-Technical Paper-

STUDY ON A NEW PRECAST POST-TENSIONED BEAM-COLUMN JOINT SYSTEM FOR RAPID ERECTION AND IMPROVED RESILIENCY

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ABSTRACT

New type of precast unbonded post-tensioned exterior beam-column connection was proposed and tested. The connection was designed with shear bracket to resist the shear force due to the gravity load and post-tensioned bars to resist moment and shear force due to the earthquake load. The design concept is to control the deformation due to the earthquake load by accepting the opening between the beam and the column face in order to keep away from the damage to the beam and the column. The test results showed efficient seismic behaviors with minor cracks occurred in beam and column, well self-centering behavior and very small beam slip.

Keywords: unbonded; post-tensioned; precast concrete, exterior connection; shear bracket;

1. INTRODUCTION

Precast concrete buildings were known as cost-effective, well-controlled quality, and fast, easy erection on site. In Japan, up to now, the design philosophy for precast concrete buildings is to simulate cast-in-situ concrete structure behavior with strong connection. The common approach is connecting precast element by cast-in situ concrete and topping slab [1]. Recently, new design philosophy was developed for precast concrete frame building. In this philosophy, the connections are detailed to be weaker than the precast elements and are allocated as location of ductile inelastic deformation. The precast elements are designed to have adequate margin strength over that of the connection and will remain elastic under seismic response. Consequently, the precast elements would not need to be detailed for ductility, hence resulting in economies. Moreover, in many cases, ductile connection could be designed to be replaceable at much lower cost than repairing of damage to a ductile reinforced concrete [2].

Pampanin, S. et al. (2006) developed and tested a new hybrid system in that unbonded post-tensioned steel was used together with external energy dissipaters and shear bracket. However, the test specimens in this experiment just simulated short-span frame and no gravity load was apply to the beam to simulate the actual situation [3].

The objectives of this study is design new type of connection which post-tensioned steel resists the moment induced by seismic and gravity loads, and shear bracket resists the shear force induced by the gravity load. The connection should have well performance at design level of loading, easy and rapid for erecting and disassembling. In order to investigate the performance of the connection, gravity load was applied together with the cyclic load. This study focuses on the exterior connection in upper stories of a large span frame building, where the gravity load dominates the design of the connection. Two test specimens, one with shear bracket and the other without shear bracket were designed and tested. The behavior of two specimens was recorded and compared.

2. EXPERIMENTAL PROGRAM

2.1 Specimens Outline

The test specimens were taken to represent exterior beam-column connection at the 10^{th} floor of a twelve-story office building, of which story height is four meters and the span length is eighteen meters. The columns and beams sections are 800x800 mm and 600x1000 mm, respectively. The test specimens modeled the region from mid-column height below the joint to the mid-column above the joint and the region from the joint to the mid-span of the beam. There were two specimens in this study. The first, named SP1, was designed without shear bracket. The second, named SP2, was designed with shear bracket to resist the shear force induced by the gravity load. Due to the limitation of the loading system, test specimens were scaled down to one-half scale of the prototype and the beam length was shortened from 4.5m to 2.415m. Brief information of the test specimens are shown in Table 1.

2.2 Specimens Design

The specimens were designed following ductile connection philosophy in which flexural and the shear strength of the beams and columns were higher than that of the connection. Dimensions and reinforcement

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details of two specimens is shown in Figure 1. The beam and column sectional dimensions are 300 x 500 mm and 400 x 400 mm, respectively. The longitudinal reinforcements for the beam are 4D19 at both top and bottom sides, and the stirrups are 4D10 with the space of 100mm. In the beam end area of which length is 250mm from the column face, the stirrups were provided at closer space of 50mm. Column longitudinal reinforcement are 12D22 uniformly arranged around the perimeter, while the hoops are 2D10 at 100mm space.

The demand moment and shear force at the beam end were 86 kNm and 288 kN, respectively. In the specimen SP1, the PC steel was designed to resist all the demand moment and shear force, while in the specimen SP2, 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.

The moment strength at opening (the crack propagates though the sectional centre of the beam) of the beam at the column face was calculated by the formula [4]:

$$M_{cr} = \frac{N.D}{3} \tag{1}$$

where: N is the initial prestressed force, $N = 0.7P_y$, and D is the beam height.
Yield moment strength was calculated as:

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

In the specimen SP1, shear force was carried by the shear friction at the beam and column interface. The volume of PC steel was designed as follow:

$$N=Q/\mu \tag{3}$$

where: N is the total initial prestressed force, $N=0.7P_y$, P_y is the yield load of the PC steel, provided by the manufacturer, Q is the total shear force at the beam end, and μ is the friction coefficient, μ = 0.5 follow the guidance of AIJ [5], N = 576 kN, hence P_y = 823 kN. Applied 2ϕ26 SBPR 785/1030 PC bars with yield load P_y=834 kN larger than required P_y = 823 kN, corresponded yielded moment strength is M_y = 187.7 kNm, satisfy to resist demand moment of 86 kNm.
In the specimen SP2, 2ϕ15 SBPR 1080/1230 PC bars with yield load P_y=382 kN were used. 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 greatly reduced. The PC bars were arranged at the middle level of the beam to avoid yielding of the PC steel at low drift level.

The shear bracket in specimen SP2 was designed as a simple corbel welded to a steel plate that embedded in the column. Shear strength of the bracket was calculated as:

Table 1 Specimens outline

Specimens		SP1	SP2
Beam	b x D (mm)	300 x 500	300 x 500
	PC bars	2ϕ26 P_y=834 kN	2ϕ15 P_y=382 kN
	Initial prestressed force	0.7P_y=592 kN	0.7P_y=267 kN
	PC length, mm	1500	1500
	Cracking moment, (kNm)	101.7	48.2
	Yield moment, (kNm)	187.7	86
Column	b x D (mm)	400 x 400	400 x 400
Shear Bracket,（mm）		no	120x19

$$Q_s = 0.9\frac{F_y}{1.5\sqrt{3}}a_w \geq Q_L \tag{4}$$

where: Q_s is the shear strength of the bracket, F_y is yield strength of the steel plate, a_w is the shear resistance area, a_w = h x t, h and t are the height and thickness of the bracket, respectively, and Q_L is the shear force at the beam end induced by the gravity load, Q_L = 255 kN. The bracket with the dimensions of 120 x 19 mm was applied.
The length of shear bracket is calculated as follows:

$$l_s = R \times 0.4D + b_m + L_t \tag{5}$$

where: R is the maximum target beam end rotation, D is the beam height (mm), b_m is the grout thickness (mm), and L_t is the manufacture tolerance (mm).
Here R= 4% = 0.05, b_m = 20mm, L_t = 10mm, hence l_s =

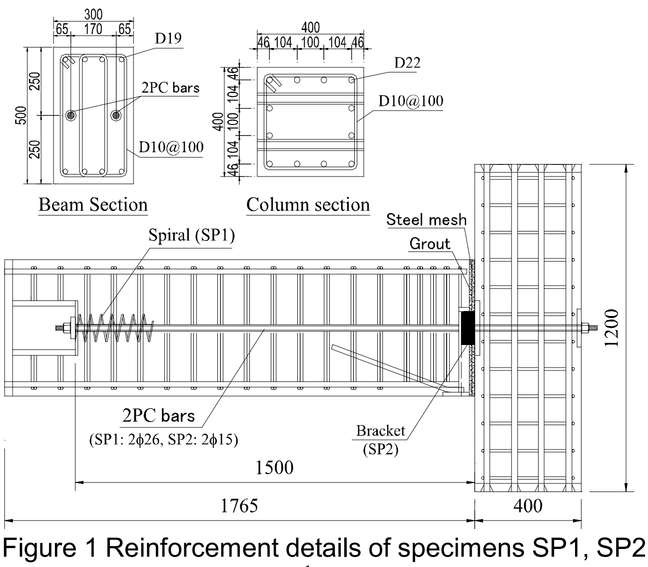


Figure 1 Reinforcement details of specimens SP1, SP2

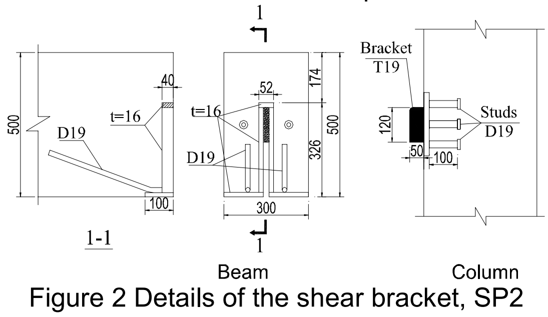


Figure 2 Details of the shear bracket, SP2

Table 2 Test result of usage materials

	Design strength (N/mm^2)	σb (N/mm^2)	Ec (N/mm^2)	σt (N/mm^2)
Concrete	60	60.4	36510	3.8
Grout	60	68.1		3.2

Steel	Usage	σy (N/mm^2)	σu (N/mm^2)	εy (%)	Es (N/mm^2)
D6(SD345)	Spiral	410.2	535.3	0.23	194280
D10(SD295)	Stirrup	329.3	459.9	0.39	171011
D19(SD295)	Beam main bar	337.7	521.6	0.18	188566
D22(SD345)	Column main bar	386.2	555.9	0.22	183244
T16(SN490C)	U-shaped box	369.0	533.9	0.26	201949
T19(SN490C)	Bracket	357	536.4	0.20	205046

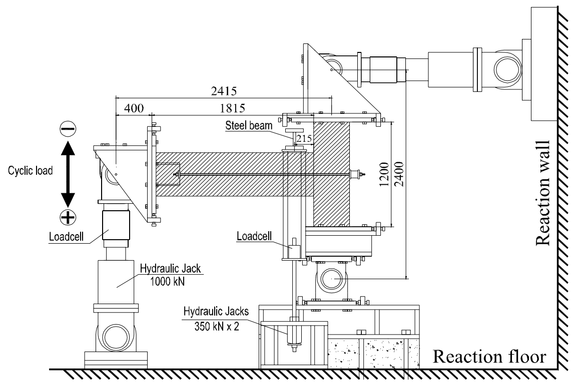


Figure 3 Test setup

40 mm. Applied l_s = 50 mm in order to prevent failure of beam by stress concentration at the place where top face of the bracket get in touch with the beam.

The bracket was welded to a steel plate which connected to the studs. The shear force and moment from the bracket was transferred to the column though these studs. At the beam end, in order to prevent the concrete from large concentrated stress, an inverted U-shaped steel box was used. The detail of the bracket and inverted U-shaped steel box is shown in Figure 2. The concrete and grout materials were tested at the starting and finishing days of the experiment, and average values of test results are shown in Table 2.

2.3 Specimens Construction

The specimens were casted from the same batch of concrete in vertical direction. At the concrete age of 8 days, the gap between the beam and the column was filled by the cement grout. A steel mesh was used to reinforce the grout and prevent it from spalling down at high drift level. Fifty-one days after casting, the beams were prestressed to the column.

2.4 Test Setup and Testing Sequence

(1) Experimental setup

The experimental setup is shown in Figure 3. The lower end of the column was connected to the strong floor by the pin while the upper end 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 jack that attached to the beam end with the pin. A concentrated vertical load of 255 kN was applied to the beam to simulate the total dead load and live load of 9.8 kN/m^2. Because the main aim of applying this load was to create the shear force at the beam end, hence it was applied as near the column face as possible. However, because of the limit of the loading system, the load was put at the distance of 215 mm from the column face.

(2) Loading history

The loading history is shown in Figure 4. 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 carried at each story drift level. However, after finishing target drift of 4%, the load was applied as pushover loading up to 6% story drift in positive direction. It should be noted that the vertical load was applied before starting of cyclic load and exist all time during the test.

(3) Data acquisitation system

The gravity load, cyclic load, and PC load were measured by loadcells. The story drift was measured by the system shown on Figure 5. The rigid frame was attached to the top and bottom pin of the column and the transducer was fixed on the frame to measure vertical displacement of the beam end. The opening of the beam at column face, vertical slip of the beam were also monitored. The strains of PC bars, four 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

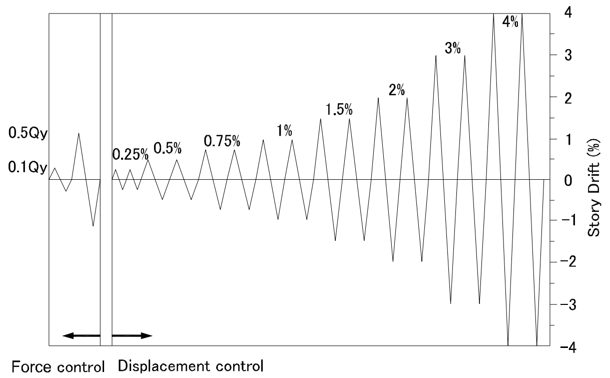


Figure 4 Loading history

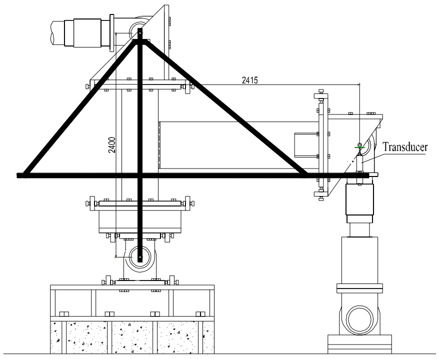


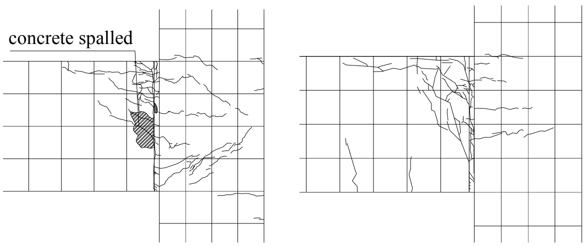
Figure 5 Measurement system of story drift

gauges were also used to measure the strain of the inverted U-shaped steel box.

3. TEST RESULTS AND DISCUSSIONS

3.1 Visual Observation

Figure 6 shows the crack pattern of specimens SP1 and SP2 after 4% drift. For the specimen SP1, beam flexural cracking was concentrated to a wide single crack at the interface between the beam and the grout. Flexural cracks started to occur in column at the drift of 0.25%, inclined cracks occurred in the joint area at a drift of 0.5% and continue to developed up to the drift of 4%. These inclined cracks might not be induced by joint shear stress but by the concrete compressive strut. All the cracks occurred in the column and joint were very small and completely close when the load was removed. Cracks also occurred in the upper part of the beam end near the column face in the negative direction while there are no cracks occurred in the lower part. This might be the result from the combination effect of compressive stress and shear stress induced by the concentrated vertical load. These cracks caused the top part of the beam became "softened". The concrete at the top part of the beam near the column face started crushing at the drift of 1.5% and continued when the drift increased. In the column, very few flexural cracks occurred. However, in the joint area, the inclined cracks distributed from the position near the top and bottom of the beam to the end of the PC bars. The grout at the bottom of the beam started to fall down at the drift of 0.75%. The concrete of the beam started to fall down at the drift of 3% at the top part and increased at higher drift level.



a) SP1 b) SP2

Figure 6 Crack pattern of specimens after 4% drift

Figure 7 Shear bracket and inverted U-shaped steel box after testing

For the specimen SP2, the crack pattern of the beam was similar to that of the specimen SP1 with cracks occurred in the upper part of the beam end. However, there were only some small horizontal cracks occurred in the joint area of the column.

In specimen SP2, no damage was observed on the shear bracket except little deformed at the top where it contacts with the steel part of the beam. The bracket could resist the beam even at the drift up to 6%. The inverted U-shaped steel box at the beam end was designed to transfer the shear force from the beam to the shear bracket on the column. In this steel part, the welding between the top horizontal plate and the vertical plates started to be damaged and the top horizontal plate was bent up. Shear bracket and the inverted U-shaped steel box after testing are shown in Figure 7.

3.2 Moment – Rotation Hysteresis Response

The beam moment – rotation angle relationships of specimens SP1 and SP2 are shown in Figure 9a and Figure 9b, respectively. In these figures, moments are the acting moment at the column face due to the cyclic and concentrated vertical loads on the beam and determined as:

$$M=P*l-Q_L*l' \tag{6}$$

where: P is the cyclic load, l is the distance from the beam-end pin to the column face, Q_L is the concentrated vertical load, and l' is the distance from the concentrated load to the column face.

The calculation curve was modeled as bilinear behavior, where the moments M_{cr} and M_y were determined from Eq. (1) and (2), and the rotation was determined from the beam stiffness. Initial stiffness of the beam was calculated as:

$$K_e = \frac{3EI}{l} \tag{7}$$

Stiffness of the beam after opening was calculated as:

$$K_2 = \frac{E_{pc}.A_{pc}}{l} d'^2 \tag{8}$$

where: E and I were the Young modulus and second moment of the beam, E_{pc} and A_{pc} were the Young modulus and area of the PC, respectively, l is the beam length, is the moment lever arm, as shown in Figure 8

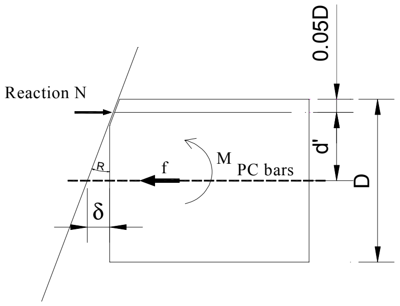
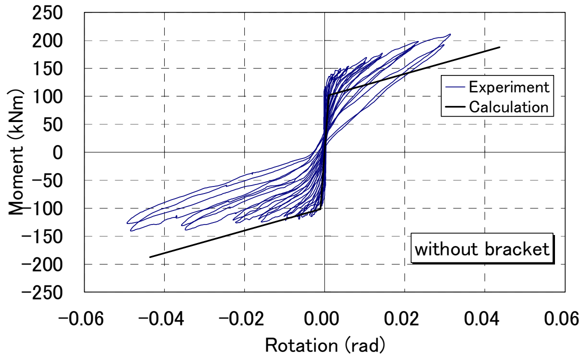
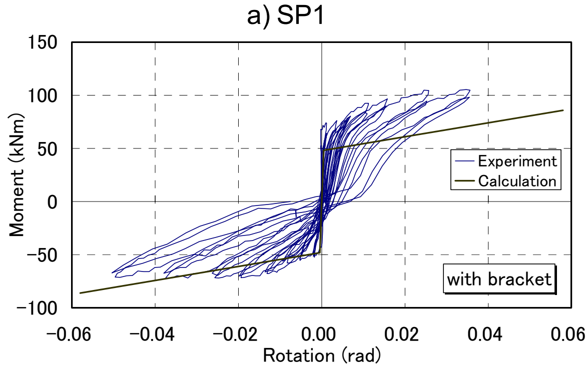


Figure 8 Determination of d'



a) SP1



b) SP2

Figure 9 Moment-Rotation Relationship

Figure 9a indicates stable hysteresis response and small residual drift. Figure 9b also shows good response except considerable residual drift at high drift level. Calculated strengths are also expressed in these figures. Both specimens show higher moment strength and stiffness after opening in the positive compared to that of the negative direction. From the visual observation, it can be seen that the "softening" of the concrete at the upper part of the beam made the concrete compressive force could not fully developed, hence moment strength was reduced. From the figures, it can be seen that the calculated moments are smaller in positive direction and larger in negative direction than the experimental ones.

3.3 Beam Slip and Friction Coefficient

From Figure 10 it can be seen that, from the drift of 0.5%, the slip in specimen SP1 increased rapidly, although this specimen was designed with the prestressed force that enough to carry the shear force by shear friction. In Figure 10, Q_b is the beam end shear force and N is the total PC force. After applying vertical load and before applying cyclic load, Q_b /N ratio was 0.418, and up to 3% drift cycle, this ratio was still less than the design friction coefficient value of 0.5. It

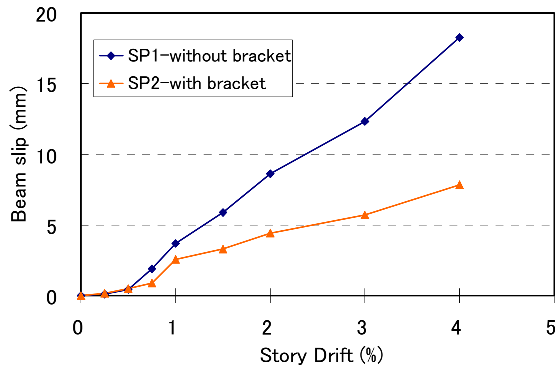


Figure 10 Slip of the beam

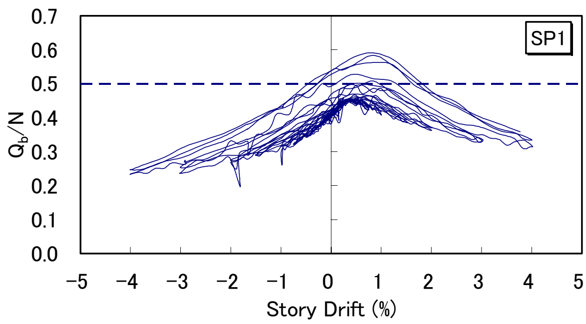
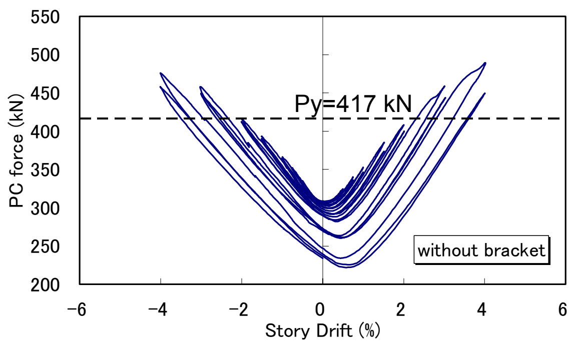
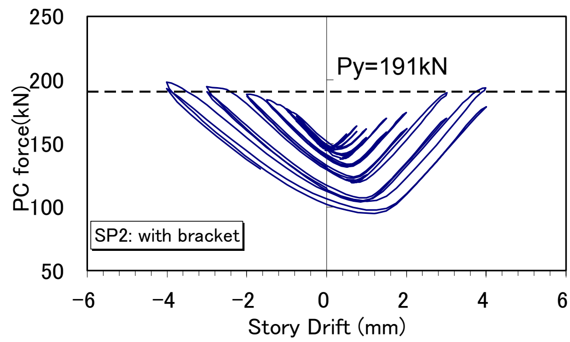


Figure 11 Variation of beam shear force/PC force ratio



a) SP1



b) SP2

Figure 12 PC force – Story drift relationship

seems that the proposed friction coefficient proposed by AIJ is not sufficient in this case and further studies should be carried out about this.

The specimen SP2 was designed with shear bracket to carry the shear force cause by the concentrated vertical load. From Figure 11, it can be seen that the bracket worked well and the slip was much smaller than that of specimen SP1. However, the small slip still happened because of some deformation at the bracket and the inverted U-shaped steel box.

3.4 PC force – Story Drift Relationship

Figure 12 shows the relationship between the prestressed forces and the story drift. The PC force here is the average force in the two PC bars. The dashed line illustrates the yield force of the PC bar. At the drift of 3%, the PC bars have yielded in both positive and negative directions in specimen SP1, while in specimen SP2, the PC bars started to yield only in negative direction. It can be seen that, the PC bars in the specimen SP1 has yielded more clearly than that of the specimen SP2. It can be concluded that, the bracket has postponed the yielding of the PC bars.

3.5 Strain of the U-shaped steel box

The relationship between strain of the vertical plate of the U-shaped steel box and story drift is shown in Figure 13. It can be seen that the strain increased as the

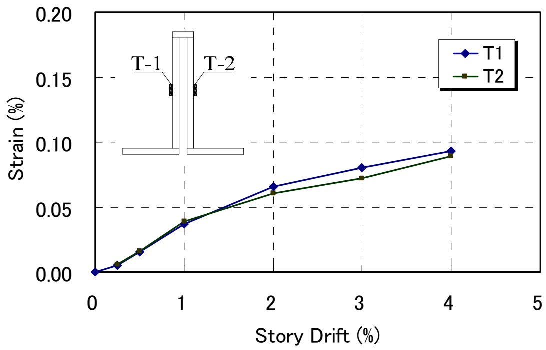


Figure 13 Strain of the U-shaped steel box

drift increased. This improves that the U-shaped steel box has worked well to transfer the beam shear force to the bracket.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

From the test results, following conclusions can be drawn.

1) The shear bracket and the inverted U-shaped steel box at the beam end on the column worked well to transfer the shear force from the beam to the column although some deformation still occurred within these parts.

2) In both specimens, the deformation concentrated on the beam end near the column face with the elongation of the PC bars and the damage of the beam concrete at the upper part of the beam near the column face. There was nearly no damage occurred with the column, especially in cases the shear bracket was used.

3) The vertical slip of the beam was excessively large in the specimen without shear bracket while it was significantly reduced in case with shear bracket.

4) The moment capacity of the beam is quite different between positive and negative side. From the visual observation, the "softening" of the upper part of the beam and vertical slip of the beam might be the main reason of the asymmetry of the moment capacity in positive and negative side.

4.2 Recommendations

From the test results, following recommendations are proposed:

1) The spiral steel should be used in the upper part of the beam end near column face to confine the concrete and avoid compression failure of the concrete.

2) The detail of the inverted U-shaped steel box should be modified to prevent bending deformation of the top horizontal plate and failure of the welding. Using the thicker plate and change the detail of this plate may help to prevent this type of damage.

3) The bracket should be checked for local concentrated stress at the top to avoid deformation of the bracket.

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