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Effects of Steel Fibres and Prestress Levels on Behaviour of Newly Proposed Exterior Dry Joints Using SFRC and CFRP Bolts

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Abstract

This study proposes a new type of dry exterior beam-column joints for precast moment-resisting concrete frames. This dry joint type uses steel fibre reinforced concrete (SFRC) and carbon fibre reinforced polymer (CFRP) bolts to improve the joint capacities. In addition, an analytical model to predict the load-carrying capacity of this precast joint type is also proposed. Five exterior beam-column joints were cast and tested under quasi-static cyclic loads until failure. The experimental results revealed that the use of SFRC significantly improved all the indices, including the load-carrying capacity, drift ratio, ductility, energy dissipation and stiffness. Also, the proposed joints outperformed the monolithic specimen in terms of load-carrying capacities, energy dissipation, and stiffness by 27%-61%, 45%-75%, and 27%-55%, respectively. Particularly, the drift ratio of the proposed joints reached 3.5%, which satisfies the requirements for ductile joints to be used in earthquake-prone regions according to various standards. Finally, the proposed model yielded good predictions as compared to the experimental results with minor errors of approximately 0.9%-2%. These exciting results indicate that the use of SFRC and CFRP bolts could help to avoid the challenging issue of corrosion in the conventional dry exterior joints and still ensure the sufficient requirements for reinforced concrete structures in non-seismic and seismic-prone areas.

Keywords: Fibre Reinforced Polymer (FRP) bolts; Steel fibres; Ductile precast joint; Prestress bolts; Exterior dry joint; Cyclic load; Concrete-end-plates.

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1. Introduction

Beam-column joints serve as an important component of a reinforced concrete (RC) structure to guarantee the integrity and overall stability when the frame is subjected to a cyclic lateral load [1]. Under seismic loading, shear stress significantly concentrates at beam-column joints whereas in some cases transverse reinforcements are not sufficient to resist this shear stress [2, 3]. Therefore, various inclined cracks in two directions develop and cause brittle shear failure in beam-column joints. This brittle shear failure causes numerous serious consequences since it often occurs suddenly without any warnings before the total collapse of buildings [1, 4]. Various recent devastating earthquakes across the world have demonstrated this dangerous brittle failure, such as the 1995 Hyogo-ken (Japan), the 1999 Kocaeli (Turkey), and the 1999 Chi-Chi (Taiwan) earthquakes and many more. The brittle shear failure is an undesirable result of non-ductile performance, caused by either inadequate anchorage of the main reinforcing bars or a shortage of transverse reinforcements in the joint [5, 6]. Therefore, it is crucial to improve the ductility of beam-column joints.

Most of existing studies focused on investigating the structural performances of monolithic, wet or hybrid joints [4, 7-13] concerning their good performances in terms of strength, stiffness, energy dissipation capacity, and especially ductility under cyclic loadings. Nevertheless, the use of these joints has revealed some shortcomings, such as higher construction time and construction cost. Actually, most of these disadvantages could be overcome with the application of dry joints. Dry joints use mechanical connections to assemble prefabricated structural components and, thus, do not require formworks [14-16]. Also, these joints require fewer efforts on the construction quality control. Additionally, it is easier and more cost-effective to have dry joints directly recycled [14-20]. Despite their advantages, the use of dry joints in reality is still limited due to their remaining disadvantages, such as inadequate ductility, strength, and vulnerability to corrosion damage. Among them, corrosion is the most costly issue and is the main cause for structure deterioration, i.e. it has been estimated that the average annual cost of maintaining and improving bridges in the United States of America could respectively reach $5.8 billion and $10.6 billion during the period of 1998 to 2017 [21]. Corrosion is more likely to occur in traditional dry precast joints since such connecting elements as steel bolts, plates, and tendon strands are not covered and protected by concrete. Corrosion of the connecting elements might lead to serious damage or even collapse of the entire structures even though other parts of the building are still in a good condition [22-24]. More problematically, in some cases, the repairing and maintaining costs of damaged members could be twice as much as the original ones [25, 26].

To improve the performance of dry joints, Ngo et al. [27] proposed dry joints with concrete-end-plate and CFRP bolts which showed excellent performance in all the important indices such as the load-carrying capacity, energy dissipation, ductility and the stiffness compared to conventional monolithic joints. Replacing steel bolts with CFRP bolts helps to resolve the corrosion issue effectively. These promising results indicated that those proposed precast joints could be potentially applied to prefabrication constructions in non-seismic and seismic-prone areas. Also, the results showed that the main failure of this joint type was caused by inclined cracks induced by shear stresses [27]. To avoid such failure, the contemporary design standards [28-31] require a high percentage of stirrups in the concrete-end-plate. These requirements could lead to steel congestion, construction difficulties, and size increase of the concrete-end-plate. Meanwhile, in order to ensure the architectural requirements, it is necessary to reduce the size of the concrete-end-plate which might cause the slip and anchorage failure of the beam’s longitudinal reinforcements.

Considering the above issues, fibre reinforced concrete (FRC), as later proposed and investigated in the current study, might be a potential solution. FRC offers higher tensile strength, greater shear resistance, higher toughness, better bond, and greater seismic resistance than conventional concrete [2, 5, 32-37]. It is well-known that conventional concrete exhibits excellent behaviours in compression but weak performance in tension. This disadvantage could be resolved by the inclusion of suitable fibre volume to improve the pre-cracking and post-cracking performance [35-37].

Fibres are popularly used in FRC in four types, including steel fibres, glass fibres, carbon fibres, and synthetic fibres. Fibres are not only used to improve the structural strength but also to other aspects of structural performances due to their numerous benefits, including chemical resistance, corrosion resistance, and abrasion resistance. Glass fibres have been used since the late 1960s and offered excellent engineering properties [5]. For example, these fibres exhibit high tensile strength (2-4GPa) and elastic modulus (40-80 GPa). However, durability is a disadvantage of glass fibres because the fibres suffer a significant strength reduction when they are exposed to harsh environments. Therefore, glass fibre reinforced concrete has not been popularly applied in reality. As the second type of fibres, carbon fibres have higher tensile strength and elastic modulus than glass fibres. However, their application has still been limited because they are more expensive than other types of fibres. Meanwhile, synthetic fibres, which are produced from the textile and petrochemical industries, contribute approximately half of all fibre utility. There are various classes of synthetic fibres but nylon, acrylic, polyester, and polyolefin dominate in the market. Also, synthetic fibres have significantly different levels of tensile strength and elastic modulus, approximately 230-1100 MPa for tensile strength and 5-19 GPa for elastic modulus [38]. Besides, steel fibres have been applied in concrete since the early of the 1990s and there have been various changes in shapes until now [5]. The early steel fibres had a round, straight, and smooth form with chopped lengths. This steel fibre type has been rarely used in recent years and is almost replaced by modern steel fibres with a deformed surface of hooked ends to ensure a better anchorage in concrete. In general, steel fibres exhibit ductile stress-strain characteristics with high tensile strength (0.5-2 GPa) and elastic modulus (up to 200 GPa) [5]. Considering the comparisons of these fibre types, steel fibres were chosen for the current study.

In the literature, there have been only two experimental studies investigating the joint behaviours using the concrete-end-plate and steel bolts [14, 17]. However, steel bolts in the connections are susceptible to corrosion, thus, Ngo et al. [27] proposed CFRP bolts to replace these steel bolts. Ngo et al. [27] found that the inclined cracks at the middle zone of the concrete-end-plate are the main failure mode of this precast joint type. Therefore, the current study investigates the use of SFRC to minimize the inclined cracks on the concrete-end-plate and thus improve the load-carrying capacity, ductility, energy dissipation, stiffness and drift ratio of these joints. In addition, the current study also investigates the effects of the prestress levels on the bolts. Finally, since there is no analytical model for this precast joint type, this study also proposes an empirical model for this precast joint type so that it could be effectively applied in practice.

2. Experimental program

Five specimens were cast and examined under quasi-static cyclic loads until failure to evaluate effects of various parameters on their performance. The five specimens, namely M1, P2-C-SP, P3-C-NSP-F, P4-S-SP-H, and P5-S-SP, were labelled based on their characteristics. The letters "M" and "P" indicate the reference monolithic specimen and the precast specimens, respectively. The letters "C" and "S" denote the use of CFRP bolts or steel bolts. The letters "NSP", "SP", and "F" represent the use of no spirals, spirals, and steel fibres in the concrete-end-plate, respectively. Among the specimens, only Specimen P4-S-SP-H was prestressed at a high level of 51 kN so the letter "H" is used to distinguish this difference while the prestressing force of other specimens was approximately 6.5-10.5 kN. The 20-mm diameter bolts were used for all the precast specimens.

2.1 Design of the specimens

To investigate the effects of steel fibres and prestress levels on the structural response of the beam-column joints using concrete-end-plate and bolts, four precast exterior beam-column joints were designed based on previous studies [14, 17, 18, 27]. It should be noted that there were no specific standards for the precast joints using bolts and concrete-end-plates. The monolithic specimen which had similar dimensions of the precast specimens was designed in accordance with ACI 352R-02 [29] and ACI 550R-96 [39], and was served as a reference specimen. The design philosophy of strong column and weak beams was applied to all the specimens. The columns in all the specimens had the same length of 1280 mm with a square-shaped cross-section of 200×200 mm^2. Four deformed steel bars with a diameter of 16 mm, placed at the four corners, were used as longitudinal reinforcements. Stirrups with a diameter of 10 mm were arranged with a spacing of 70 mm to resist shear forces in the columns (Fig. 1).

There are two components on the beams of the precast specimens, including (1) the concrete-end-plate and (2) a rectangular beam, namely Beam A. All the beams also had a square cross-section of 150×150 mm^2 and were reinforced with 16-mm deformed bars. Additionally, the shear reinforcements of these beams, which had the same diameter of 10 mm, were also placed with a spacing of 70 mm. The size of Beam A was 150×150×520 mm^3 while that of the concrete-end-plate was 350×150×200 mm^3 as exhibited in Fig. 1.

To monitor strain of concrete inside the concrete-end-plate, 6.1-mm aluminium bars were placed in the assumed direction of the concrete strut and strain gauges were attached to these bars as shown in Fig. 2. In addition, to ensure the sufficient bond between the concrete and the aluminium bars, notches were created by a cutting machine. All the beams and columns had a clear concrete cover of 35 mm as per ACI 318-11 [28]. The prestress level and the tensile force in the bolts during the test are important parameters which affect the load-carrying capacity and joint opening of this kind of joint [27]. Therefore, this current study used two load cells with a capacity of 20 ton to monitor the tensile forces in the bolts during the tests as shown in Fig. 3.

2.2 Mechanical properties of materials

Ready-mixed concrete from local suppliers (i.e., Western Australia) was used to cast the specimens. For the beam of Specimen P3-C-NSP-F, the concrete was mixed with 1% fibre volume by a concrete-mixer in the Civil Engineering Laboratory, Curtin University. The average compressive strength (f'_c) and tensile strength (f_{ct}) on the testing day were 38.4 MPa and 3.8 MPa for conventional concrete, and 32.3 MPa and 4.3 MPa for SFRC, respectively. It is noted that Specimen P3-C-NSP-F was cast by a different concrete batch from the other four specimens. The hook-ended steel fibres were supplied by TEXO company [40]. The length and the diameter of steel fibres were 35 mm and 0.55 mm, respectively. The tensile strength, elastic modulus, and aspect ratio (*l/d*) were 1350 MPa, 210 GPa, and 65, respectively. Conventional steel bars of 8 mm, 10 mm, and 16 mm were used for spirals, stirrups, and longitudinal reinforcements, respectively. Bolts, nuts, and plates were produced from CFRP material and were provided by J and R Metalwork Industry CO. [41]. Other properties of these steel bars, aluminium bars, CFRP bolts, and CFRP nuts, which were provided by the manufacturers, are presented in Tables 1 and 2. Strain gauges with the length of 60 mm and 5 mm were attached on concrete surfaces, longitudinal reinforcements, aluminium bars, and stirrups to measure their strain. Locations of these strain gauges are specified in Figs. 2 and 3.

2.3 Test setup

The columns of the precast specimens were connected to a steel frame, followed by the setup of the beam. CFRP plates with a size of 150×90×20 mm^3 were positioned into the CFRP bolts before the nuts were tightened with a torque wrench. The torsion level applied to the nuts was 80 Nm to avoid concentrated stresses around the bolts [27]. The main hydraulic jack was placed at the beam tip with a distance of 550 mm from the column surface to apply a vertical quasi-static cyclic load. Another hydraulic jack was placed on the column top to apply the vertical restraint with an axial force of 15 kN. This force was maintained as low as possible to minimize beneficial effects of axial force to the capacity of the joints [42]. The schematic setup and details are presented in Fig. 4. All the exterior joints were subjected to quasi-static cyclic loads until failure and the loading history is shown in Fig. 5. All the testing process was conducted under displacement control at the rate of 6-9 mm/min.

2.4 Experimental results and discussion

2.4.1 Hysteretic response

The applied load versus drift ratio responses of all the specimens are presented in Figs. 6 and 7. Overall, the applied load-displacement hysteretic responses were almost symmetrical in both push and pull directions because there were the same top and bottom reinforcements in all the tested specimens. Nevertheless, after the peak loads, the hysteretic curves were asymmetrical because of large and irreversible deformation, which was also reported in the previous studies [14, 18]. The hysteresis loops of Specimens P2-C-SP, P4-S-SP-H, and P5-S-SP were quite similar to each other, with almost linear responses until 2% drift ratio and then the applied load slowly increased until achieving the peak load at 3% drift ratio. After 3% drift ratio, the applied load dropped although the displacement at the beam tip still increased. During this stage, the CFRP bolts and steel bolts did not reach the yielding points until the specimen failure while the longitudinal reinforcements still behaved linearly up to 3% drift ratio as seen in Fig. 8, which shows the measured strain of the reinforcements on the beams and the concrete-end-plates. The dots in this figure show their maximum strain at each cycle. Consequently, the concrete governed the main failure with various inclined cracks on the concrete-end-plate. These inclined cracks resulted in degradation in the applied load after reaching the peak load. Meanwhile, Specimen P3-C-NSP-F underwent more ductile responses than other precast specimens when reaching the peak load at the drift ratio of 3.5%. After 1.5% drift ratio, the hysteresis loop exhibited nonlinear behaviours up to the peak load. Afterwards, the applied load approximately obtained a plateau within a range of the drift ratio from 3.5% to 5.0%. This is a favourable response of an important structural member such as beam-column joints. This great improvement was due to the contribution of steel fibres inside the concrete-end-plate. Since SFRC had higher tensile strength (4.3 MPa) than conventional concrete (3.8 MPa), the inclined cracks on the concrete-end-plate, which caused the main failure of this kind of joint [27], were effectively minimized (see Fig. 9 (P3-C-NSP-F)).

The hysteretic curves of Specimens P4-S-SP-H and P5-S-SP depict the influence of the prestress level of the bolts. These two specimens had the same design except the prestress force in Specimen P4-S-SP-H (51 kN) was higher than that of Specimen P5-S-SP (10.5 kN). The high prestress levels did not change the shape of hysteretic curves until failure but it affected the peak loads. The maximum loads of Specimens P4-S-SP-H and P5-S-SP were 50.3 kN and 41.8 kN, respectively, which exhibits an increase of 20%. The high prestress levels in bolts improved the load-carrying capacity of this precast joint type. Interestingly, the envelope curves of all the precast specimens depict that the applied loads remained almost unchanged from 4% to 5% of the drift ratio as shown in Fig. 7. This phenomenon could be attributed to the yielding and strength hardening of the longitudinal reinforcements. The reinforcements started to yield at approximately 4% drift ratio and then stresses in the reinforcements still raised. This performance was a result of the fact that all the beams were designed as over reinforced. Therefore, the longitudinal reinforcements caused the plateau responses of the precast specimens from 4% and 5% drift ratio. This behaviour prevented the occurrence of brittle failure and offered necessary warnings before complete collapse of structures. This observed behaviour also shows that the use of this precast joint type may offer a safer solution as compared to other joints.

For Specimen M1, the elastic response was observed up to drift ratio of 1%. From 1% drift ratio upwards, the number of cracks gradually increased, which signified the energy dissipation capacity of the specimen and also caused the nonlinear responses of Specimen M1. At 2.7% drift ratio, Specimen M1 began to yield before achieving the peak load at 5% drift ratio. From 5% to 6.5% drift ratio, the applied load continuously decreased and the test stopped at 6.5% drift ratio.

2.4.2 General behaviours and failure patterns

The maximum tensile forces in the CFRP bolts and steel bolts were significantly lower than their nominal tensile strengths. For instance, the maximum tensile force of the CFRP bolts in Specimen P2-C-SP was 34.1 kN, which was approximately 34% of its ultimate tensile strength as given in Table 2. Hence, no failure was observed in the CFRP bolts during the tests. Cracking patterns of all the tested specimens are shown in Fig. 9. No cracks could be found on the columns of all the precast joints while only minor cracks were observed on the column of the monolithic joint. The columns showed excellent behaviours with no failure, which makes all the specimens satisfy the requirements of the design principle of weak beams-strong columns. In addition, although this precast joint type did not have corbels or brackets to resist shear forces, the LVDT data demonstrated that there were no slips between the concrete-end-plates and column interfaces. It means that the friction between the column and the concrete-end-plate is sufficient to resist shear forces.

To further investigate the failures of all the specimens, it is necessary to estimate the load-carrying capacities of the joints, which are governed by the capacities of either the concrete-end-plate or the beam (Beam A). Assuming failure occurs at Beam A, the design load of the monolithic joint is 28 kN while all the precast beams have a design load of 44 kN, except for Specimen P3-C-NSP-F with 39 kN. The design load of Specimen P3-C-NSP-F was lower than that of other precast specimens due to its lower compressive strength (f'_c= 32.3 MPa for P3-C-NSP-F and f'_c= 38.4 MPa for other precast specimens). If the maximum load of a specimen was lower than the estimated design load of the beam, it could be concluded that this specimen failed in the joint. Otherwise, this specimen failed on the beam. Specimens P2-C-SP and P5-S-SP reached their maximum loads of 37.7kN and 41.8 kN at 3% drift ratio, respectively. These peak loads were slightly lower than the estimated design loads of the corresponding beams. Therefore, it can be concluded that the main failure of Specimen P2-C-SP and P5-S-SP occurred in the joints, specifically at the concrete-end-plates, whereas that of Specimens M1 (32.3 kN), P3-C-NSP-F (47.0 kN), and P4-S-SP-H (50.3kN) occurred on the beams at the fixed-ends as seen in Fig. 9. The maximum loads of these beams were higher than the estimated capacities, which could be attributed to the variation between the actual stress and the nominal tensile strength of the longitudinal reinforcements and also the conservative design.

During seismic loadings, beam-column joints could fail as a result of shear forces or diagonal compressive forces [13, 43, 44]. The previous study by Ngo et al. [27] indicated that the main failure of the proposed precast joint was due to inclined cracks in the middle region of the concrete-end-plate. The data of strain gauges in Fig. 8 (a) show that the compressive strain (< 110 $\mu\varepsilon$) in the compressive struts of the concrete-end-plate did not reach the ultimate compressive strain of concrete, while the ultimate tensile strain (131 $\mu\varepsilon$) significantly exceeded approximately > 10 times. For example, the tensile strain of concrete of Specimen P2-C-SP was 1333 $\mu\varepsilon$ at 3% drift ratio (see Fig. 8). The ultimate tensile strain of concrete was calculated according to its tensile strength (f_{ct}) and the Young's modulus. The above observations could be attributed to the effects of strut angles. Fig. 10 shows two assumed struts, namely QU and QK, which could appear on the concrete-end-plate. The angle of the compressive strut (QU) was only 34^0 which led to low compressive stress inside Strut QU. Normally, if the strut angle is greater than 45^0, compressive failure may occur [14, 45]. For Strut QK, although this strut had a larger angle (50^0) than that of Struts QU, it did not fail by compressive stress. The data of strain gauges in Fig. 11 and observations during the test exhibit that the concrete-end-plate failed by tensile cracks in the middle zone.

Steel fibres were applied in this study to resist tensile cracks and, therefore, significantly increased the joint capacity. Fig. 9 demonstrates that Specimen P3-C-NSP-F had the least number of cracks among the four precast specimens. Only four minor inclined cracks were found in the concrete-end-plate at 1.8% drift ratio. The crack development could be summarized in four stages: the first-crack, main-crack, ultimate-crack and specimen failure. The initial stage was considered from the beginning of the test to the moment when the first-crack appeared. The first-crack occurred when the tensile stress in the concrete-end-plate and Beam A overcame the tensile resistance of concrete. For the joint, its first-crack mainly depended on the concrete strength, rather than the stirrups inside the concrete-end-plate [32]. Therefore, the applied load of Specimen P3-C-NSP-F at the first-crack was greater than that of other precast specimens due to higher tensile strength (4.3 MPa) compared to that of conventional concrete (3.8 MPa). For example, the applied load of Specimen P3-C-NSP-F at the first crack increased by approximately 21% if compared to Specimen P2-C-SP. After this initial stage, the inclined crack started to develop in the concrete-end-plate which led to a strain increase in the stirrups when the applied load increased. The stage after the appearance of the first crack to the moment when the stirrups in the concrete-end-plate started to yield was so-called the main crack stage. For Specimen P3-C-NSP-F, the stirrup strain (2233 $\mu\varepsilon$) at 3.5% drift ratio corresponding to the peak load did not reach the yield strain (2800 $\mu\varepsilon$) . This phenomenon could be explained that bridging actions of steel fibres helped to prevent the formation of the main tensile cracks on the concrete-end-plate [5, 35-37]. Therefore, the main failure position was on the beam instead of the joint, with a plastic hinge forming at the fixed-end as shown by the pink curve in Fig. 9 (P3-C-NSP-F). After that, during the ultimate-crack stage, the applied loads increased with a limited rate until reaching the peak load. In the final stage, pulled-out fibres and spalling of concrete occurred at the fixed-end which caused the failure of Specimen P3-C-NSP-F. This final stage occurred after the specimen reached the peak load until the end of the test.

As shown in Fig. 9, Specimens P4-S-SP-H and P5-S-SP exhibited different failure modes and trends of the crack development while Specimens P2-C-SP and P5-S-SP behaved similarly irrespective of the bolt materials. For Specimens P2-C-SP and P5-S-SP, which had similar design except for the bolts (CFRP bolts vs steel bolts), the flexural crack initially developed at the fixed-end at ±0.5% drift ratio when the tensile strain of concrete at this section reached its maximum tensile strain. After this stage, the longitudinal reinforcements of Beam A started to make the main contribution to the flexural resistance. Therefore, the inclined cracks firstly propagated into the middle zone and then the top and bottom zones of the concrete-end-plate which caused the main failure for Specimens P2-C-SP and P5-S-SP. It means that the bolt material did not affect the failure mode, indicating CFRP was successfully used to replace steel bolts. Meanwhile, Specimens P4-S-SP-H and P5-S-SP had the same design except for the only difference in the prestress levels (i.e., 51 kN for Specimen P4-S-SP-H and 10.5 kN for Specimen P5-S-SP). Specimen P5-S-SP exhibited much more number of inclined cracks in the concrete-end-plate as compared to Specimen P4-S-SP-H (see Fig. 9). In other words, the inclined cracks on Specimen P5-S-SP thoroughly distributed across the entire concrete-end-plate surface while Specimen P4-S-SP-H showed cracks concentrated in the middle zone, except for only one developed in the top and bottom of the concrete-end-place after reaching the peak load of 50.3 kN at 3.0% drift ratio (see the orange curves in Fig. 9 (P4-S-SP-H)). This difference could be attributed to the high compressive stress which was established in the top and bottom zones of the concrete-end-plate due to high prestress level. Therefore, the tensile cracks in the top and bottom zones could only be developed when the tensile stress counteracted this compressive stress. From the above analysis, it could be concluded that the high prestress level in the bolts reduced the number of cracks in the top and bottom zones of the concrete-end-plate. In addition, the prestress level in the bolts also affected the failure mode of precast specimens because high prestress level effectively minimized the appearance of tensile cracks in the top and bottom zones of the concrete-end-plate. For instance, Specimen P4-S-SP-H failed at the fixed-end of the beam while the failure of Specimen P5-S-SP occurred in the concrete-end-plate. Therefore, the load-carrying capacity of Specimen P4-S-SP-H was significantly improved (i.e., approximately 20% in push direction compared to Specimen P5-S-SP).

The primary failure of the reference Specimen M1 occurred at the fixed-end of the beam. Minor spalling of concrete initially occurred at the fixed-end when the drift ratio reached 2%. Specimen M1 achieved its maximum capacity at 5.0% drift ratio. Afterwards, the failure was caused by the vertical cracks and spalling of concrete at the fixed-end of the beam.

2.4.3 Drift ratio and load-carrying capacities

The drift ratio is an important parameter to evaluate the joint performances under earthquake loadings. The drift ratio is defined as the ratio of vertical displacement (Δ) at the loading point of the beam to the distance (l = 550 mm) between the column face and the loading point, as follows:

$$R = \Delta / l \tag{1}$$

In most of the previous studies, the drift ratio of precast beam-column joints is usually lower than that of corresponding monolithic joints [46, 47]. The drift ratio of precast specimens usually varies from 1.5% to 3% [14, 48] whereas the requirements for structures to be applied in earthquake-prone regions are approximately 3.5% according to ACI T1.1-01 [49], 2.5% in CSA A23.3-07 [50], and 2% in ASCE 41-06 [51] to ensure the life safety. The precast joint using the concrete-end-plate and steel bolts in the previous study by Saqan [14] only reached the drift ratio of 1.5%. Meanwhile, Ngo et al. [27] improved this kind of joint and achieved a higher drift ratio of 3%. This drift ratio satisfied the requirement of CSA A23.3-07 [50] and ASCE 41-06 [51] but still lower than the requirement of ACI T1.1-01 [49]. This issue has been resolved in the current study with the addition of steel fibres. The results show that the 3.5% drift ratio of the currently tested specimens satisfied the requirement of ACI T1.1-01 [49] so that the proposed joint can be effectively used in earthquake-prone regions. Table 3 and Fig. 12 show the maximum loads with the corresponding drift ratios of all the tested specimens.

In general, the proposed dry joints showed excellent performance in terms of drift ratio and load-carrying capacity as compared to those from the previous studies [14, 17, 18]. Specimen P3-C-NSP-F with steel fibres reached 3.5% drift ratio and satisfied the requirement for use in earthquake-prone regions of the contemporary standards, namely ACI T1.1-01 [49], CSA A23.3-07 [50], and ASCE 41-06 [51]. The average peak load of Specimen P3-C-NSP-F (43.3 kN) was higher than those of Specimens P2-C-SP (36.8kN) and P5-S-SP (39.8kN). In addition, the load-carrying capacity of this specimen was also 49.1% higher than that of the monolithic specimen M1. Although Specimen M1 exhibited a ductile load-displacement response with the highest drift ratio of 5%, the load-carrying capacity of this specimen was the lowest among all the tested specimens. For failure at the fixed-end, the above results might be explained that the lever arm from the fixed-end to the loading point of Specimen M1 (550 mm) was longer than that of the precast specimens (350 mm) due to the absence of the concrete-end-plate, while all the specimens had the same square cross-section of 150×150 mm2 at the fixed-end. Therefore, the moment at the fixed-end of Specimen M1 was approximately 57.1% higher than that of the precast specimens with the same applied load. It implies that the use of the concrete-end-plate enhances the load-carrying capacity of an exterior precast joint if the failure occurs at the beam. For the failure in the middle zone of the concrete-end-plate, the load-carrying capacity of all the precast specimens depended on the thickness and the height of the concrete-end-plate. This study purposefully modified the thickness in the previous study by Saqan [14]. Therefore, the load-carrying capacity of the precast specimens was significantly improved if compared to that of the reference specimen M1.

In the meantime, the previous study by Ngo et al. [27] suggested that the prestress levels affected the load-carrying capacity while the use of CFRP bolts to replace steel bolts did not significantly change the load-carrying capacity of this precast joint type. These two main parameters are investigated in this study. The comparisons between the two specimens with similar design except for the prestressing force in the bolts (i.e., Specimens P4-S-SP-H and P5-S-SP) were conducted to explore the influence of the prestress levels. The prestress level of Specimen P4-S-SP-H was 51 kN whereas that of Specimen P5-S-SP was 10.5 kN. The experimental results revealed that the load-carrying capacity of Specimen P4-S-SP-H was higher (20%) than that of Specimen P5-S-SP. Therefore, it is concluded that the high prestress level improved the load-carrying capacity. For replacing steel bolts by CFRP bolts, Specimens P2-C-SP and P5-S-SP which had the same design, but different bolt material showed a similar load-carrying capacity and drift ratio of 3% (36.8 kN for Specimen P2-C-SP and 39.8 kN for Specimen P5-S-SP). Therefore, it can be concluded that steel bolts could be effectively replaced by CFRP bolts to resolve the corrosion issue without negative effects on the load capacity and the drift ratio of the joint.

2.4.4 Ductility of Joints

Structures are considered as ductile if they can dissipate significant energy during inelastic cyclic deformations [52]. Ductility indicates the capacity that a structure can withstand without any significant degradation in its strength during deformation. It is considered as a crucial parameter in seismic performances of a structure in order to avoid brittle failure. In this study, the ratio of the ultimate displacement (Δ_u) to displacement at the yield loads (Δ_y) is defined as the ductility of a structure, as expressed in Eq. 2 as follows:

$$\mu = \Delta_u/\Delta_y \tag{2}$$

In a reinforced concrete structure, the applied load versus displacement relationship does not reveal a clear yield point due to either the nonlinear performance of materials or the onset of yield at different load levels in various parts of structures. Consequently, the definition of yielding deformation is quite subjective and thus the yield and ultimate displacement of all the tested specimens are determined as shown in Fig. 13.

Due to the limitations of the hydraulic jack, the ultimate displacements of Specimens P2-C-SP and P4-S-SP-H were stopped at 28.1 mm, which corresponded to 90% of the peak load. For other specimens, a new hydraulic jack was utilised so that the ultimate displacement could be determined at 85% peak load. The ductility of all the tested specimens is presented in Table 4. The results reveal that Specimen P5-S-SP exhibited the highest ductility (μ = 2.8) among all the tested specimens. Specimens P3-C-NSP-F and P5-S-SP revealed higher ductility than Specimen M1 (μ = 2.4), with approximately 8.3% and 16.6% increase, respectively. Specimens P2-C-SP and P4-S-SP-H exhibited the ductility of μ = 2.1 and μ = 2.3, which were close to the reference specimen M1. It is noted that the applied loads of these two specimens were stopped at 90% of the peak loads. Therefore, it is expected that if the applied load continued to 85% of the peak loads, the ductilities of these specimens would be similar or even higher than that of Specimen M1. The precast specimens exhibited excellent ductility due to the beneficial influences of steel fibres and steel spirals inside the concrete-end-plate. As an evidence, the ultimate displacement of Specimen P3-C-NSP-F (36.2 mm) was 31.6% higher than that of Specimen P2-C-SP (27.5 mm) in the push direction. From the above analysis, it is clear that the addition of steel fibres into the concrete mixture could significantly enhance the ductility of the precast joint using concrete-end-plate and bolts.

2.4.5 Energy dissipation capacities

The energy dissipation capacity of the exterior joints is determined as the area enclosed (A_h) inside the applied load-displacement hysteretic loop in that corresponding load cycle. A beam-column joint under quasi-static cyclic loads is classified as a ductile joint if it can dissipate sufficient energy while there is no significant reduction of its strength and stiffness [47, 53]. The energy dissipation capacity versus drift ratio curves of all the specimens are shown in Fig. 14. As can be seen up to 1% drift ratio, similar trends and values were observed in the energy dissipation capacity of all the specimens since they behaved elastically. Nevertheless, the overall energy dissipation capacity of Specimen P3-C-NSP-F was lower than the other precast specimens from 1%-3.5% drift ratio. When this specimen reached the peak load at 3.5% drift ratio, the energy dissipation capacity dramatically increased until failure. This interesting phenomenon could be explained that steel fibres minimized the appearance of inclined cracks in the concrete-end-plate from the initial stage to 3.5% drift ratio. After this stage, the inelastic deformation and the crack width significantly increased because steel fibres were progressively pulled out from the matrix. Consequently, the toughness and dissipated energy of this specimen significantly increased. Moreover, the energy dissipated by Specimen P4-S-SP-H was approximately similar to that of Specimen P5-S-SP, indicating that the prestress level did not considerably affect the energy dissipation capacity of the precast joints (409 kN.mm for Specimen P4-S-SP-H and 387 kN.mm for Specimen P5-S-SP at 4% drift ratio).

The energy dissipation capacity at 4% drift ratio of all the precast specimens (P2-C-SP, P3-C-NSP-F, P4-S-SP-H and P5-S-SP) was 45.1%, 65.9%, 74.5%, and 65.3% greater than that of Specimen M1, respectively, which could be mainly attributed to fatter hysteretic loops as shown in Fig. 6. Therefore, it could be concluded that the proposed precast specimens exhibited excellent energy dissipation capacity for earthquake loading resistance.

2.4.6 Stiffness degradation

The stiffness degradation of all the specimens under cyclic loads was evaluated in term of the effective stiffness. The effective stiffness at each cycle was determined based on the slope of the line connecting the peak-to-peak loads in the positive and negative directions during the first cycle of the two reversal cycles at each drift ratio (see Fig. 15) [27]. In general, the effective stiffness of all the specimens was continuously decreased when the applied load increased. It is expected that the stiffness reduction only occurs when a structure has partial damage or enters the plastic stage. The envelope curves in Fig. 7 show almost linear responses of the specimens from the initial stage to the drift ratio of 1.5-2%. However, the stiffness reduction of these specimens occurred at quite an early stage of about 0.5% drift ratio, which can be explained by two reasons. Firstly, since the tensile cracks at fixed-end appeared quite early at 0.5% drift ratio, it means that there was minor damage at this stage. Secondly, the strain of concrete reached 30%$\varepsilon_c^{'}$ so concrete began to behave nonlinearly. Fig. 16 presents the stiffness degradation for all the specimens versus their drift ratios.

Fig. 16 shows that the effective stiffness of all the precast specimens was greater than that of Specimen M1 until failure because the use of the concrete-end-plate improved the stiffness of the precast specimens. It is noted that the design of the concrete-end-plate was improved in this study so that premature failure at the concrete-end-plate did not occur as observed in the previous studies [14, 27]. The stiffness of Specimen P3-C-NSP-F was approximately 26.7% higher than that of Specimen P2-C-SP at 4% drift ratio. This observation could be explained that the use of steel fibre concrete has increased the joint stiffness and minimized damage to the concrete. Moreover, the effective stiffnesses of Specimens P4-S-SP-H and P5-S-SP were almost the same, indicating that the prestress level did not significantly affect the initial stiffness (5.1 kN/mm for P4-S-SP-H and 5.2 kN/mm for P5-S-SP). This observation agrees well with findings from other studies [54-56]. Le et al. [55] also found that the prestress level showed unnoticeable influences on the initial stiffness of structures. In Fig. 16, the stiffness degradation curves of these two specimens from 1% to 4% drift ratio were also parallel, so the overall stiffness of this precast joint type using bolts was not significantly affected by the prestress levels. However, it is worth mentioning that this parallel curves did not occur between 0.5% and 1% drift ratio because the trend of crack development on both specimens was totally different in the initial stage as discussed in section 2.4.2. Therefore, the stiffness degradation was not similar from 0.5% to 1% drift ratio. In addition, the stiffness of specimens using steel bolts (i.e., P4-S-SP-H and P5-S-SP) in the initial stages was greater than that of specimens using CFRP bolts (i.e., P2-C-SP and P3-C-NSP-F), (i.e., approximately 23.8% at 0.3% drift ratio) because steel bolts had higher elastic modulus (200 GPa) than CFRP bolts (100 GPa). Therefore, the specimens with steel bolts showed higher stiffness than the specimens with CFRP bolts.

3. Analytical calculations

In this section, an analytical model to estimate the load-carrying capacity of the precast beam-column joints, using concrete-end-plates and bolts, is proposed. As previously discussed in Section 2.4.2, there are two possible failure sections in this precast joint type, including (1) at the fixed-end and (2) in the middle zone of the concrete-end-plate. Maximum applied load (P_{max}) is the minimum applied load at the loading point of the beam when the failure occurs either at the fixed-end (P_{max1}) or the middle zone of the concrete-end-plate (P_{max2}).

3.1 Failure at fixed-end

When failure occurs at the fixed-end of the beam, P_{max1} is estimated based on the flexural capacity of the beam. This applied load is calculated with a reference to the nominal moment strength (M_{n1}) of Beam A at the fixed-end. P_{max1} can be calculated as follows:

$$P_{max1} = \frac{M_{n1}}{L_1} \quad (3)$$

where L_1 is the distance from the loading point to the fixed-end (L_1 = 350 mm in the present study).

In a general circumstance, M_{n1} of Beam A is determined with the assumption that the longitudinal reinforcements have reached a yielding point when Beam A fails. However, all the specimens in the current study were designed as over reinforced. Therefore, M_{n1} is calculated based on the actual strain of the longitudinal reinforcements.

3.2 Failure in the middle zone

When the failure occurs in the middle zone of the concrete-end-plate, P_{max2} is determined based on the assumption that the stirrups have yielded. Fig. 17 presents the free body diagram and global equilibrium of the specimens, in which T_1 vs T_2 and T_3 are the tensile forces of the bolts and reinforcements, T_4 is the compressive forces in the reinforcements, f is the friction between two surfaces of the column and concrete-end-plate, and q_1 and q_2 are the compressive stress in concrete on the left and right of the concrete-end-plate, respectively. It is worth mentioning that the tension forces (T_1 and T_2) of the bolts are caused by two sources, including the applied load (P) and the prestressing force. As a result, T_1^a represents the tensile force caused by the applied load while T_1^b is caused by the prestressing force. From the force-equilibrium as shown in Fig. 17 (b), the tensile force in the top bolts (T_1) could be determined as follows:

$$T_1 = T_1^a + T_1^b = V_{hs2} + \gamma P_r \tag{4}$$

where P_r is the prestress levels of bolts, γ refers to the prestress level loss in bolts, γ is determined from the initial prestress force (P_r) and tensile force in the bolt at 0% drift ratio immediately after the applied load has reached the peak load. All these tensile forces were taken from data of the load cells in the current study, in which $\gamma = 0.84$ is for steel bolts and $\gamma = 0.25$ for CFRP bolts, V_{hs2} is the horizontal shear force which is calculated based on the inclined shear force (V_{s2}) at Section S2-S2 (see Fig. 17 (b)) as follows:

$$V_{s2} = V_s sin\alpha + V_c + V_f \tag{5}$$

$$V_{hs2} = V_{s2} cos\alpha \quad (6)$$

where $V_s = nA_v f_{yt}$ is the shear force contributed by the stirrups, n, A_v, and f_{yt} are the number of stirrup legs, cross-section area, and yield strength of the stirrups, respectively. The strain gauges attached on the stirrups inside the concrete-end-plate show that strain of two side stirrups was significantly lower than that of the middle stirrup. For instance, the strain of the middle stirrup of Specimen P2-C-SP was 3286 με whereas that of side stirrups was only 155 με at 3% drift ratio. Therefore, only the contribution of the middle stirrup was considered to compute V_s while the contribution of the other two stirrups was ignored. For the shear force carried by the concrete, $V_c = \beta h_p b_p \sqrt{f_c'}$, $\beta = \frac{1}{6}$ is an empirically derived function whose value is adopted in this study based on the model of the beam in ACI 318-11 [28], h_p and b_p are the thickness and width of the concrete-end-plate. For shear force contributed by steel fibers, $V_f = 2\frac{l_f}{d_f} v_f h_p b_p$, l_f, d_f, and v_f are the length, diameter, and volume fraction of steel fibres [32].

P_{max2} is determined from the equilibrium condition as shown in Fig. 17(c), as follows:

$$P_{max2} = \frac{T_1(H-2a_p)}{(L+0.5h_p)} \quad (7)$$

where L, a_p, h_p, and H are the length of Beam A, the distance from the extreme-top fibre of the concrete-end-plate to the centroid of the top bolts, the thickness and height of the concrete-end-plate, respectively (see Fig. 17 (c)).

The maximum applied load (P_{max}) of this precast joint type is determined in the following equation:

$$P_{max} = min(P_{max1}, P_{max2}) \quad (8)$$

To compare the results of the proposed model and those of the experiment, the maximum applied load from the experiment ($P_{exp-max}$) for each specimen is taken from the data of the main load cell which was connected to the hydraulic jack at the beam tip (see Fig. 4). The data of the main load cell and the load cell on bolts are shown in Figs. 7 and 18. It is noted that no data of the load cell on the top bolt of Specimen P3-C-NSP-F was recorded due to malfunction of the data acquisition system during the test. ($P_{exp-max}$) of all the precast specimens are summarized in Table. 5. The Pmax comparisons between the proposed model and the experimental results are indicated in Fig. 19.

As can be seen from Table 5 and Fig. 19, where the actual data from the test was used to verify the accuracy of the proposed model, the proposed model could well predict the load-carrying capacity of all the precast specimens. For the failure at the fixed-end, the errors between the experimental results and the proposed model results of Specimens P3-C-NSP-F and P4-S-SP-H were minor, only 0.9% and 1.6%, respectively. For the failure in the middle zone, Specimens P2-C-SP and P5-S-SP had the same failure modes in the middle zones, indicating this proposed model accurately predicted the maximum applied load (P_{max}) with a low error of 2%. The above analyses prove that the proposed model could be well applied to predict the capacity of this precast joint type.

4. Conclusions

This study carried out an experimental investigation on the effects of steel fibres and prestress levels on the structural performance of the proposed dry joints. In addition, a new model to estimate the load-carrying capacity of this precast joint type was proposed. The excellent performances of all the precast specimens show that this precast joint type can be potentially utilised in both non-earthquake and earthquake-prone regions. Findings from this study can be summarized as follows:

1. The proposed dry joints outperformed the corresponding monolithic joint in terms of load-carrying capacities, energy dissipation, and stiffness. In addition, the proposed model could well predict the maximum applied load of all the precast specimens with minor errors ranging from 0.9% to 2%.
2. Using SFRC could significantly improve the load-carrying capacity (18%) and ductility (53%) of the proposed dry joint.

3. The average peak loads of all the precast specimens were from 27% to 61% higher than that of the monolithic specimen.
4. All the precast joints have higher effective stiffness than that of monolithic joint M1 until failure.
5. The high prestress level improved the load-carrying capacity of the precast beam-column joints but did not increase the initial stiffness.

In general, CFRP bolts could be effectively applied in dry beam-column joints using concrete-end-plates and bolts. They could effectively resolve the corrosion issue in steel bolts while still assure excellent performance under quasi-static cyclic loads.

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List of Figures

List of Tables

Nomenclature

Notations

a_p	distance from the extreme-top fibre of the concrete-end-plate to the centroid of the top bolts	P_{max2}	applied load with the failure in the middle zone
A_v	cross-section area of of stirrup legs	P_r	prestress levels of bolts
b_p	width of the concrete-end-plate	q_1	compressive stress in concrete on the left of the concrete-end-plate
d_f	diameter of steel fibres	q_2	compressive stress in concrete on the right of the concrete-end-plate
f	friction	T_1	tensile force of the top bolts
f'_c	compressive strength of concrete	T_1^a	tensile force of the bolts is caused by the applied load
f_{yt}	yield strength of the stirrups	T_1^b	tensile force of the bolts is caused by the prestressing force
H	height of the concrete-end-plate	T_2	tensile force of the bottom bolts
h_p	thickness of the concrete-end-plate	T_3	tensile force of the reinforcements
l	distance from the loading point to the column face	T_4	compressive force of the reinforcements
L	length of Beam A	V_c	shear force is contributed by the concrete
L_1	distance from the loading point to the fixed-end	V_f	shear force is contributed by the steel fibres
l_f	the length of steel fibres	v_f	volume fraction of steel fibres
M_{n1}	nominal moment strength	V_{hs2}	horizontal shear force
n	number of stirrup legs	V_s	shear force is contributed by the joint stirrups
R	drift ratio value	V_{s2}	inclined shear force at Section S2-S2
P	applied load	β	an empirically derived function
P_1	applied load at 75% peak load	γ	prestress level lost in bolts
P_2	applied load at 85% peak load	Δ	vertical displacement of the beam tip at the position of the applying force
$P_{exp\text{-}max}$	maximum applied load from the experiment	Δ_u	ultimate displacement
P_{max}	peak load	Δ_y	displacement at yield
P_{max1}	applied load with the failure at the fixed-end	μ	ductility value

Table **1**

Steel reinforcements and aluminium bars properties.

Bar diameters (mm)	f_y (MPa)	f_u (MPa)	E_s (GPa)	Area (mm^2)	Notes
8	377	522	200	50	Spirals
10	560	675	200	78	Stirrups
16	597	706	200	201	Longitudinal reinforcements
6.3	110	150	69	31	Aluminium bars

Table 2

Details of CFRP bolts, nuts, and plates [41].

Names	Size	Tensile strength	Shear strength	Bending strength	Compressive strength	Ultimate load	Impact strength	Elastic modulus	Weight
	mm	MPa	MPa	MPa	MPa	kN	kJ/m^2	GPa	g
Bolts	φ20	≥ 850	≥ 160	480	760	≥ 267	185	100	376
Nuts	φ20	*	*	*	*	100	*	100	44
Plates	150×90×20	*	*	*	*	≥ 100	*	100	540

Note: * not given

Table 3

Details of load-carrying capacities with corresponding drift ratios.

Specimens	Peak load (kN)		Increase (%)		Average (kN)	Increase (%)	Drift ratio at peak load (%)	
	Push	Pull	Push	Pull			Push	Pull
M1	25.8	32.3	-	-	29.1	-	5.0	5.0
P2-C-SP	35.8	37.7	38.6	16.9	36.8	26.6	3.0	3.0
P3-C-NSP-F	39.6	47.0	53.2	45.7	43.3	49.1	3.5	3.5
P4-S-SP-H	50.3	43.3	94.7	34.3	46.8	61.2	3.0	3.0
P5-S-SP	41.8	37.8	62.0	17.1	39.8	37.1	3.0	3.0

Note: - = not applicable

Table 4

Displacement and ductility of all the tested specimens.

Specimens	Force	P_{max}	P_1	Δ_y	P_2	Δ_u (85%)	$\mu=\Delta_u/\Delta_y$	Average (μ)	Decrease (%)
	Unit	kN	kN	mm	kN	mm			
M1	Push	25.8	19.4	15.6	22.0	35.8	2.3	2.4	-
	Pull	32.3	24.2	14.5	27.4	35.6	2.5		
P2-C-SP	Push	35.8	26.9	12.2	32.2[a]	27.5[a]	2.3	2.1	-12.5
	Pull	37.7	28.3	11.1	34.0[a]	21.5[a]	1.9		
P3-C-NSP-F	Push	39.6	29.7	13.8	33.7	36.2	2.6	2.6	8.3
	Pull	47.0	35.3	14.0	40.0	35.0	2.5		
P4-S-SP-H	Push	50.3	37.7	11.4	45.3[a]	26.5[a]	2.3	2.3	-4.2
	Pull	43.3	32.5	11.4	39.0[a]	26.3[a]	2.3		
P5-S-SP	Push	41.8	31.4	11.0	35.5	28.0	2.5	2.8	16.6
	Pull	37.8	28.4	10.4	32.1	31.2	3.0		

Note: - = not applicable

[a] at 90% of the post-peak load

Table 5

The comparisons of P_{max} between the experiment and analytical model.

Specimens	Experimental results	Theoretical results (fixed-end)			Theoretical results (middle zone)			
	$P_{exp\text{-}max}$	M_{n1}	P_{max1}	%	V_{hs2}	T_1	P_{max2}	%
	kN	kN.m	kN		kN	kN	kN	
P2-C-SP	37.7	15.7	44.7	-	64.9	66.5	37.0	-2.0
P3-C-NSP-F	47.0	16.3	46.6	-0.9	98.0	100.0	55.6	-
P4-S-SP-H	50.3	17.3	49.5	-1.6	64.9	107.7	59.9	-
P5-S-SP	41.8	14.9	42.7	-	64.9	73.8	41.0	-2.0

Note: - = not applicable