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SEISMIC PERFORMANCE OF BEAM-COLUMN JOINT CONNECTIONS WITH REDUCTION IN TRANSVERSE REINFORCEMENTS IN RIGID-FRAMED RAILWAY BRIDGES BY USING PP-ECC

R. Zhang(1), K. Matsumoto(2), T. Hirata(3), Y. Ishizeki(4), J. Niwa(5)

(1) *Lecturer, School of Civil Engineering, Southwest Jiatong University, rayz430@home.swjtu.edu.cn*
(2) *Project Assistant Professor, Institute of Industrial Science, the University of Tokyo, km312@iis.u-tokyo.ac.jp*
(3) *Researcher Engineer, Technical Research Institute of Obayashi Corporation, hirata.takayoshi@obayashi.co.jp*
(4) *Researcher Engineer, Technical Research Institute of Obayashi Corporation, ishizeki.yoshikazu@obayashi.co.jp*
(5) *Professor, Department of Civil Engineering, Tokyo Institute of Technology, jniwa@cv.titech.ac.jp*

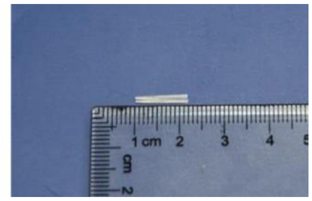
Abstract

Bridges are vital components of transportation that requires a high degree of protection to ensure their safety during a strong earthquake. The extensive damages of RC bridges observed in the past earthquakes such as Northridge, Kobe and 2011 Great East Japan earthquakes triggered extensive researches on the behavior of beam–column joint connections for designing and constructing a safer infrastructure, which further resulted in the improvements of design codes focusing on providing sufficient ductility in the vulnerable structural member to prevent its brittle failure during a major seismic event. Accordingly, for reinforced concrete (RC) structures, a considerable amount of steel reinforcements are required to be provided in these vulnerable regions, such as the plastic hinge in the beam end adjacent to the column face in a beam–column joint connection in the rigid-framed railway bridges, to confine the concrete to realize the formation of ductile inelastic behavior in the plastic hinge. However, the increased and elaborated reinforcement details bring the difficulties in fabricating this complicated steel reinforcement cage as well as placing and consolidating concrete in it during the construction phase. The contradiction between increased high cost for design and construction due to these complicated reinforcements with accordingly raised requirements on seismic performance becomes more and more apparent. In this research, a cementitious composite combined with fabricated polypropylene fibers named Polypropylene Fiber Reinforced Engineered Cementitious Composites (PP-ECC) with improved bond properties exhibiting the pseudo strain hardening and multiple fine cracking of ECC was utilized to reduce the transverse reinforcements in beam–column joint connections of rigid-framed bridges. Compared with widely used polymer fibers such as polyvinyl alcohol (PVA) fibers or polyethylene (PE) fibers, polypropylene (PP) fiber is softer, costs lower and disperses faster, which all results in better workability. In addition, because of the hydrophobic and non-polar nature of PP fiber, PP-ECC has better durability in an alkaline environment. The loading tests including two phases were conducted: the basic mechanical properties of PP-ECC in compression and tension were confirmed by cylinder compression and uniaxial tensile tests in the first phase and a total of three one-sixth scaled T-shaped beam–column joint connections which were prepared based on the design standards for existing railway bridges in Japan were tested under the applied reversed cyclic load to verify the possibility of reducing transverse reinforcements. The experimental results reveal that the PP-ECC is effective in replacing transverse reinforcements in the beam-column joint connections of railway rigid-framed bridges without deterioration of seismic performance.

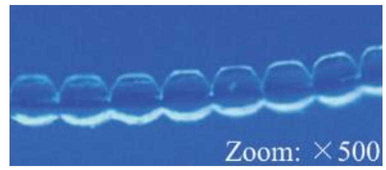
Keywords: PP-ECC, beam-column joint, stirrup ratio, flexural failure, railway bridge

Fig. 1 – Rigid-framed railway bridges

(a) A cluster of fibers



(b) Length of fiber



(c) Cross section view under a microscope

Fig. 2 – Polypropylene fiber in this study

1. Introduction

Bridges are vital components of transportation that requires a high degree of protection to ensure their safety during a strong earthquake. However, reinforced concrete (RC) bridges suffered extensive damage in the past earthquakes such as Northridge, Kobe, Wenchuan and 2011 Great East Japan earthquakes. It triggered extensive researches on the behavior of beam-column joint connections for designing and constructing a safer infrastructure, which further resulted in the improvements of design codes focusing on providing sufficient ductility in the vulnerable structural member to prevent its brittle failure during a major seismic event. Accordingly, for RC structures, a considerable amount of steel reinforcements are required to be provided in these vulnerable regions, such as the plastic hinge in the beam end adjacent to the column face in a beam-column joint connection in the rigid-framed railway bridges (Fig. 1). However, the increased and elaborated reinforcement details bring the difficulties in fabricating this complicated steel reinforcement cage as well as placing and consolidating concrete in it during the construction phase.

A newly developed fiber-reinforced cement-based material named Engineered Cementitious Composites (ECC) exhibits multiple fine cracking, pseudo strain hardening behavior, large strain capacity between 1% and 5% and superior ductility. Its superior strain capacity makes it an ideal material for use in the plastic hinge of beam-column joint connections to undergo large inelastic deformation and reduce the quantity for transverse reinforcements. The steel reinforced ECC (R/ECC) structural members such as column with reduction of shear reinforcements [1] have been confirmed in previous studies. However, the amount of previous research on applying ECC in the beam-column joints was limited and mainly focuses on the interior beam-column joints in the buildings. The previous research [2], [3] have revealed the feasibility of total elimination of transverse reinforcements in the joint and increasing stirrup spacing in beam plastic hinge in beam-column joins constructed with Polyethylene Fiber Reinforced Engineered Cementitious Composites (PE-ECC). In this research, a cementitious composite combined with fabricated polypropylene fibers (Fig. 2) named Polypropylene Fiber Reinforced Engineered Cementitious Composites (PP-ECC) with improved bond properties exhibiting the pseudo strain hardening and multiple fine cracking of ECC [4] was utilized to reduce the transverse reinforcements in beam-column joint connections of rigid-framed bridges. Compared with the other polymer fibers, polypropylene (PP) fiber is softer, costs lower and disperses faster, which all results in better workability. In addition, because of the hydrophobic and non-polar nature of PP fiber, PP-ECC has better durability in an alkaline environment [5]. The previous research of PP-ECC beams with varying shear reinforcement ratios [6] has indicated the possibility of eliminating of shear reinforcement. In this research, a total of three one-sixth scaled T-shaped beam-column joint specimens which were prepared based on the design standards for existing railway bridges in Japan [7] were tested under the applied reversed cyclic load to verify the possibility of reducing transverse reinforcements.

2. Mechanical properties of the PP-ECC

2.1 Production of PP-ECC

The target nominal compressive strength of the PP-ECC is 30 N/mm^2. The material components and mixture proportion used in this investigation were based on a study by Hirata et al. [4]. The mix proportion of PP-ECC was tabulated in Table 1. The PP fibers, as shown in Fig. 2, are fibrillated fibers having diameter of 36 μm, length of 12 mm, tensile strength of 482 MPa and elastic modulus of 5 GPa. This fibrillated polypropylene fiber with rugged surface results in improvement of bond properties and exhibits the pseudo strain hardening and multiple fine cracking of ECC under tensile stress [4.

2.2 Compressive characteristics

The PP-ECC cylinders with diameter of 100 mm and height of 200 mm were prepared for compression tests. The compressive characteristic of PP-ECC and normal strength concrete in this study are compared in Table 2. Different from the normal concrete in mixture, since the PP-ECC in this study uses no coarse aggregates instead of PP fibers, the average value of elastic modulus of the PP-ECC cylinders is 1.5×104 N/mm2 which is lower than that of the normal strength concrete while the elastic modulus of the concrete with compressive strength of 30.0 N/mm2 is 2.8×104 N/mm2 in this study. However, as shown in Table 2, the compressive strength of the PP-ECC in this study is almost equivalent to that of the normal strength concrete.

2.3 Tensile characteristics

Tensile behavior was investigated by employing the uniaxial tensile method in this study. Fig. 3(a) shows the dimensions and setup of specimens for tensile tests. Two linear variable differential transformers (LVDTs) setting parallel to the loading direction at both sides of the plate as shown in Fig. 3(b), were used to measure the axial tensile deformation. The speed of load head was selected as 0.1mm/min. Tensile stress versus strain curves of the three specimens are shown in Fig. 4. The test result clearly shows typical pseudo strain hardening behavior of

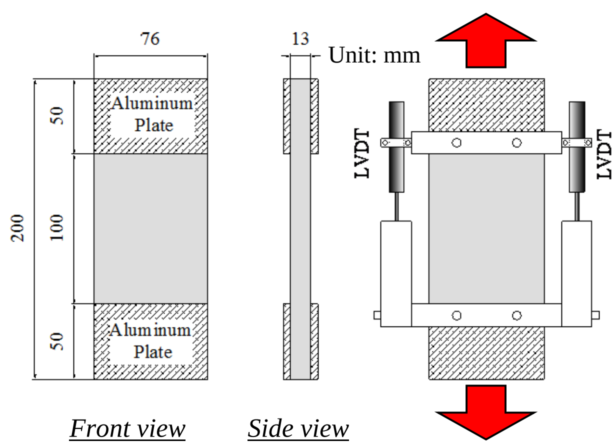
Table 1 – Mix proportion of PP-ECC

Slump flow (mm)	*W/B* (%)	*FA/B* (%)	Unit weigth (kg/m^3)			
			W	*B*	PP fiber	AE
Approx. 500	27	33	371	1400	27	7

W: water; *B*: binder; *FA*: fly ash; AE: air-entrainment.

Table 2 – Compressive characteristics of PP-ECC and concrete.

Material	Compressive strength (N/mm^2)	Compressive strain (%)	Elastic modulus (N/mm^2)
PP-ECC	33.6	0.4	1.5×10^4
Concrete	30	0.2	2.8×10^4



(a) Dimensions of specimen. (b) Setup of tests.

Fig. 3 – Uniaxial tensile tests of PP-ECC.

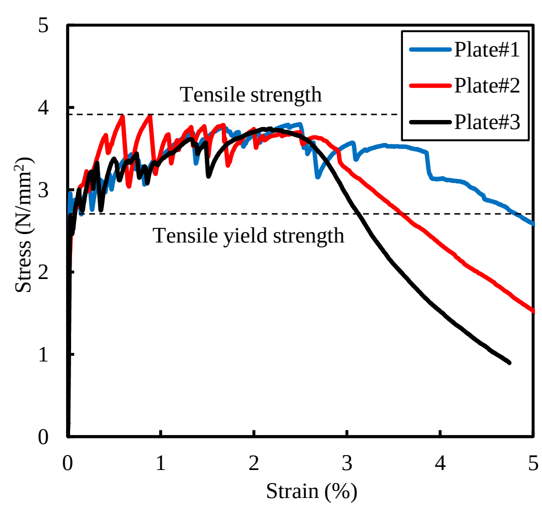


Fig. 4 – Results of uniaxial tensile tests.

ECCs. The tensile strength is defined as the maximum stress in the tensile stress-strain curve obtained from uniaxial tensile tests. In this study, the yield and tensile strength were greater than 2.5 and 3.5 N/mm2, respectively.

3. Experimental program of the beam-column joints

3.1 Test specimens

In the loading tests of beam-column joint specimens, an existing railway bridge designed following the Japanese code, "Design Standard for Railway Structures and Commentary (Concrete Structures)" [7], was considered as a prototype structure in this study. Since this type of rigid-framed railway bridge is far stiffer in longitudinal direction but vulnerable in transverse direction to earthquakes, the frame structure in transverse direction was selected as the study target. Totally three specimens with different amount of transverse reinforcements were constructed on one-sixth scale, corresponding to a beam-column joint connection in the prototype bridge formed by three inflection points under an idealized laterally applied seismic load, in which two points locates at the middle points of the column section above and below the intermediate beam and another one locates at the middle point of the intermediate beam as illustrated by Fig. 5. Fig. 6(a) shows a sketch of the test setup used in this study and overall specimen details. The height of a column was considered as 1500 mm and the length of a beam from the face of a column to the end was considered as 900 mm. The column cross section was 250 mm×250 mm, and the beam was 170 mm wide and 200 mm deep as shown in Fig. 6(b). The thickness of cover concrete was 20 mm. The transverse reinforcements in the joint were eliminated for all specimens and the amounts of longitudinal reinforcements in beams and columns were constant in these three specimens. In the specimen TJ-1, the longitudinal and transverse reinforcements were provided in the beam and column according to the prototype bridge but without ties in the joint. Based on the specimen TJ-1, the stirrups in testing span of the beam were eliminated in the specimens TJ-2, while the amount of longitudinal and ties in the column was kept unchanged. In the specimen TJ-3, the amount of transverse reinforcements not only in the beam but also in the column was reduced to the minimum for fabricating a reinforcement cage. PP-ECC would be used in the beam-column joint and the beam region in TJ-1 and TJ-2 while the normal concrete would be used in the column region. However, only PP-ECC would only be used in TJ-3 specimen. Reinforcements with nominal diameter of 6.35 mm and yield strength of 325 N/mm2 were used for both longitudinal and transverse reinforcements in all three specimens. The mix proportions of normal concrete and PP-ECC used for the all beam-column joint specimens as well as their compressive strength are tabulated in Table 3 and Table 4, respectively.

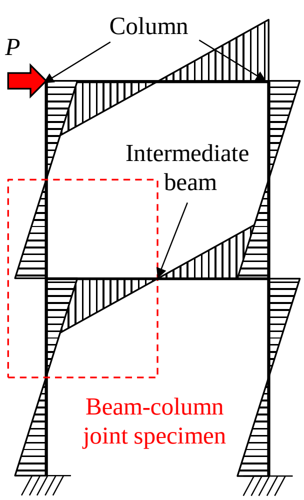
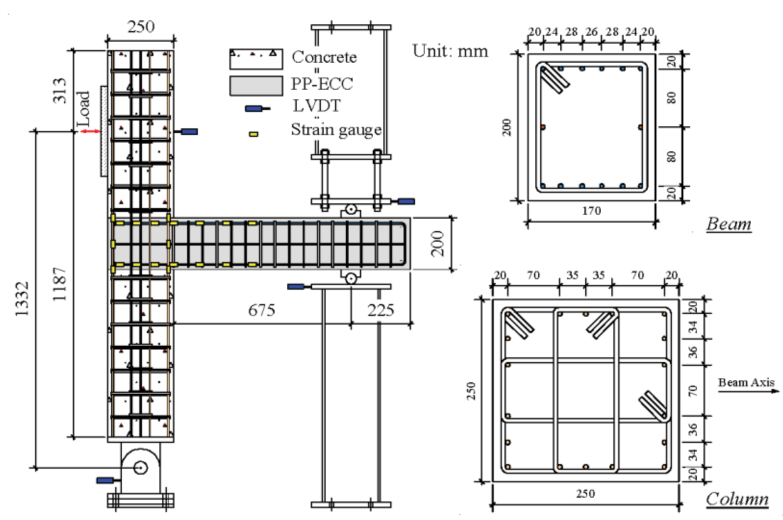


Fig. 5 – Bending moment diagram under the seismic force



(a) Specimen dimensions

(b) Cross sections of columns and beams

Fig. 6 – Specimen dimensions

Table 3 – Compressove strengths of PP-ECC in specimens

Specimen	TJ-1	TJ-2	TJ-3
f'_{ECC} (N/mm^2)	48.2	33.6	33.6

Table 4 Properties of concrete in specimens

Specimen	f'_c (N/mm^2)	G_{max} (mm)	W/C (%)	Unit weigth (kg/m^3)				
				W	C	S	G	Superplasticizer
TJ-1	50.0	15	60	169	281	830	896	2.82
TJ-2	45.2	15	60	169	282	830	896	2.82

G_{max}: themaximum size of greval; S: sand; G: gravel.

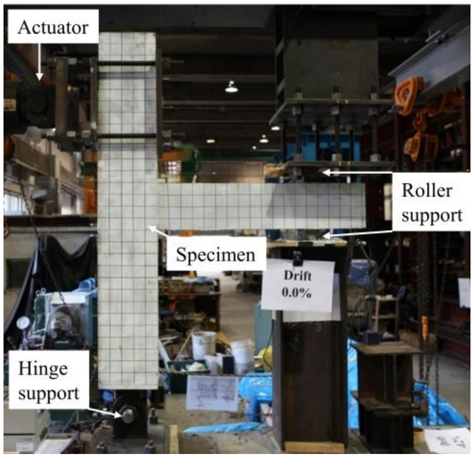


Fig. 7 – Test setup

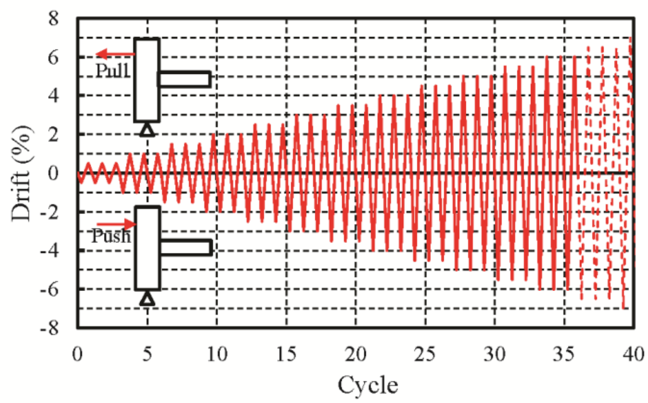


Fig. 8 – Lateral displacement history

3.2 Experimental setup and procedure

All specimens were tested under a reversed cyclic load provided by a digital closed-loop controlled hydraulic loading system. The experimental setup is shown in Fig. 7. The reversed cyclic lateral displacement controlled loading was applied on the column. The bottom of the column was pinned to a strong loading frame to simulate middle height inflection point during a seismic event. At the beam length of 675 mm from the column face, two roller supports were fixed at the locations as illustrated in Fig. 6(a) to simulate middle length inflection point of intermediate link beam. The applied displacement history included 36 reversed displacement cycles ranging from 0.5% to 6.0% drift, with three cycles performed at each drift level, as shown in Fig. 8. When the column was pulled towards the actuator, the displacement was considered as positive displacement and vice versa. The drift level is defined as the ratio of the lateral displacement to the height from the hinge to the level of the applied lateral load.

4. Experimental results

4.1 Crack pattern and failure process of beam-column joints

Fig. 9 shows the crack patterns of all specimens observed after testing. The first fine flexural cracks initiating from the bottom edge of beams for all specimens were observed at the first cycle in the drift of 0.5%. The large number of cracks appeared before the peak loads while few cracks were developed after the peak load. The localized cracks in all specimens were noticed in the top of beam adjacent to the column face after the peak load and developed to be more and more obvious due to their opening and closing under the increased cyclic loads. In all specimens, except very limited numbers of fine flexural cracks were observed in TJ-1 and TJ-3, most of the fine flexural cracks were developed in the beam span from the column face to the roller supports and concentrated in the beam near the column face leading to the formation of flexural plastic hinge in the beam. Meanwhile, different amounts of fine inclined shear cracks were developed in the beam-column joint in all specimens. On increasing the applied drift, with opening and closing of shear cracks in the joint core region, the joint distortion and expansion continued to increase. The shear cracks adversely affected the bond between the PP-ECC and the steel reinforcements leading

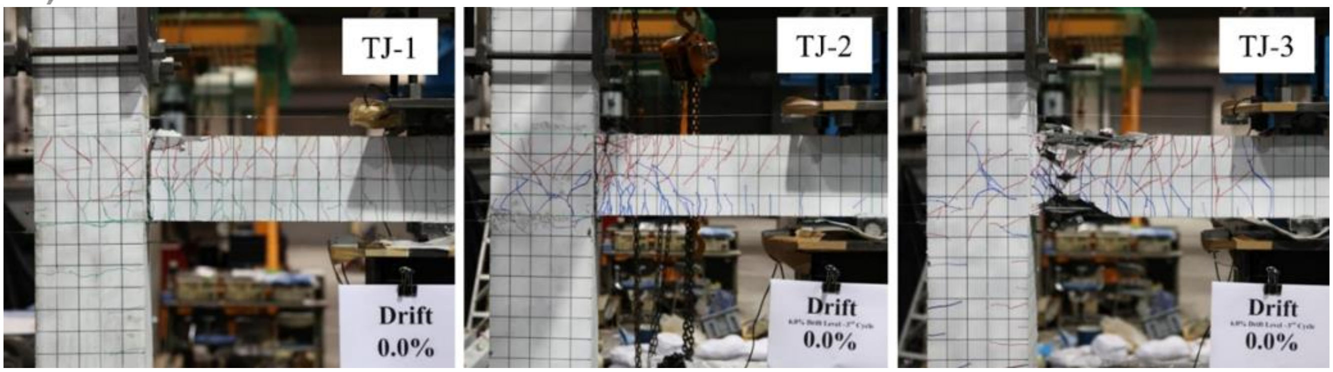


Fig. 9 – Crack pattern after loading tests

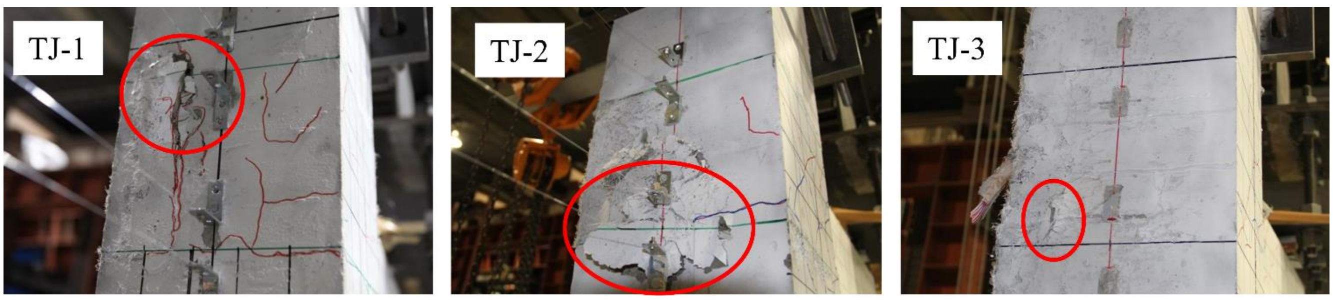


Fig. 10 – Pull out of beam longitudinal reinforcements after loading tests

to the bond deterioration. Without the occurrence of localized shear cracks in all specimens, the sufficient shear strength of joints provided by PP-ECC can be developed even without tie bars in the joints, thereby allowing the formation of plastic hinges in the beams. The similarity of the failure process of TJ-1 and TJ-2 indicated that the feasibility that the PP-ECC was able to be the replacement of stirrups in the beam. Different from TJ-1 and TJ-2, the crushing of partial PP-ECC with fewer fractions of PP fibers on the view side in the plastic hinge region in TJ-3 occurred at the peak load due to the inadequate distribution of PP-ECC fibers. As for the effects of this crushing, it would be discussed in the following section.

Fig. 10 shows the pull out of beam longitudinal reinforcements on the external face of a column in all specimens after the loading test due to the slipping between beam longitudinal reinforcements and the PP-ECC in the joints. Different extents of pull out of reinforcements in all specimens were observed. There were pulling out of top beam reinforcements in TJ-1 as shown in Fig. 10(a), but all beam reinforcements buckled and five out of six top beam reinforcements ruptured by the end of the loading test. The similar damage that the pull out of bottom beam reinforcements as shown in Fig. 10(b) in TJ-2 was also observed, but all top beam reinforcements buckled and only two out of six top reinforcements ruptured. The pull out of beam reinforcements in TJ-3 as shown in Fig. 10(c) was quite insignificant compared to the specimens TJ-1 and TJ-2, manifesting that the bonding between beam reinforcements and the PP-ECC in the joint of TJ-3 was the best and finally two out of six top beam reinforcements and four out of six bottom beam reinforcements ruptured. This is because the limited space between beam reinforcements and the interface in the column impaired the bonding between reinforcements and the PP-ECC.

4.2 Load-displacement hysteretic results in beam-column joint tests

The load-displacement hysteretic loops and envelops obtained from the cyclic loading tests are shown in Fig. 11. The hysteretic loops of all specimens were pinched to an equivalent level. The dashed lines with green color in each subfigure indicate the peak loads of each specimen in positive and negative loadings. The peak load of TJ-1 in positive loading cycles was the highest among all specimens, which was mainly attributed by its higher compressive strength of PP-ECC among all specimens. It is noted from Fig. 11 that three specimens performed similarly within 2.0% drift. On applying the cyclic load in TJ-1 after the peak load at 3.0% drift, the load dropped drastically when TJ-1 was loaded towards 4.5% drift in the first cycle due to the ruptures of beam top longitudinal

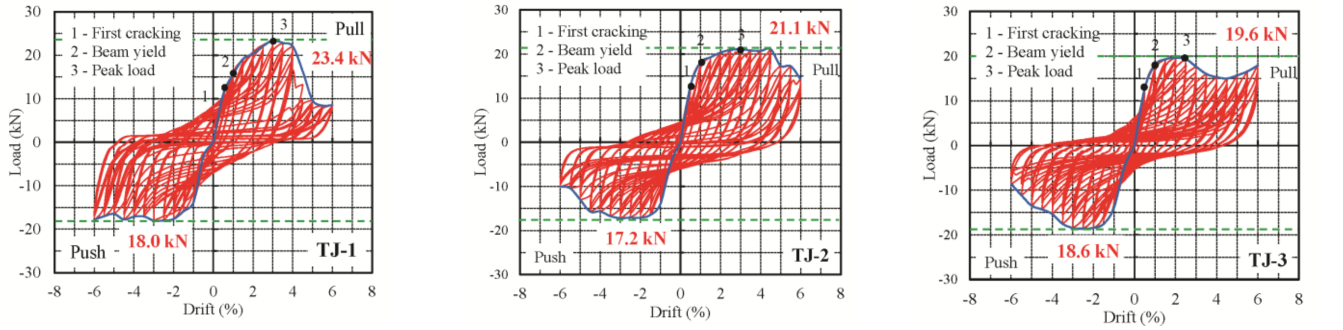


Fig. 11 – Load-displacement hysteretic loops

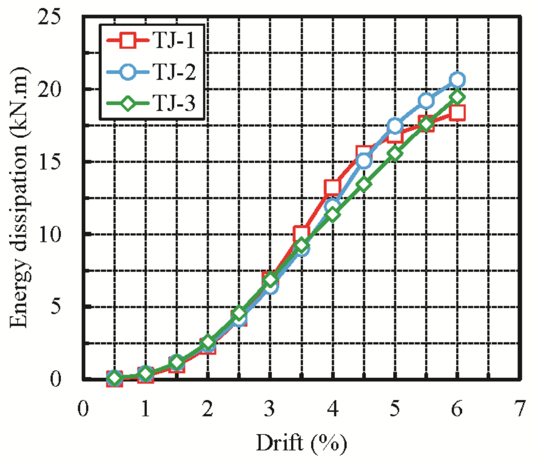


Fig. 12 – Energy dissipation

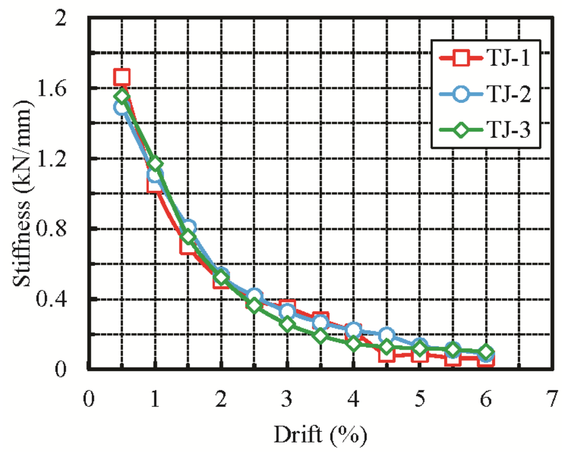


Fig. 13 – Stiffness degradation

reinforcements. On reversing the load, only the beam bottom longitudinal reinforcements participated to carry the load. However, after the peak load, the strength marginally decreased in subsequent cycles up to 4.5% drift forming a plateau in load-displacement envelope curve of TJ-2 and slightly decreased from 2.5% to 4.5% but increased from 4.5% to 6.0% forming a flat basin in the load-displacement envelope curve of TJ-3 due to inelastic behavior of plastic hinge adjacent to the column face and the bond deterioration in the joint region.

4.3 Energy dissipation and stiffness degradation of beam-column joint tests

Energy dissipation indicates the capability of structures to dissipate energy through yield mechanism with satisfactory performance in the inelastic range, which occurs due to induced damages in the specimens in terms of cracking of concrete, yielding and buckling of steel reinforcements and debonding of fibers in ECC. Fig. 12 shows energy dissipation capacity of each specimen. Energy dissipation was assessed by computing the cumulative energy dissipation at each load cycle, namely, the area enclosed by the corresponding load-displacement hysterical loops. Due to the pinching and strength degradation in all specimens, the energy was not proportionally increased to the increase in the applied drift. Up to the drift of 3.0%, all specimens dissipated almost the same amount of energy. At the drift level of 4.5%, TJ-1 dissipated 15.7% more energy than the specimen without transverse reinforcements TJ-3. The growth of energy dissipation in TJ-1 slowed down markedly due to the rupture of top beam reinforcements in TJ-1 at the drift level of 4.5% while fewer ruptures of beam reinforcement in of TJ-2 and TJ-3 at this drift. As a result, the TJ-2 and TJ-3 dissipated more energy than that of TJ-1. Among all specimens, the energy dissipated by TJ-2 and TJ-3 was almost equivalent to that dissipated by TJ-1. The comparable energy dissipation even after reducing the amount of transverse reinforcements in the specimens TJ-2 and TJ-3 also highlighted the shear reinforcing effectiveness of PP-ECC.

Fig. 13 shows the stiffness degradation of all specimens during the cyclic loading in the positive direction, which was assessed by computing the slope of the line connecting the peak load and zero load at half cycle of each drift level. Even with the elimination of transverse reinforcements in TJ-2 and TJ-3, TJ-2 and TJ-3 exhibited the comparable performance of TJ-1, indicating that the little effect on the stiffness degradation due to the reduction of transverse reinforcements by using PP-ECC. In addition, it was noted that although the crushing of the defective PP-ECC on the view side in TJ-3 was observed during the loading tests, the comparable energy dissipation and stiffness degradation exhibited by TJ-3 indicates that the effect resulting from this defective PP-ECC was ignorable.

5. Conclusions

1. The reduction of transverse reinforcements in the beam, the column and the joint improved the workability and increased economy of using PP-ECC.

2. The failure mode of the specimens even with elimination of transverse reinforcements was still flexural failure, which was the same as that of the specimen without elimination of transverse reinforcements indicating that the PP-ECC can be the replacement of transverse reinforcements to provide sufficient shear strength.

3. The peak loads of PP-ECC joint specimens with elimination of transverse reinforcements remained comparable to the specimen without elimination of stirrups in the beam without shear failure, indicating that the PP-ECC can act as transverse reinforcements to carry the applied load.

4. Sufficient ductile behavior could be achieved even with reduction of the transverse reinforcements by using PP-ECC.

5. The specimens with the reduction of transverse reinforcements by using PP-ECC dissipated more energy than that without elimination of transverse reinforcements.

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