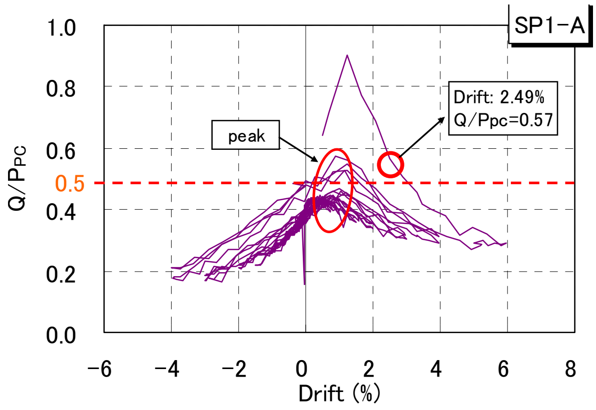


Figure 3.7 Beam shear force/ PC force, SP1-A

4. CONCLUSIONS

From the test results, following conclusions can be drawn.

All three specimens were successfully passed the story drift of 4% in both directions and 6% drift in positive direction, without the fracture of PC steel bar.
The specimens with shear bracket expressed very well seismic performance even under the combination action of very large gravity load and story drift, with small residual deformation, nearly no beam slip and little damage of the beam and column elements.
The specimens without shear bracket experienced large beam slip, the slip occurred from 0.5% drift and gradually increased according to drift level. That caused large residual deformation for this specimen. Hence, the shear transfer by shear friction mechanism only should not be used for this type of structure.

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