

## **Effect of connection reinforcement details on performance of full-scale precast concrete beam-column connection designed for gravity**

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### **Abstract**

This paper presents the test of full scale precast concrete beam-column connection under reversed cyclic load. Two full scale specimens representing the actual 5-storey precast concrete frame building without shear wall are tested. The structural design and reinforcement details of these two specimens considered gravity load only without taking into account horizontal force caused by earthquake. The two specimens have different details of connection between beam and column. In the first specimen, the connection is by means of the welded splice of beam longitudinal bars with additional short reinforcement piece. In the second specimen, the connection is achieved through the 90 degree hook of beam bar into the joint core. Experimental result demonstrated that both specimens exhibited different failure modes. The first specimen fails by shear failure at the end of beam, while the second specimen fails by flexural failure. The load-displacement relations show that both specimens are moderately ductile, with the ductility ratio of 3.5. The specimens can resist earthquake of moderate intensity even though they are designed for gravity load only. However, the hysteretic loops are pinched in both specimens, indicating low energy dissipation.

### **1. Introduction**

The development of precast concrete construction has continually expanded and became one of popular alternatives for construction recently. In Thailand, both precast beam-column frame system and bearing wall system have found wide structural applications. Traditionally, the design of concrete structures, both cast-in-place and precast systems, considers gravity and wind loads only. The seismic effect is not included in the design since there is no regulation to enforce the consideration of earthquake load. Recent seismic studies [1] indicated that damaging earthquakes can occur in South-East Asia regions. The most recent example was Sichuan Earthquake that caused violent shaking of several tall buildings in Bangkok though their epicenters were about 2000 km away. In the precast concrete beam-column frame system, the precast beam was connected to column by means of wet joint cast afterwards. The design of connection was mainly designed to assure that the beam was tightly fixed to column. No seismic effect was recognized in designing such connection so far. Since beam-column connection was very important in the lateral load path of the frame, the failure of beam-column joint may lead to catastrophic collapse of the building. Moreover, the precast structure is usually characterized by low redundancy and limited load path. Due to the piece by piece of nature construction of precast, the

performance of construction joint needed special investigation especially under seismic load. The significance of beam-column joint thus requires the consideration of seismic force. The objective of this paper is to examine the seismic performance of full-scale precast beam-column joint typical of gravity only design practice in Thailand. Since, no seismic effect is considered, no confining tied-bars are placed in the joint. The authors tested two specimens, typical of 5-story precast beam-column frame building without shear wall. This paper reports the behavior of these two specimens. The result of this study is expected to lead to the design improvement of precast beam-column joint for earthquake resistance.

## 2. Experimental Program

### 2.1 Specimens

The test consisted of two full scale cruciform-shaped specimens, representative of the first floor of 5 storey precast concrete frame buildings. The beam was precast specimen and the column was cast-in-place concrete. The preparation of specimen was conducted so as to follow the actual construction as much as possible as shown in Fig. 1. The dimensions of specimens as well as reinforcement details are shown in Fig. 2. In beam, six prestressing wires are used, in addition to 4 deformed bars, as main reinforcements. Thus, the beam is partially prestressed. The partial prestressing can help reduce the cracks in the joint and improve the ductility as compared to the conventional reinforced concrete frames.

In the laboratory, the bottom part of the column was cast first, then the precast beam was assembled to the column with approximately 30 mm seat length on the lower column.

The joint and top part of column were then cast together so as to connect beams to the joint. The dimension and reinforcement details of beam and columns are identical in both specimens, but the detail of connection was different. In specimen 1, the connection was achieved through welding lap splice of beam longitudinal bars as shown in Fig. 3a. The splice length was 50 mm on each side. In specimen 2, the connection was achieved through 90 degree hook in the joint. The top bars were hooked downwards and the bottom bars were hooked upwards as seen in Fig 3b. It was noticed that there were two tied bars of 9 mm diameter provided in joint core. The supply of these joint tied bars was arbitrary as there is no design rule to enforce such placement. In terms of bond, the connection of specimen 2 with 90 degree hook satisfies the ACI [2] development length whereas that of specimen 1 merely ties beam bars on two sides together without satisfying the ACI requirement [2].

### 2.2 Test setup and boundary condition

The experimental set-up is shown in Fig. 4. Both ends of the beam were supported by rollers that allow horizontal movement to simulate lateral drift. The bottom end of column was pin-supported at the base. The load was applied by hydraulic actuator at the top of column. The actuator was reacted against 500 kN reaction frame fixed to the strong floor. In order to simulate the axial force on column, prestressing tendons were stressed in the column to apply an axial force of 600 kN. Fig. 5 shows photo of specimen set-up in the laboratory.

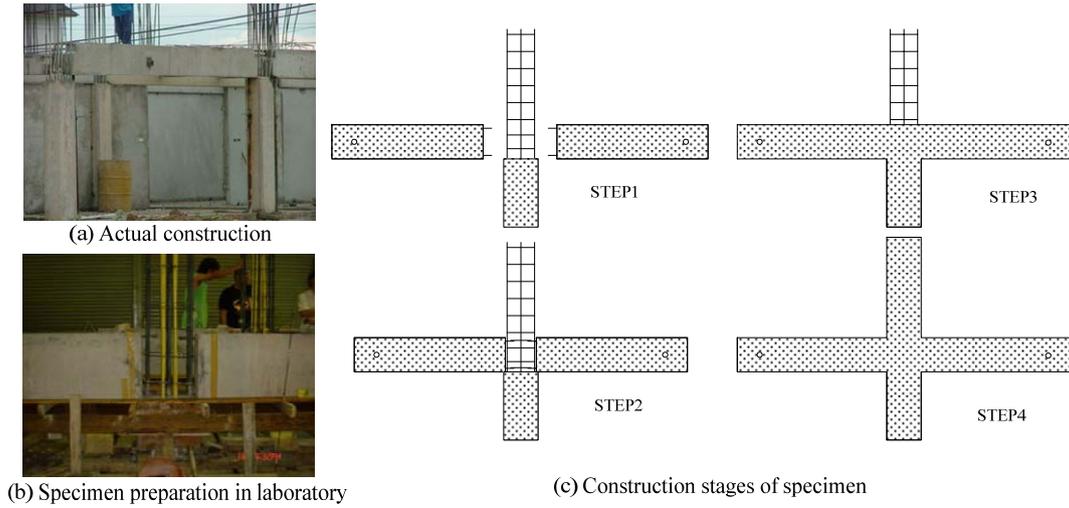


Figure 1 Specimen preparation

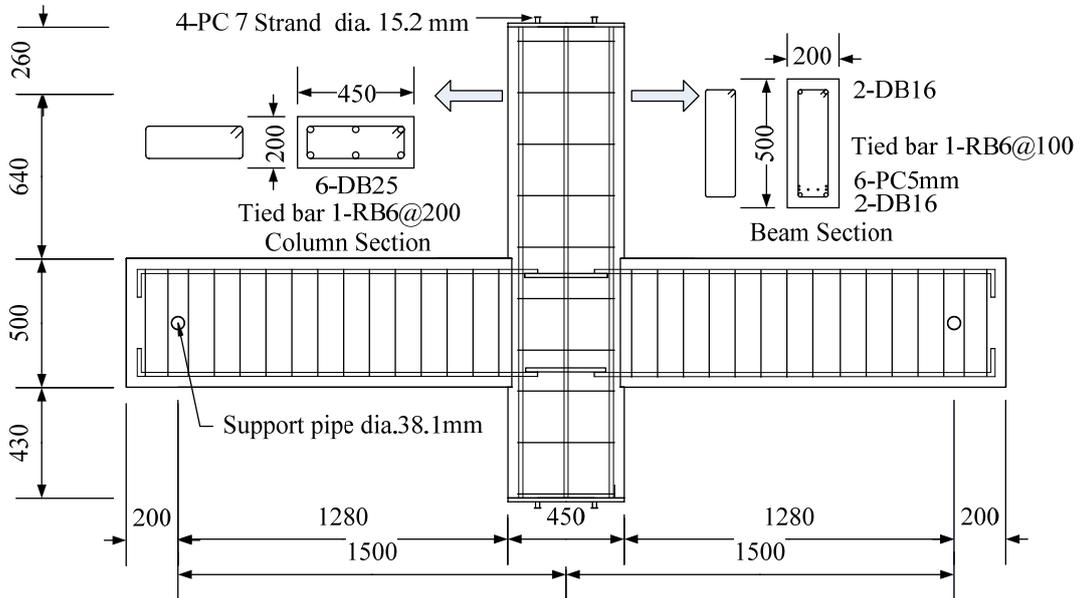


Figure 2 Dimension and reinforcement details of specimen

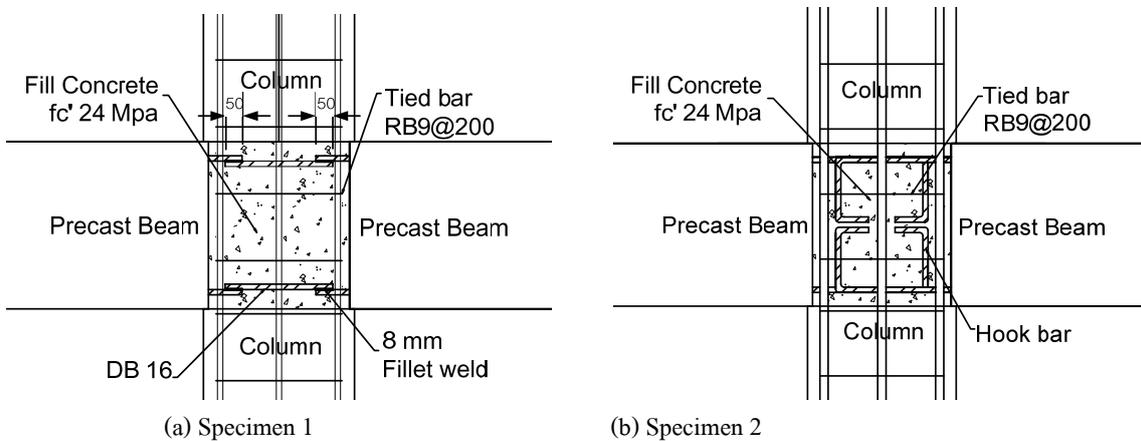


Figure 3 Reinforcement detail in the beam-column connection

**2.3 Material Properties**

The concrete used for casting specimens was ready mixed concrete obtained from nearby local plant. Table 1 lists the average tested compressive strength of concrete in various parts of specimens. Table 2 lists the average tested yield and ultimate tensile strengths of steel bars.

**Table 1 Tested properties of concrete**

Items	Average concrete compressive stress (MPa)			
	Lower column	Upper column	Beam	Joint
Specimen 1	49.8	47.5	46.1	29.3
Specimen 2	49.8	47.5	46.1	29.3

**Table 2 Tested properties of longitudinal steels and tied bars**

Type of reinforcing bar	Yield stress (MPa)	Ultimate stress (MPa)	Modulus of elasticity, (MPa)
DB 25 SD40	454	686	200,000
DB 16 SD40	474	706	200,000
RB 6 SR24	306	387	200,000

The load applied to the specimen was laterally cyclic displacement-controlled. The column was pushed forward and pulled backwards with increasing interstory drift ratios of  $\pm 0.25\%$ ,  $\pm 0.5\%$ ,  $\pm 0.75\%$ ,  $\pm 1\%$  and so on as shown in Fig. 6. At each drift ratio, the displacement was repeated twice to check the stability of the loop as well as to investigate the energy dissipation.

**3. Experimental results and discussions**

The test results are summarized in Table 3.

**3.1 General observation and cracking process**

(a) Specimen 1

At  $\pm 0.25$  drift ratio, the first observed crack was the vertical crack at the interface between precast beam and joint, i.e., at

construction joint as shown in Fig. 7. This crack lengthened more than half beam depth. At  $\pm 0.5$  drift ratio, the diagonal cracks occurred in the beams on both sides. At  $\pm 0.75$  drift ratio, the diagonal crack was found in the joint. During  $\pm 1.0\%$  to  $\pm 1.75\%$  drift ratios, the beam diagonal cracks expanded in number and size. During  $\pm 2.0\%$  to  $\pm 3.0\%$  drift ratios, the diagonal cracks in beam widened considerably on both sides and splitting cracks were also observed along longitudinal bars. The beam diagonal cracks were seen as intersecting X-shaped cracks at approximately half effective depth from the joint face. It was noted that there are comparatively less cracks in joint and column. At  $\pm 3.5\%$  drift ratio, the compressive area of the beam section close to the column crushed and spalled off. The beam longitudinal reinforcement could be seen obviously. As lateral drift increased further, the diagonal cracks became larger and caused failure to the specimen. It is interesting to note that the joint did not fail even though the compressive strength of joint concrete was only approximately 64% of that of beam.

(b) Specimen 2

At  $\pm 0.25$  drift ratio, the first observed crack was the vertical crack at the interface between precast beam and joint, i.e., at construction joint as shown in Fig. 8. At  $\pm 0.5$  drift ratio, the flexural-shear cracks were formed in the beams. At  $\pm 0.75$  drift ratio, the diagonal crack appeared at the joint. During  $\pm 1.0\%$  to  $\pm 1.50\%$ , the beam flexural-shear cracks expanded obviously while other cracks remained inactive. At  $\pm 1.75\%$  drift ratio, concrete at the joint corner and at the compressive side of the beam section began to spall off. Then the beam reinforcements became exposed. On the contrary, cracks in joint and column were nearly unchanged. At  $\pm 3.5\%$  drift ratio, the beam longitudinal reinforcement was fractured.

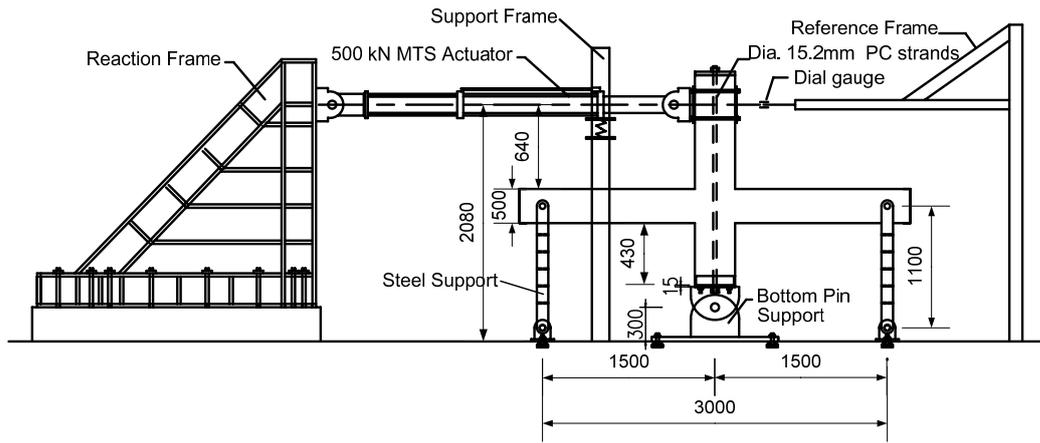


Figure 4 Test set-up

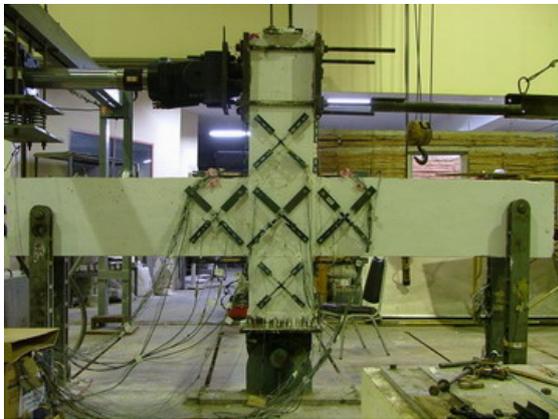


Figure 5 Photo of specimen set-up in laboratory

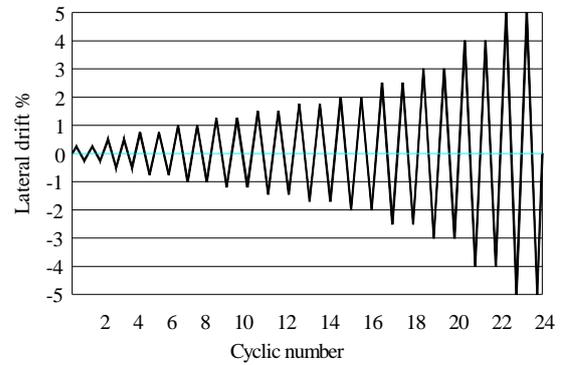


Figure 6 Load pattern

Table 3 Summary of test results

Properties	Specimen 1	Specimen 2
Yield strength	110 kN at 1.0 % drift	98 kN at 1.0% drift
Maximum strength	122 kN at 2.5 % drift	104 kN at 3.0% drift
Failure point	103 kN at 3.5% drift	98 kN at 3.5% drift
Remained strength	71 kN (58%) at 4.0% drift	96 kN (98%) at 4.0% drift
Mode of failure	Post yield shear failure in beam	Flexural failure
Cumulative energy dissipation at 3.5% drift	45.63 kJ	54.15 kJ
Ductility ratio	3.5	3.5

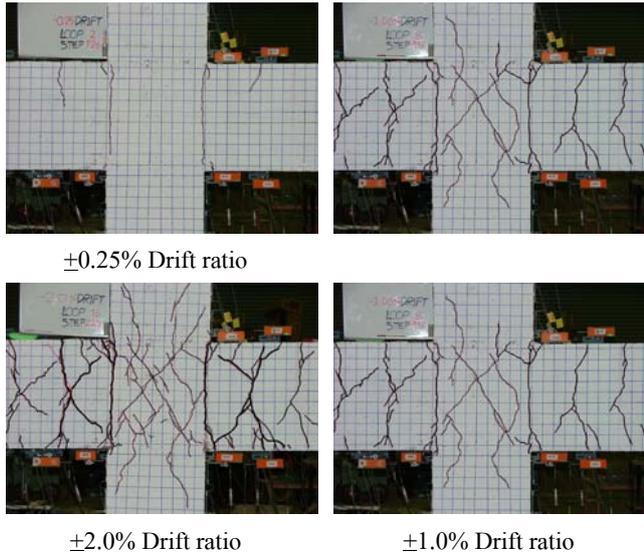


Figure 7 Cracking propagation in specimen 1

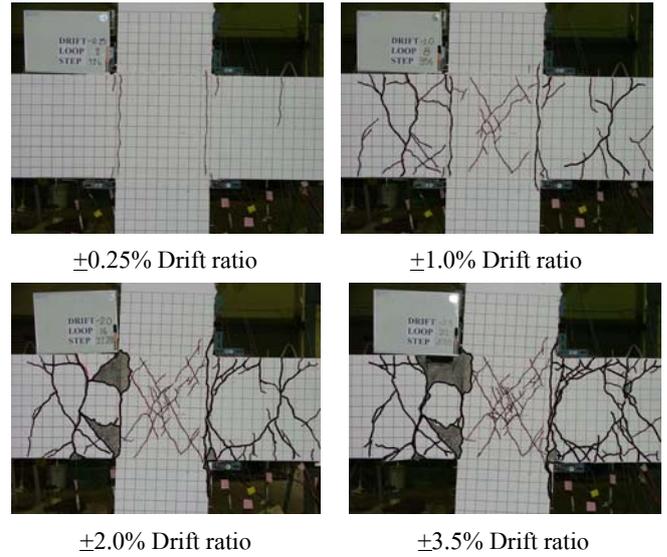


Figure 8 Cracking propagation in specimen 2

**3.2 Failure mode**

The test results showed that both specimens experienced extensive damages in beam at half effective depth ( $d/2$ ) from the column face. In contrast, the damage in column was slight. At the joint, intersecting diagonal cracks appeared but no joint concrete spalled off. The performance of joint and column seemed satisfactory. As for the construction joint, a large vertical crack was observed but did not lead to failure. The tendency of a separation at construction joint arose from the fact that the joint concrete and beam were cast at different times with different strength and shrinkage properties. This construction joint, if not adequately controlled, may lead to fixed end rotation and dissipate energy from pull-out nonlinearity. This was the case of specimen 1 where the welded lap splice could not provide sufficient prevention of construction crack propagation. Hence the hysteresis loops were pinched and narrow as will be seen in the next section. In case of specimen 2, due to the hook connection detail which prevented the growth of construction joint, a better cyclic behavior with wider loops was obtained.

For specimen 1, large diagonal cracks were observed in beam on both sides. The failure could be classified as post-

yield shear failure. For specimen 2, besides large shear crack, the beam section at column face was also subject to crushing. The failure of specimen 2 was classified as flexural failure with fracture of longitudinal steel bar. Observing the hysteresis loops of both specimens, it was clear that the loops of specimen 1 were more narrow and pinched than those of specimen 2. This pinching was attributed to bond slip of longitudinal beam bars. For specimen 2, the location of bar fracture was the same as the location of spot weld. Hence, it might be suggested that the spot weld reduced the ductility of longitudinal bars.

**3.3 Story shear – lateral drift relation**

**(a) Specimen 1**

The story shear versus lateral drift relation of specimen 1 is shown in Fig. 9. During 0% to  $\pm 0.5$  drift ratio, the specimen demonstrated elastic behavior. The yielding took place at  $\pm 1.0\%$  drift with the force of 110 kN. After yielding, the specimen could maintain lateral force until it reached peak load of 122 kN at  $\pm 3.0\%$  drift. The loop was stable up to the peak point. After peak load, the instability was obvious as noticed from the reduction in both strength and stiffness of

the repeated cycle. The failure of specimen 1 was caused by the propagation of beam diagonal cracks at approximately half effective depth from column face. The active extension of diagonal crack was obvious since  $\pm 1.5\%$  drift. At  $\pm 3.5\%$  drift, the remained strength was 84% of the peak strength. The ductility ratio was calculated to be 3.5. At  $\pm 4.0\%$  drift, the test was terminated and the specimen had the remained strength of 71 kN, which was 58% of the maximum strength.

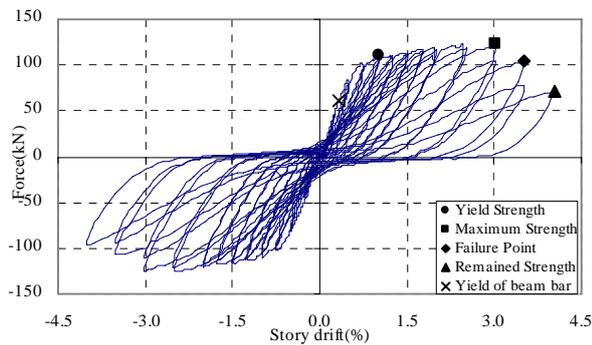


Figure 9 Story shear – lateral drift relation of specimen 1

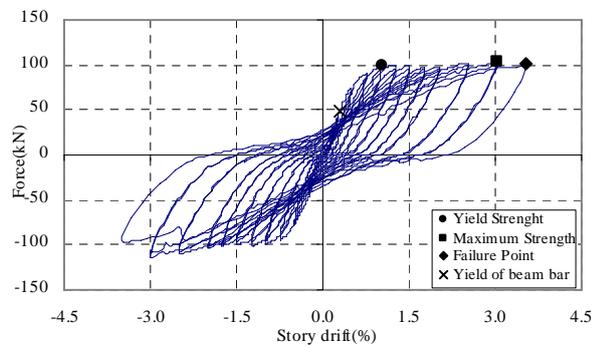


Figure 10 Story shear – lateral drift relation of specimen 2

(b) Specimen 2

The story shear versus lateral drift relation of specimen 2 is shown in Fig. 10. During 0% to  $\pm 0.5\%$  drift ratio, the specimen demonstrated elastic behavior. The yielding took place at  $\pm 1.0\%$  drift with the force of 98 kN. After yielding, the specimen could maintain lateral force until  $\pm 3.0\%$  drift ratio.

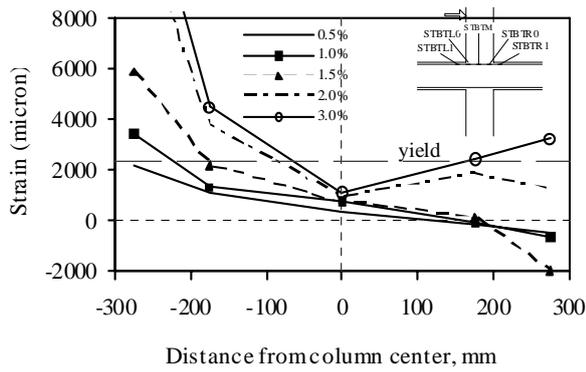
The hysteresis loops of specimen 2 were obviously larger than those of specimen 1. The better hysteresis performance was rooted in the 90 degree hook connection detail that prevented the slip of longitudinal beam bars. In terms of loop stability, the hysteresis remained stable throughout the entire loading since the repeated loop did show a drop in strength and stiffness.

3.4 Longitudinal beam bar strains

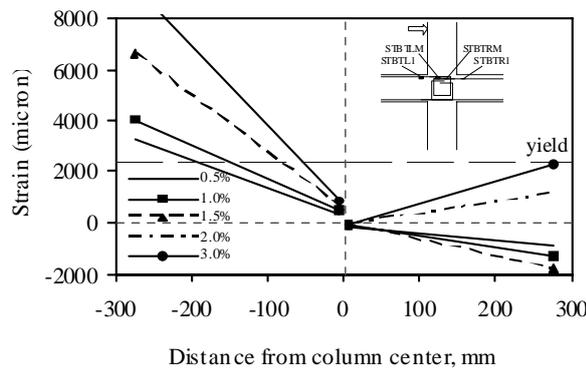
To gain an insight into the behavior of each specimen, local strains were measured at critical regions of the specimen. Figure 11 exhibits the strain profiles for the beam top longitudinal bars, measured at the peak displacements of each story drift ratio.

The strains of two specimens were similar to each other. Both specimens reached yielding and developed large plastic strain in beam region. For a given load direction, the steel was tensile (positive) strain on one side and compressive (negative) strain on the opposite side. The magnitude of compressive strain was generally lower since concrete carried compressive force partly.

At certain drift ratios, the compressive strain was gradually decreased and turned into tensile one. This was due to the local interface pull-out crack taken place in the previous opposite load direction. On reversed loading, the pull-out crack was not closed perfectly, thus, tensile strain remained in steel. As a result of this accumulative tensile strain, the steel on compression side became positive strain when the load was reversed.



(a) Specimen 1



(b) Specimen 2

Figure 11 Strain profiles for the beam top longitudinal bars.

When comparing the difference between the strain measured at the middle of joint and in beam sections at 50 mm away from column face, it exhibited obvious different magnitude of strain inclination. Thus, good anchorage bond is observed in both specimens. The welded lap splice with 50 mm lap length of beam passing through joint core in specimen 1 and the 90 degree hook of beam bar into the joint core in specimen 2 can develop good bond behavior. Another reason is column depth to bar diameter ratio ( $h_c / d_b$ ) of both specimens which calculated to be 28 according to the ACI code [3] minimum requirement of ( $h_c / d_b$ ) 20.

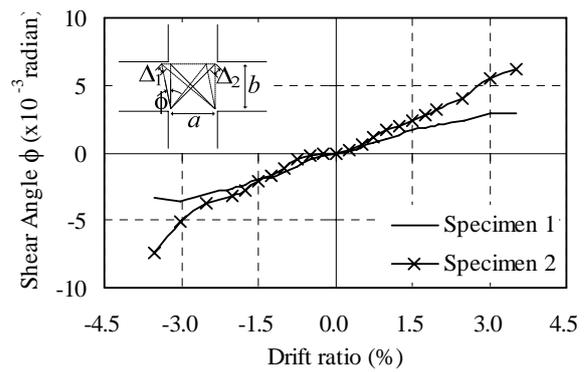


Figure 12 Relationship between joint shear angle and story drift

### 3.5 Joint shear deformation-story drift relation

The relation between joint shear deformation and story drift of both specimens was presented in Figure 12. It was apparent that the joint shear deformation of both specimens is almost same event though a different reinforcement detail in beam bar passing through joint core. The arbitrary transverse steel provided in joint of both specimens exhibited substantially lower joint shear deformation since the nonlinearity was mainly concentrated around the beam ends. The little higher joint shear deformation gave rise to the small pinching characteristics in specimens 1 while specimen 2 with discontinue of longitudinal beam bars in joint region had little wider loops as small damage observed in joint region.

### 3.6 Energy dissipation

The energy-dissipating capacities of beam-column connections are a function of the area under the load-displacement curve and indicate the degree of effectiveness of the connection to withstand earthquake loading. A desirable behavior for a beam-column sub-assembly under cyclic loading requires a sufficient amount of energy dissipation without a substantial loss of strength and stiffness.

Due to the different material strengths of the specimens, the most practical means to compare the energy

dissipation was to plot the dimensionless cyclic dissipated energy against the story drift [3]. The dimensionless quantity of cyclic energy dissipated was determined by dividing the energy dissipated per cycle by four times the product of the maximum experimental load and the maximum displacement for that cycle. The denominator is multiplied by four because this will then yield a rectangular perfectly elastoplastic loop, and the values on the ordinate axis represent, as a ratio, the energy dissipated for that cycle to the elastoplastic loop. In Fig. 13, the normalized cyclic energy was plotted against the story drift and best-fit curves were drawn through these points. It is clear that, specimen 2 dissipates more energy than specimen 1 for all story drift ratio.

The total cumulative energy dissipation of specimen 1 from the beginning till  $\pm 3.5\%$  drift was calculated as the total area under strong shear-drift ratio curve and is equal to 45.63 kJ. The energy dissipation was low and the hysteresis loops were pinched. As mentioned, the pinching behavior was mainly caused by pull-out rotation of longitudinal beam bars as well as diagonal cracks in beam. The total cumulative energy dissipation of specimen 2 up to  $\pm 3.5\%$  drift ratio was calculated to be 54.15 kJ. The failure of specimen was due to fracture of longitudinal bar and crushing of concrete.

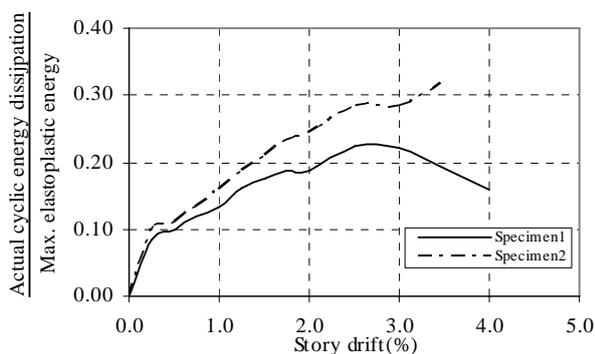


Fig. 13 Normalized cyclic energy dissipation versus story drift relation

#### 4. Conclusion

This research presents a cyclic load test of full-scale precast beam-column connection typical of gravity design in low-rise buildings. From the test program, it was found that the failure of both specimens was moderately ductile though they are built from precast members without consideration of earthquake effect. The construction joint at the junction between precast beams and cast-in-place column does not affect the strength and maximum displacement but could lead to low energy dissipation if it is not adequately controlled. The reinforcing detail of connection has a considerable impact on the failure mode and energy dissipation characteristics. The welded lap splice with 50 mm lap length was not adequate to prevent bar slip. The 90 degree hook provides better bond performance. The good bonding condition leads to high energy dissipation and flexural failure mode.

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