Performance of Dry Exterior Beam-Column Joints Using CFRP Bolts and SFRC under Cyclic Loading

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Abstract

Prefabrication construction has been intensively studied and commonly used in practice because of its numerous advantages. One advantage is that it greatly accelerates the construction by assembling prefabricated structural components onsite, significantly reduces the site disruption, especially when the prefabricated components are connected with dry joints. However, one problem with the dry joint construction is the corrosion of steel bolts and plates normally used at dry joints. Without protection by a concrete cover, the steel components at joints are prone to corrosion, which increases the lifecycle maintenance costs of structures and may even negatively affect the integrity of the structures. This study proposes and investigates the performances of a new type of dry exterior beam-column joints for precast moment-resisting concrete frames by using corrosion-resistant carbon fibre reinforced polymer (CFRP) bolts and steel fibre reinforced concrete (SFRC). This is the first time in the literature that both CFRP bolts and SFRP were applied to improve the ductility of precast joints. Three dry exterior concrete joints and one monolithic joint were cast and tested under quasi-static cyclic loads until failure. The proposed joints exhibited excellent performance in terms of the load-carrying capacity, ductility, and energy dissipation as compared to the monolithic joint. The ductility of the proposed precast joints was even higher than that of the monolithic joint. The drift ratio of these proposed joints also exceeded 3.5%, which satisfies the requirements for ductile joints to be applied in earthquake-prone regions according to several internationally recognised standards such as ACI T1.1-01, ASCE 41-06, and CSA A23.3-07. The proposed precast joints possess some obvious advantages compared to the conventional monolithic joint and precast joints with conventional steel connectors, such as shorter construction time, lower construction cost, better recyclability and excellent corrosion resistance.

Key words: fibre reinforced polymer (FRP) bolts; steel fibres; ductile precast joint; prestress bolts; exterior dry joint; cyclic load; concrete-end-plates.

1. INTRODUCTION

Beam-column joints serve as a crucial component of a building to ensure the integrity and overall stability when the building is subjected to a seismic loading (Zhao et al. 2019). Under earthquake loading, numerous inclined cracks occur and cause brittle shear failure in beam-column joints. This brittle shear failure results in numerous serious consequences because it often occurs suddenly without any warnings before the total collapse of structures (Le-Trung et al. 2010; Zhao et al. 2019). The consequences of this dangerous brittle failure were observed in various recent devastating earthquakes around the world, such as the 1999 Kocaeli (Turkey), the 1995 Hyogo-ken (Japan), and the 1999 Chi-Chi (Taiwan) earthquakes. The brittle shear failure is unexpected due to non-ductile performance and is attributed to either lack of transverse reinforcements or inadequate anchorage in the joint region. Meanwhile, shear stress often considerably concentrates at these zones (Abbas et al. 2014; Liu 2006; Paul and Melvin 1989; Shafaei et al. 2014). Therefore, it is necessary to improve the ductility of beam-column joints.

Most of the relevant current studies concentrated on examining the structural behaviours of monolithic, wet or hybrid joints (Ascione and Berardi 2011; Bahrami et al. 2017; Kaya and Arslan 2009; Le-Trung et al. 2010; Mostofinejad and Akhlaghi 2017; Parvin and Granata 2000; Singh et al. 2014; Yekrangnia et al. 2016) because these joint types offer various advantages such as high load-carrying capacity, stiffness, energy dissipation capacity, and especially excellent ductility under seismic loadings. However, these joint types have also revealed some disadvantages including high construction time and construction cost. Actually, most of these shortcomings could be overcome if these joint types have been replaced by dry joints. In dry joint, mechanical connections are used to assemble prefabricated structural components, so it does not require formworks. Specimens are cast in a factory and then delivered to construction sites for the assembling process (Hassan et al. 1996; Prabhakaran et al. 1996; Saqan 1995). In addition, the quality control in the construction process is easily conducted and components (i.e., beams and columns) could be more easily and cost-effectively recycled (Hanaor and Ben-Arroyo 1998; Hassan et al. 1996; Kaya and Arslan 2012; Nzabonimpa et al. 2018; Palmieri et al. 1996; Prabhakaran et al. 1996; Saqan 1995). Moreover, the damaged structures can be more easily dismantled and replaced, hence make the structures more resilient. In spite of their advantages, the application of dry joints in reality is limited because the dry joint designs have some weaknesses, such as insufficient strength, ductility, and vulnerability to corrosion damage. Among these shortcomings, corrosion is the most costly issue and the main cause of structural damage. Corrosion usually occurs in connecting elements (i.e., steel tendon strands, bolts) of conventional dry joints because they are not protected by concrete. Corrosion of the connecting components could lead to serious deteriorations or even collapse of the whole building whereas other parts are still in a good condition (Clifford 1991; Woodward and Williams 1988; Wouters et al. 1999). For instance, Kitane et al. (2004) estimated that the average annual cost of improving, repairing, and maintaining bridges in the United States of America could reach approximately $5.8-$10.6 billion during the period of 1998 to 2017. More problematically, in some cases, the maintaining and repairing costs of damaged components could be twice as much as the original ones (Lawler and Polak 2010; Yunovich and Thompson 2003).

There have been only two experimental studies investigating the beam-column joint performances using steel bolts and the concrete-end-plate in the literature (Hanaor and Ben-Arroyo 1998; Saqan 1995). Most recently, Ngo et al. (2019a); (Ngo et al. 2019b) proposed the use of CFRP bolts to replace these steel bolts considering the fact that steel bolts in the connections are susceptible to corrosion. As reported by Ngo et al. (2019a), the inclined cracks in the middle zone of the concrete-end-plate cause the main failure pattern of this precast joint type. This study, therefore, investigates the use of Steel Fibre Reinforced Concrete (SFRC) to minimize the inclined cracks on the concrete-end-plate and thus improve the peak load, energy dissipation, ductility, drift ratio, and stiffness of these joints.

2. EXPERIMENTAL PROGRAMME

As previously mentioned, to investigate the effects of steel fibres and CFRP bolts on the structural response of precast beam-column joints, three dry exterior beam-column joints and one monolithic specimen were cast and tested under quasi-static cyclic loads until failure. These specimens include (1) Monolithic; (2) CFRP-Spiral-NoFibre; (3) CFRP-NoSpiral-Fibre; and (4) Steel-Spiral-NoFibre which were named base on their characteristics. “CFRP” and “Steel” denote the use of CFRP bolts and steel bolts, respectively. Spirals inside the concrete-end-plate of Specimen CFRP-NoSpiral-Fibre were replaced by steel fibre with a volume fraction of 1%. The average compressive strength (f'_c) and tensile strength (f_{ct}) on the testing day were 32.3 MPa and 4.3 MPa for SFRC while these of conventional concrete were 38.4 MPa and 3.8 MPa, respectively. It should be noted that Specimen CFRP-NoSpiral-Fibre was cast by a different concrete batch from the other three specimens. All the precast specimens were investigated with low prestress levels of 6.5-10.5 kN. Figure 1 shows the geometry and reinforcement details of the monolithic and precast specimens. All the specimens were tested under cyclic loading with displacement control. Two load cycles were conducted at each drift ratio with the displacement rate of 6-9 mm/min. Figures 2 and 3 show the loading history and schematic setup for the testing process.

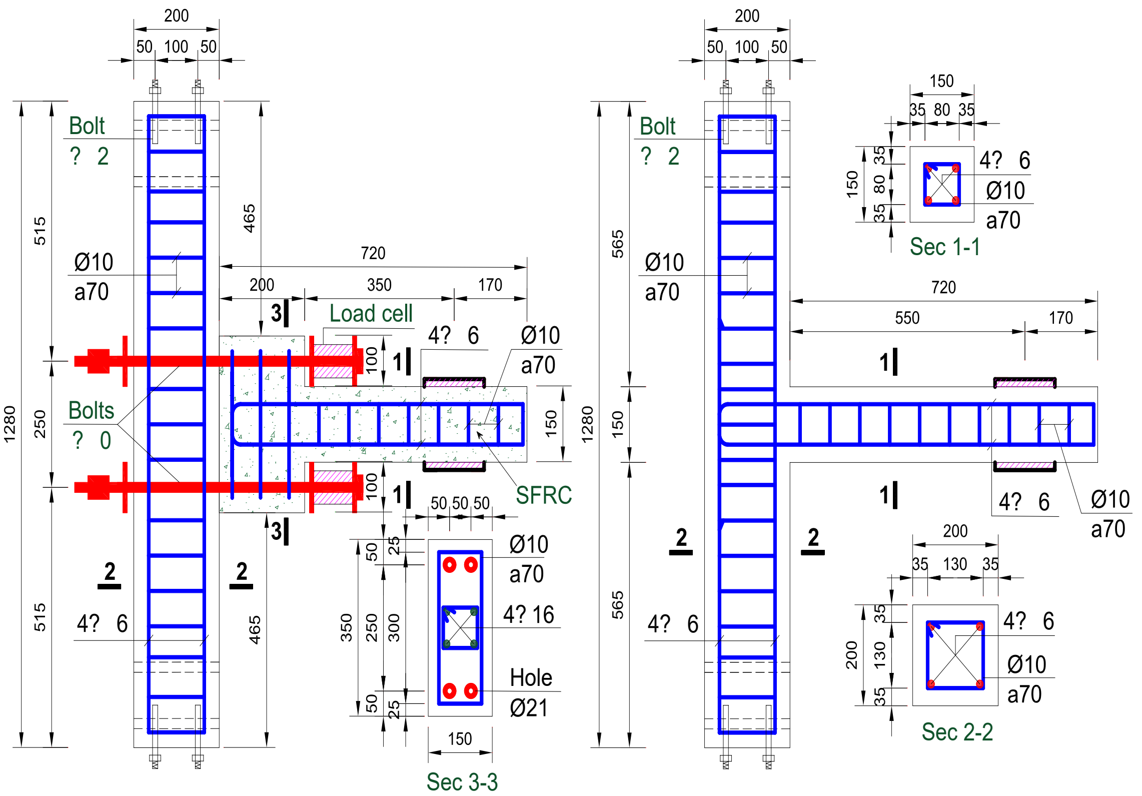

Figure 1. Designs of the precast specimen (Left) and monolithic specimen (Right) (unit: mm).

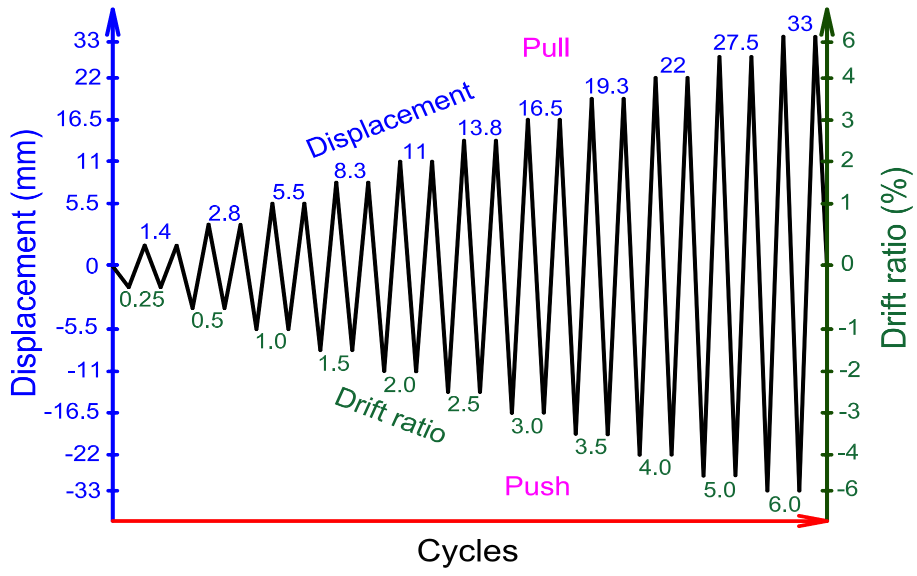

Figure 2. Cyclic loading history.

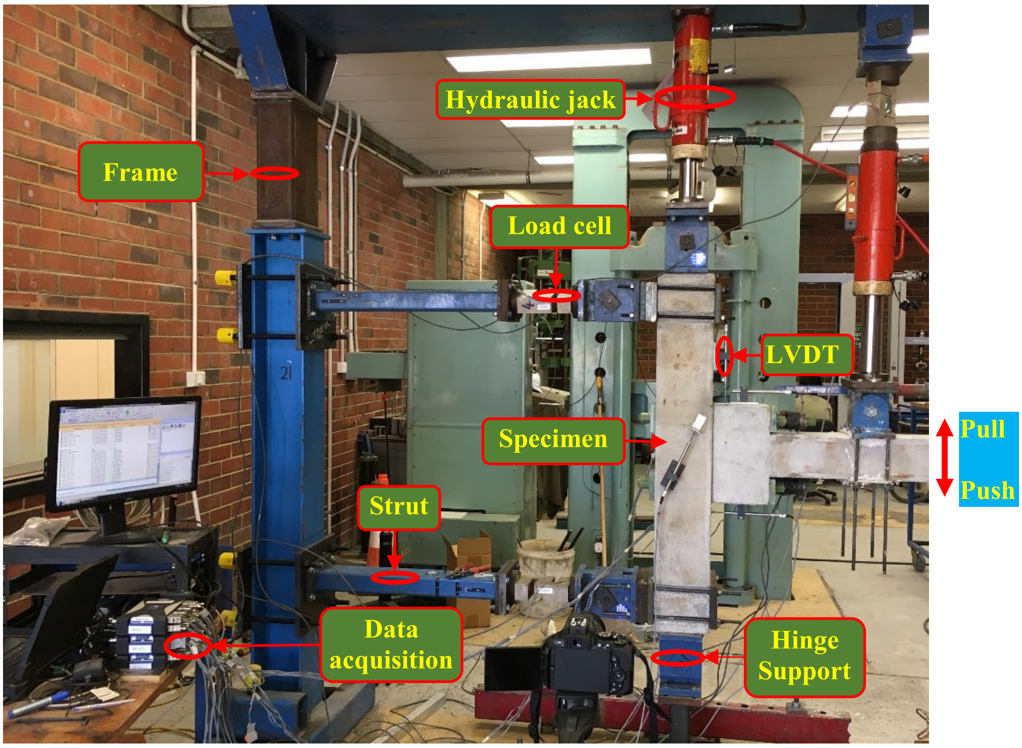

Figure 3. Details of the test setup.

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1. Drift ratio and ductility

Drift ratio and ductility are crucial parameters to evaluate the joint behaviours under earthquake loadings. The drift ratio is defined as the ratio of the vertical displacement of the beam to the distance from the loading point to the column face while the ratio of the ultimate displacement to displacement at the yield loads is calculated as the ductility of a structure. In most of the previous studies, the drift ratio of precast joint usually achieves between 1.5 to 3% (Jin et al. 2016; Saqan 1995) while the requirements for the building to satisfy the life safety are approximately 2% according to ASCE 41-06 (2006), 2.5% in CSA A23.3-07 (2007), and 3.5% according to ACI T1.1-01 (2001). This current study uses steel fibre and steel spiral to improve drift ratio and ductility. It is noted that Saqan (1995) reported the maximum drift ratio of only 1.5%. In the current study, Specimens CFRP-Spiral-NoFibre and Steel-Spiral-NoFibre achieved 3% drift ratio while Specimen CFRP-NoSpiral-Fibre reached 3.5% drift ratio which satisfy most of the current standards for application to aseismic designs.

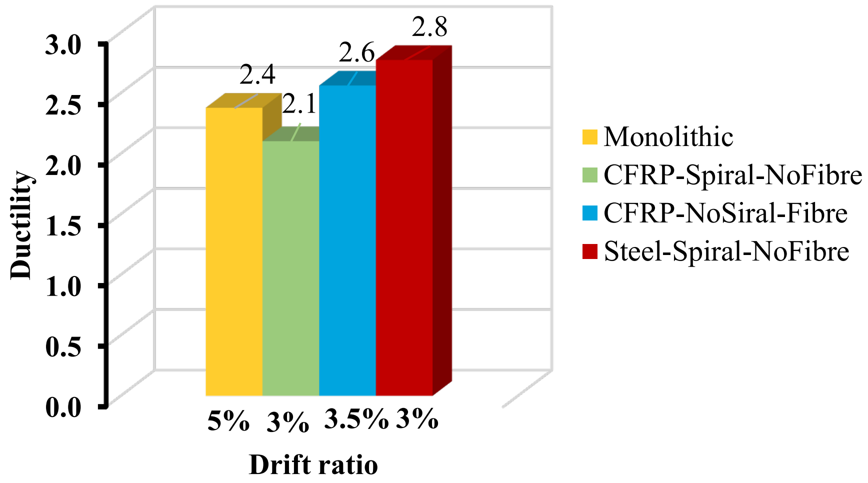

Figure 4. Comparison of ductility and drift ratio of all the specimens.

Figure 4 shows the comparison of ductility and drift ratio of all the tested specimens. According to Park (1989), the buildings are attributed to ductile structures if they can dissipate significant amount of energy during inelastic cyclic deformations. The results present that Specimen Steel-Spiral-NoFibre exhibited the highest ductility (μ=2.8) among all the tested specimens. Specimens CFRP-NoSpiral-Fibre and Steel-Spiral-NoFibre revealed higher ductility than Specimen Monolithic (μ=2.4), with an increase of approximately 8.3% and 16.6%, respectively while the ductility (μ=2.1) of Specimen CFRP-Spiral-NoFibre were similar to the reference Specimen Monolithic. It is worth mentioning that the ultimate displacement of Specimen CFRP-Spiral-NoFibre was determined at 90% of the peak loads. Therefore, it is expected that if the ultimate displacement was monitored at 85% of the peak loads, the ductilities of this specimen would be similar or even higher than that of Specimen Monolithic. The precast specimens exhibited excellent ductility and drift ratio due to the beneficial influences of steel fibres and steel spirals inside the concrete-end-plate. Therefore, it could be concluded that this precast joint type could be effectively applied in non-seismic and seismic-prone areas.

3.2. Load-carrying capacities

Figure 5 shows the envelope curves of all the tested specimens. The average load-carrying capacity of Specimen CFRP-NoSpiral-Fibre (43.3 kN) was greater than those of Specimens CFRP-Spiral-NoFibre (36.8kN) and Steel-Spiral-NoFibre (39.8kN). This result could be explained that SFRC had higher tensile strength (4.3 MPa) than conventional concrete (3.8 MPa). Therefore, the inclined cracks on the concrete-end-plate, which caused the main failure of this precast joint type (Ngo et al. 2019a), were effectively minimized. Meanwhile, the maximum applied loads of all the precast joints were higher than that of the monolithic joint. Although Specimen Monolithic revealed a ductile load-displacement performance with the highest drift ratio of 5%, the maximum applied load of this specimen (29.1 kN) was the lowest among all the tested specimens. For example, the maximum applied loads of Specimens CFRP-Spiral-NoFibre, Steel-Spiral-NoFibre, and CFRP-NoSpiral-Fibre were 27%, 37%, and 49% greater than that of Specimen Monolithic, respectively. This result is attributed to the effects of the concrete-end-plate. If the failure occurred at the fixed-end, the above result might be explained that the square cross-section at the fixed-end of all the specimens is similar (150×150 mm^2) whereas the lever arm between the loading point and the fixed-end of the monolithic joint (550 mm) was longer than that of all the precast joints (350 mm). Consequently, the moment at the fixed-end of the monolithic joint was higher than that of the precast joints with the same applied load. For the failure in the middle zone of the concrete-end-plate, the maximum applied loads of all the precast joints were governed by the thickness and the height of the concrete-end-plate. This study purposefully modified the thickness from the design of the previous study by Saqan (1995). Consequently, the maximum applied loads of the precast joints were considerably improved compared to the reference specimen.

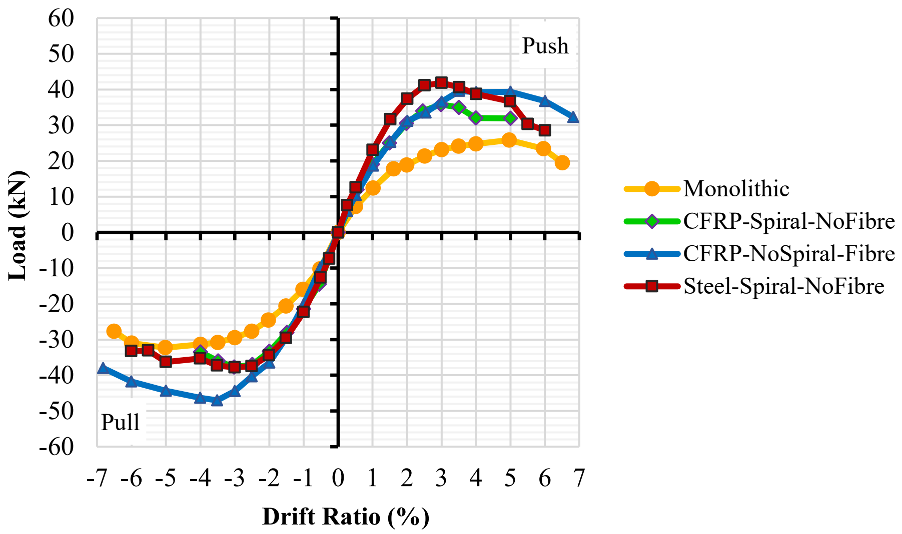

Figure 5. Load-drift ratio envelopes of all the specimens.

3.3. Energy dissipation capacities

The energy dissipation capacity is an essential parameter to evaluate how effective a joint withstands the seismic loading. A beam-column joint under quasi-static cyclic loads is attributed to the ductile joint if enough amount of energy is dissipated without a considerable reduction of its strength and stiffness (Vidjeapriya and Jaya 2013; Xue and Zhang 2014). This parameter is determined as the area enclosed (A_h) inside the load-displacement hysteretic loop in that corresponding load cycle. The energy dissipation comparisons of all the tested specimens are shown in Figure 6. The energy dissipation capacities of all the specimens revealed similar trends and values up to 1% drift ratio since they behaved elastically in the initial stage. However, the overall energy dissipation capacity of Specimen CFRP-NoSpiral-Fibre was lower than the other precast specimens between 1% and 3.5% drift ratio. After reaching the maximum applied load at 3.5% drift ratio, the energy dissipation capacity of Specimen CFRP-NoSpiral-Fibre dramatically increased until failure. This favourite phenomenon might be explained that inclined cracks in the concrete-end-plate were effectively minimized due to the contributions of steel fibres from the beginning of the test to 3.5% drift ratio. After this stage, steel fibres were progressively pulled out from the matrix which caused the high inelastic deformation and various cracks. Therefore, the dissipated energy and toughness of this specimen were considerably improved compared to other specimens. As expected, the energy dissipation capacities of all the precast joints (CFRP-Spiral-NoFibre, CFRP-NoSpiral-Fibre, and Steel-Spiral-NoFibre) were higher than that of Specimen Monolithic approximately 45.1%, 65.9%, and 65.3%, respectively, which could be mainly attributed to fatter hysteretic loops. These results support the conclusion that the proposed precast joints exhibited excellent energy dissipation capacity for seismic loading resistance.

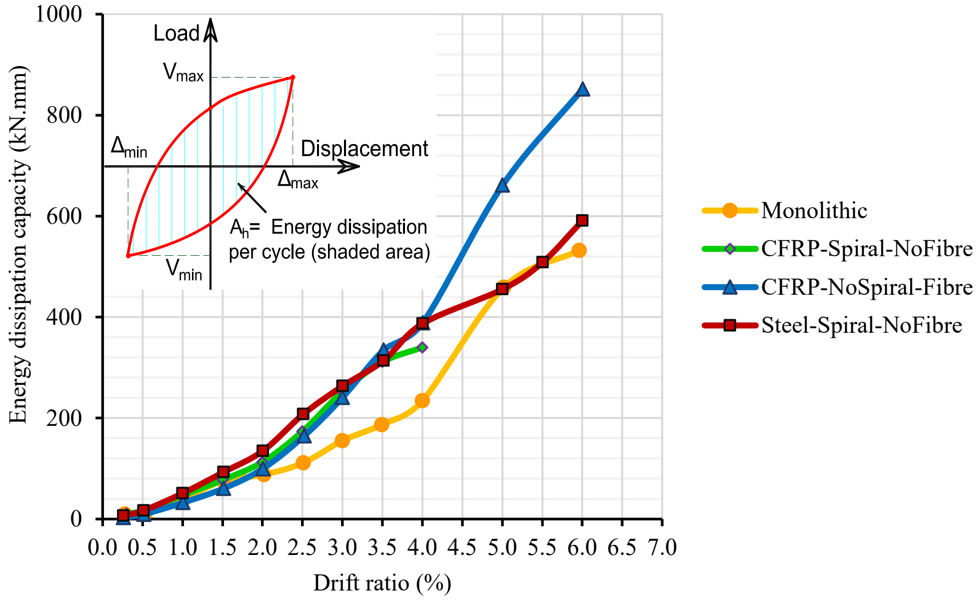

Figure 6. Comparison of energy dissipation capacity.

4. CONCLUSIONS

This study proposes and investigates the structural performance of a new kind of dry beam-column joint made of SFRC and connected by using CFRP bolts. The excellent performances of the test results demonstrated that the proposed dry joints outperformed the monolithic joint in term of ductility, maximum applied load, and energy dissipation capacity. The maximum applied loads of all the dry joints were from 27% to 49% greater than that of the monolithic joint and the ductility was almost the same or even higher than the reference specimen. In addition, using SFRC considerably improved the maximum applied load and ductility by approximately 18% and 53%, respectively. In general, CFRP bolts could effectively replace steel bolts to resolve a very costly issue of corrosion while still ensure excellent behaviours under seismic loading.

5. ACKNOWLEDGEMENT

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