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Performance of Geopolymer Concrete in Monolithic and Non-Corrosive Dry Joints Using CFRP Bolts under Cyclic Loading

Tuan T. Ngo[1]; Tung T. Tran[2]; Thong M. Pham[3]*; and Hong Hao[4]*

Abstract

This study evaluates the performances of beam-column joints made of geopolymer concrete (GPC). A new dry joint type made of GPC and carbon fibre reinforced polymer (CFRP) bolts was proposed for moment-resisting concrete frames under earthquake loadings. Cyclic loading was applied to test the four specimens which were preparatorily cast by ordinary portland concrete (OPC) and GPC. Compared to monolithic joints, the proposed dry joints showed better performances in the maximum load-carrying and energy dissipation capacity. Additionally, new analytical models to design GPC monolithic and GPC precast joints are proposed. These models well predict the peak loads, main failure modes, failure positions, and horizontal shear strength with a minor variation of 1.3%. The application of GPC promises to effectively recycle a large amount of industrial wastes. Furthermore, the proper design made sure the CFRP bolts survive during the test without brittle failure and shear failure. Therefore, they could be potentially applied in the proposed dry joint to well resolve the corrosive issues in conventional precast joints, as well as satisfying the requirements for construction in sesmic regions.

Keywords: Geopolymer concrete (GPC); FRP bolts; Ductile dry joint; Cyclic loading.

[1] PhD Scholar, Center for Infrastructural Monitoring and Protection, School of Civil and Mechanical Engineering, Curtin University, WA 6102, Australia and The Faculty of Engineering and Technology, Quy Nhon University, Binh Dinh, Viet Nam. Email: tangtuan.ngo@postgrad.curtin.edu.au
[2] PhD Scholar, Center for Infrastructural Monitoring and Protection, School of Civil and Mechanical Engineering, Curtin University, WA 6102, Australia. Email: thanhtung.tran@postgrad.curtin.edu.au
[3] Lecturer, Center for Infrastructural Monitoring and Protection, School of Civil and Mechanical Engineering, Curtin University, WA 6102, Australia. (*Corresponding author*) Email: thong.pham@curtin.edu.au
[4] John Curtin Distinguished Professor, Center for Infrastructural Monitoring and Protection, School of Civil and Mechanical Engineering, Curtin University, WA 6102, Australia. (*Corresponding author*) Email: hong.hao@curtin.edu.au

1. Introduction

One of the most challenging global issues is the alarming increase of CO_2 emission from cement manufacturing due to ever-growing demand for construction. CO_2 emission is also responsible for global warming which has negative effects on human health and our planetary ecology [1]. If no actions are taken, the amount of CO_2 emitted from the global cement industry is warned to reach 2.34 billion tons by 2050 [1, 2]. Therefore, it is urgent to investigate and introduce new “green” binders which could completely or partially replace the ordinary portland cement in the nearest future. GPC is so-called a “green” material because it uses industrial wastes (i.e., calcium fly-ash, slag, silica fume, rice-husk ash) to produce the new binder replacing ordinary portland cement [3]. Three main components in GPC include fly ash, slag, and alkaline chemicals. Fly ash is a product found in coal-fired power stations while slag is a left-over product after a targeted metal has been successfully refined out of its raw ore. If OPC is replaced by GPC, industrial wastes could be effectively recycled to produce new binders (i.e., slag and fly ash) in GPC.

GPC has been intensively researched for over 20 years. Although many different mixes have been proposed to achieve different strengths of GPC, the application of GPC in construction is still limited partially because of a lack of design guide of structures made of GPC. Most of existing studies focus on seeking the optimal mixture design and mechanical properties of GPC [4, 5]. Wang et al. [6] and Rafeet et al. [7] reported that increasing the slag/fly ash ratio could improve the mechanical properties of GPC (e.g., compressive strength, elastic modulus). Xie et al. [8] suggested that the use of GPC with a combination of 50% slag, 50% fly ash and 0.5 water/binder ratio showed good mechanical behaviours and workability. In addition, 80 °C for 12-24 hour was an optimum curing condition of GPC [6]. Therefore, various studies suggested that the use of GPC under heat-cured condition offers numerous advantages such as little drying shrinkage, low creep, high compressive strength and bond strength, and excellent resistance in acid and sulphate environments [9, 10]. Nevertheless, the mechanical properties of GPC also have some disadvantages. Among these disadvantages of reinforced-GPC structures, two unfavourable characteristics are low elastic modulus [11] and brittleness [12]. The low elastic modulus affects the stiffness degradation of structures while the ductility of the structure could be decreased due to the brittleness of GPC. Therefore, it is necessary to improve the weaknesses of GPC toward better performances.

Despite numerous studies on the mechanical properties of GPC, the behaviours of GPC-based structures are still understudied with contrary findings reported, which limits the wide applications of GPC in construction. Several studies presented that the performances of heat-cured GPC beams and columns were almost similar to those of OPC beams and columns. Therefore, the use of current standards for OPC to design the GPC beams and columns could be accepted [13, 14]. However, some recent investigations showed that the strength of ambient cured and over reinforced GPC beams and columns was overestimated by conventional sectional analysis procedures [15, 16]. Furthermore, current studies concentrated on heat-cured GPC, which could only be applied to precast concrete structures, but difficult to cast-in-situ concrete structures. A previous study has shown that the performances of ambient and heat-cured GPC are different [9], i.e. shrinkage and brittleness. Therefore, further studies investigating the structural performances of ambient cured GPC structures are necessary for possible wide applications of GPC to both precast and monolithic structures.

Beam-column joints are a crucial member of a building under earthquake loading because it relates to the strength development of the adjacent beams and columns [17]. Numerous recent devastating earthquakes across the world showed that if beam-column joints are destroyed, the buildings collapse even though the beams and columns are still in good conditions. Beam-column joints often fail by shear stress due to insufficient transverse rebars in these joint regions [18-22]. In most cases, the brittle shear failure is abruptly experienced, even without any cautionary evidence about the collapse of the structures [17, 23]. This unexpected failure is attributed to non-ductile performances of structures [17]. Therefore, it is necessary to enhance the ductility of the joints under seismic loading.

The application of dry joints could resolve many disadvantages of monolithic, wet, and hybrid joints such as long construction time, high construction cost, and negative effects on the environment [24, 25]. However, dry joints with steel bolts are vulnerable to corrosion which is commonly considered as one of the priciest issues and also the critical causes for structural deterioration. In some circumstances, the costs for maintaining and repairing deteriorated components can be incredibly greater than constructing the new ones [26, 27]. For instance, Kitane et al. [28] reported that during the period of 1998 to 2017, it cost the United States of America an annual average of $5.8 billion to maintain bridges. This issue could be effectively resolved by using non-corrosive FRP bolts with excellent corrosion resistance. However, the application of FRP bolts in the reality of structural engineering is still limited because these bolts have relatively low shear capacity if compared to conventional steel bolts. The tensile strength of FRP bolts is higher than that of steel bolts but the shear capacity, elastic modulus, and torsion resistance of FRP bolts are lower than those of steel bolts. It is noted that FRP bolts are made of anisotropic materials and behave linear-elastic stress-strain characteristic up to failure [19]. This feature could cause brittle failure if FRP bolts govern the main failure of the beam-column joints. Some researchers [29, 30] reported, however, that specimens did not show brittle failure if FRP material did not govern the main failure of specimens.

There have been a few published studies investigating GPC monolithic joints [31-34] while there has been no research examining the performances of ambient-cured GPC precast joints under cyclic loading in the open literature. The majority of previous studies related to GPC joints were based on testing on small size samples. No analytical model or design procedure has been proposed for GPC joints. According to the above review, ductility of the joint is crucial while GPC reveals very brittle performances at the peak load. Therefore, it is necessary to investigate joint performances using GPC cured under ambient condition. This paper aims to investigate the performances of a GPC monolithic joint and a new GPC precast joint type using CFRP bolts and concrete-end-plate (CEP). Additionally, an empirical model to calculate the maximum applied loads of the monolithic and precast joints made of GPC is proposed. In order to evaluate the effects of the GPC utility and the accuracy level of the proposed model, the behaviours and the results of the model are sequentially compared with those of the OPC control specimens. It should be noted that the performances of OPC monolithic joints have been fully investigated in many previous studies and model to design these OPC monolithic joints have been proposed in various standards [35-38].

2. Experimental program and analytical calculations

2.1. Design of the experimental specimens

Two monolithic joints and two precast joints, namely specimens MO1-MG2 and PO3-PG4 were prepared and tested in the study. The letters “O” and “G” denote the use of OPC and GPC to cast these specimens. The two precast joints used CFRP bolts with a diameter of 20 mm to connect beams and columns. The CFRP bolts were applied a prestress level of approximately 6 kN. Two 20-ton load cells were used to determine these tensile forces in the bolts (see Fig. 1). All the monolithic joints were designed following ACI 550R-96 [39] and ACI 352R-02 [38]. The use of longitudinal rebars and stirrups was based on the requirements of ACI 318-11 [40]. The two precast joints were designed in reference to the previous studies [17, 29] since no standards are available for this precast joint type. In addition, the weak beam-strong column principle is applied to design all the specimens. Therefore, cross-sections of the columns (200×200 mm^2) were larger than those of the beams (150×150 mm^2). These cross-sections were chosen based on previous studies [29, 41, 42]. The beams of the precast joints consisted of two parts: (1) Beam A and (2) the CEP. Details of specimen dimensions and rebars can be found in Fig. 2.

2.2. Mechanical properties of materials

Table 1 presents the mixture proportions of 1 m^3 GPC and OPC. Two specimens were cast with GPC concrete based on the mixed design of the previous study [43] as presented in Table 1. Low calcium fly ash (FA) and ground granulated blast furnace slag (GGBFS) were used as binder materials. Their chemical compositions are presented in Table 2. A mixture of 12M sodium hydroxide (NaOH) and D-grade sodium silicate (Na_2SiO_3) solution was utilized as an alkaline activator (Aa). It is noted that 480-g solid NaOH was mixed with 1-litre water to create 12M NaOH. The D-grade sodium silicate consisted of 14.7% Na_2O, 29.4% SiO_2, and 55.9% H_2O. 99% purity of solid NaOH and liquid Na_2SiO_3 were provided by Chem-supply Pty Ltd [44] and PQ-Australia Pty Ltd [45], respectively. Silica sand was used as fine aggregate. The mechanical properties of GPC and OPC were determined according to AS 1012.8.1-14 [46] and AS 1012.9.1-14 [47]. Three cylinders (200-mm height and 100-mm diameter) for compressive tests and three cylinders (300-mm height and 150-mm diameter) for splitting tensile tests were prepared for each GPC batch. The GPC mixture had low workability which needs be improved in future work. After the casting process, plastic sheets were utilized to cover the top surface of all the specimens. Ambient curing condition was applied for all the GPC specimens until the testing day. Specimens MO1 and PO3 were respectively tested on the 28th and 29th day while two GPC specimens were tested on the 56th day after casting. The testing-day compressive strength (f'_c) and tensile strength (f_{ct}) of GPC were 66.1 MPa and 5.5 MPa, respectively, while those of OPC were 38.4 MPa and 3.8 MPa, respectively. The mechanical properties of GPC and OPC were different due to different concrete batches. Two OPC specimens (MO1 and PO3) were cast with ready-mixed concrete from a local supplier whereas GPC specimens (MG2 and PG4) were mixed manually at a structural laboratory. This difference of the mechanical properties was also reported in some previous studies [29, 48-50]. As informed by the concrete supplier, the sizes of crushed stone aggregate and slump test results were 7 mm and 150 mm, respectively. Therefore, 7-mm crushed stone aggregates were also used in GPC batches to be compatible with OPC batches. The small aggregate was applied in this study to limit micro-crack width caused by aggregate restrained shrinkage [51]. 10-mm deformed steel bars were used for stirrups whereas the diameter of longitudinal rebars was 16 mm. The top and bottom longitudinal rebars were similarly chosen because cyclic loading was applied for these joints. CFRP bolts, nuts, and plates were supplied by a company in China [52]. As informed by the supplier, GB/T 1447-05 [53] was applied to check the mechanical properties of CFRP bolts. The number of the samples which were used for the testing was 20-30 with the length of 800-1000 mm. The properties of all the rebars and CFRP bolts provided by the manufacturer are summarized in Tables 3 and 4. The geometry and dimensions of the CFRP bolts and nuts are shown in Fig.3.

2.3. Test setup

Fig. 4 shows the typical test setup of all the specimens. As shown in this figure, vertical displacements of the top and bottom column ends were resisted by a hydraulic jack, hinge, and a strut system. This hydraulic jack applied an initial axial force of 15 kN to the column. It is noted that the applied axial force may bring beneficial effects to the joint behaviours [54] which, however, were not investigated in this study. The load was applied on the beam tip under manual displacement control by a 500-kN hydraulic jack with the level of 6-9 mm/min based on ACI 374.1-05 [55]. Fig. 5 shows the incremental cyclic load history. Two fully reserved cycles were applied at each drift ratio (DR) with an initial ratio of 0.25% to ensure that all the specimens showed linear elastic responses in this initial stage. For the precast specimens, the columns were the first to be set up on the reaction frame. Then, the beams were connected to the columns by four CFRP bolts with a diameter of 20 mm. The four holes on the beams and columns were created by four plastic tubes with the outside diameter of 21 mm. These plastic tubes were embedded into formworks and steel cages before the concrete casting and were removed one day after the casting. In order to easily remove these plastic tubes, a cutting line was created on each tube with an electric hand cutting machine. Outside tubes were also covered by cling wrap and oil. All CFRP bolts were applied a prestress level of 6.5 kN for Specimen PO3 and 5.3 kN for Specimen PG4.

2.4. Analytical model to estimate the maximum applied loads and the main failure position

In order to apply these GPC monolithic and GPC precast joints into reality, it is necessary to propose a model for designers to estimate the load-carrying capacity or the horizontal shear resistance (V_{jh}) in the middle zone of the joints. Based on this model, the main failure mode and failure position could be also determined. This section adopts the model proposed by Ngo et al. [29] with some modifications for the GPC specimens. The results of the proposed model are compared to the experimental results of this current and previous studies. This kind of dry joints could fail at the fixed-supports and the joint areas. Therefore, the load-carrying capacity (P_{max}) is the minimum of applied load (P_{max1}) and (P_{max2}) corresponding to the failure occurrence at these two locations. It is noteworthy that two assumptions were adopted to determine P_{max2} of the precast joints: (1) Only the middle stirrup inside CEP mainly resisted the shear force in the joint area [29], and (2) The middle stirrup and longitudinal rebar yielded at the maximum applied load.

2.4.1. Failure at the fixed-support

P_{max1} was calculated following the nominal moment strength (M_{n1}) of beams. The following formula was used to calculate P_{max1}:

$$P_{max1} = \frac{M_{n1}}{L_1} \tag{1}$$

where L_1 is the length of cantilever beams (L_1 is 550 and 350 mm for monolithic and precast specimens, respectively).

For Specimens MG2 and PG4, previous studies showed that current standards and available models of OPC beams can be applied to design GPC beams with high accuracy, however, some modifications are needed if the beam is over reinforced [16]. In reality, the cross-sections of beams need to be reduced to ensure requirements of architecture so the beams could be designed with over reinforced. Most of the previous studies have ignored this issue [56, 57]. In the current study, the specimens were designed over reinforced which ensures the practicality of the results. Therefore, the use of existing standards to calculate M_{n1} requires some modification in k_3 of the rectangular stress-block parameters [16]. In this study, M_{n1} of OPC specimens was determined based on ACI 318-11 [40] while M_{n1} of GPC specimens was calculated based on the study by Tran et al. [16] in which the value of k_3 was changed from 0.9 to 0.7. M_{n1} of all the specimens were calculated by using the nominal yield strength of rebars.

2.4.2. Failure in the middle zone

For monolithic specimens, P_{max2} was determined based on the nominal shear strength (V_{ACI}) recommended by ACI 318-11 [40] as follows:

$$V_{ACI} = \gamma\sqrt{f_c'}A_j \tag{2}$$

where γ depends on the effects of confinement in a joint, $\gamma = 1.7$, $\gamma = 1.2$, and $\gamma = 1.0$ if beams are confined at all four faces, two or three faces, and other cases of joint, respectively; the compressive strength of concrete is denoted as f_c', A_j is an effective joint area. If h_{col} and b_j represent the effective joint depth and width, respectively, and b_c and b_b denote the width of the column and beam (Fig. 6), respectively, it has

$$A_j = b_j h_{col}, b_j = min\{b_b + 2x,\ b_b + h_{col}\} \text{ if } b_c \geq b_b, b_j = b_c \text{ if } b_c < b_b \quad (3)$$

As shown in Fig. 7, P_{max2} of the monolithic specimens is determined as follows:

$$V_{c1} = T_3 - V_{ACI} \quad (4)$$

$$P_{max2} = \frac{V_{c1} H_c}{L_b} \quad (5)$$

where V_{c1}, T_3, H_c, and L_b are the shear force at column top, the tensile forces of the rebars, the height of the column, and the distance from the loading point to the centroid of the column, respectively.

For the precast specimens, stirrups were assumed to yield for calculating P_{max2}. Fig. 8(b) shows the free body diagram of the tested specimens. The tensile force in the top bolts (T_1) is expressed as follows:

$$T_1 = T_1^a + T_1^b = V_{hs2} + \gamma_r P_r \quad (6)$$

where P_r and γ_r are the initial prestress forces and the prestress rate lost in the bolts (i.e. γ_r is 0.25 and 0.84 for CFRP bolts and steel bolts, respectively [29]). It is noticeable that the tensile forces (T_1 and T_2) in the bolts consist of two components (i.e., T_1 including T_1^a and T_1^b) caused by the applied load and prestress force, respectively. At Section S_2-S_2, the horizontal shear force (V_{hs2}) is determined as follows:

$$V_{s2} = V_s sin\alpha + V_c \quad (7)$$

$$V_{hs2} = V_{s2} cos\alpha \quad (8)$$

where $V_s = nA_v f_{yt}$ refers to the shear resistance of the stirrups, n is the number of legs of stirrups, and f_{yt} and A_v are respectively the yield strength and cross-sectional area of stirrups. According to the previous study [29], only the middle stirrup inside CEP mainly resisted the shear force. Therefore, the middle stirrup was considered to determine V_s. For the shear resistance of the OPC, $V_c = \beta h_p b_p \sqrt{f_c'}$, $\beta = \frac{1}{6}$ is adopted [40], b_p and h_p denote the width and thickness of CEP. For the shear resistance of the GPC, $\beta = 0.29$ is adopted to determine V_c based on the upper value of the shear strength in ACI 318-11 [40].

Fig. 8(c) shows the free body diagram of the precast beam. P_{max2} is calculated as follows:

$$P_{max2} = \frac{T_1(H-2a_p)}{(L+0.5h_p)} \tag{9}$$

where L is the distance from the fixed-support to the loading point, a_p refers to the distance between the centroid of the top bolts and the extreme-top fibre of the CEP, H and h_p denote the height and thickness of the joint area, respectively.

The maximum applied load (P_{max}) of both joint types (i.e., monolithic and precast joint) is determined as follows:

$$P_{max} = min(P_{max1}, P_{max2}) \tag{10}$$

For the horizontal shear resistance *(V_{jh})* in the middle zone of the precast specimens. V_{jh} of the proposed model is determined based on the tensile forces in the bolts (T_1) and in the longitudinal rebars (T_3). T_1 was calculated by Eq. 6 whereas T_3 is calculated from a bending moment (M) when $P_{max2}$was determined by using Eq. 9. Therefore, V_{jh} can be determined as follows:

$$V_{jh} = T_1 - T_3 \tag{11}$$

This study also adopts the experiment results of Ngo et al. [29] and Saqan [24] to evaluate the accuracy of the proposed models. The maximum applied loads ($P_{exp-max}$) of all the tested specimens were taken from experimental results. Specimens PO3, P5-S-SP, and DB-TC failed in the joint area. Therefore, V_{jh} of these specimens is determined by Eq. (11), in which T_3 is calculated from the nominal yield strength of the longitudinal rebars. The main parameters are summarized in Table 5.

As can be observed from Table 5, the proposed model predicts that Specimen PO3, P5-S-SP, and DB-TC experienced the main failure in the joint areas whereas other specimens failed at the fixed-support. The variations of P_{max1} between the proposed model and the experiment results are from 10% to 32%. Specimen MG2 presents the highest variation of 32%. This high variation could be attributed to the inaccuracy in estimating the moment capacity of the GPC beam. It should be noted that no standards have been introduced to accurately estimate the nominal moment strength of GPC beams yet. Therefore, this study adopted a model proposed by Tran et al. [16], which modified the stress block parameters for use in GPC beams based on analytical derivations and limited testing data. The accuracy and the applicability of the proposed model by Tran et al. [16] need to be studied further since the beam moment strength depends on many parameters which are most likely nonlinear intercorrelated. The model proposed based on limited testing data and analytical derivations with an ideal assumption of beam conditions does not necessarily cover the beam conditions in this study. On the other hand, P_{max1} of Specimens MO1, MG2, PG4, and P4-S-SP-H were lower than the $P_{exp\text{-}max}$, indicating reasonable safety margin for the joints.

Concerning the failure in the joint areas, the maximum applied loads were well predicted by the proposed model with a variation between 2 and 26%. Also, the variations of the horizontal shear strength in Specimens PO3 and P5-S-SP were 26% and 46%, respectively. Specimens PO3 and P5-S-SP were over reinforced. The nominal yield strength of the rebars was adopted to calculate T_3. As a result, the high variation of the horizontal shear strength was attributed to this assumption. For example, only 1.3% variation of the horizontal shear strength in Specimen PO3 is observed if the actual strain of the rebars is adopted to calculate T_3. In addition, the variation of the horizontal shear strength between ACI 318-11 [40] and the experiment reached approximately 35% in Specimen PO3. This high variation is understandable because ACI 318-11 [40] model is meant for monolithic joints, not for precast joints with the CEP and bolts as in the current study.

2.5. Experimental results and discussion

2.5.1. General behaviours and failure patterns

All the specimens were tested under cyclic loading. CFRP bolts and the longitudinal rebars remained in a linear elastic range up to the maximum applied loads. No failure occurred in either the CFRP bolts or longitudinal rebars. For example, the maximum tensile strength of the CFRP bolts in Specimens PO3 and PG4 was 32 kN and 45 kN, which accounted for 32% and 45% of their ultimate tensile strength (100 kN), respectively. The main failure of all the tested specimens was governed by concrete as also reported in the previous studies [17, 29]. This design ensured that the application of CFRP material did not cause the brittle failure in this precast joint type. Fig. 9 shows the failure modes of all the specimens. All the columns of the precast specimens were designed with higher capacity compared to the beams. There was no failure on the columns of the precast specimens while some minor inclined cracks were observed on the columns of the monolithic specimens. This design ensured that all the joints satisfied the requirements of the weak beam-strong columns for the reinforced-concrete structures under earthquake loading. Fig. 10 shows that strain in longitudinal rebars of the beams was considerably higher than that of the columns. For instance, in Specimen PO3, the maximum strain in the longitudinal rebars of the beam was 2559 $\mu\varepsilon$ whereas that of the column was only 619 $\mu\varepsilon$. In addition, slips between the column and CEP were not recorded by LVDT (linear variable differential transformer) during the test due to high friction between their two interfaces. Therefore, shear stress did not cause failure in CFRP bolts during the tests.

As can be seen in Fig. 9, Specimens MO1 and MG2 exhibited similar failure patterns and trends of crack development whereas two precast specimens showed different failure modes and failure positions. Except for the concrete, two monolithic specimens were designed similarly. Specimen MO1 was cast by OPC while GPC was used to cast Specimen MG2. The vertical flexural cracks appeared initially at ±0.5% DR when the tensile strain of concrete at the beam soffit reached its nominal tensile strain. It is noted that cracks on Specimen MG2 developed later than those on Specimen MO1 at the same DR (see Fig. 11). For example, the inclined cracks on Specimen MO1 propagated into the joint at 1% DR whereas there were only flexural cracks appearing on the beam of Specimen MG2 at the same DR. This phenomenon could be attributed to the different tensile strengths of concrete. The tensile strength of GPC (5.5 MPa) was higher than that of OPC (3.8 MPa). Therefore, the tensile cracks and shear cracks developed in Specimen MG2 slower than those in MO1. After 1% DR, the inclined cracks rapidly spread into the joint region when the longitudinal rebars mainly contributed to resist the bending moment of the beam. The inclined cracks on both the monolithic specimens initially concentrated in the middle joint area and then these inclined cracks propagated to two corners of the column. In addition, minor concrete crushing appeared at the fixed-support at 2% DR for Specimen MO1 and 2.5% DR for Specimen MG2. Specimen MO1 reached the maximum applied load at 5% DR while that of Specimen MG2 was 4%. The different results are attributed to the brittleness of GPC at the peak load. Therefore, although cracks occurred later, the applied load dropped immediately when Specimen MG2 reached the peak load. Figs. 9(MO1) and 9(MG2) show that both the monolithic specimens failed due to the crushing of concrete and vertical cracks at the fixed-support of the beams.

According to the previous studies [24, 58-60], beam-column joints could fail due to either diagonal compressive forces or shear forces. However, data of the strain gauges attached on aluminium bars to measure concrete strain (see Figs. 12 and 13) show that the main failure of these dry joints was a result of the tensile cracks and shear cracks in the joint areas. Therefore, the use of spirals in the top and bottom zones (see Fig. 13) did not improve the peak loads compared to Specimen without spirals. Concerning the analysis in Section 2.4.2, increasing the diameter of the middle stirrups inside the CEP is a promising solution to enhance the performances of these dry joints. Both precast joints exhibited different failure modes as shown in Fig.9. Specimen PO3 failed in the middle zone of CEP whereas Specimen PG4 failed at the fixed-support. Flexural cracks also occurred quite early at 0.5% DR because concrete tensile strain at the fixed-support exceeded its limit. Following this stage, the longitudinal rebars of the beam mainly resisted bending moment. Therefore, tensile cracks spread into the middle area of the CEP. Numerous inclined cracks formed in the CEP from 0.5% to 3% DR as shown in Fig. 11(PO3). Two yellow cracks with a width of 1.5 mm caused the main failure of this specimen (see Fig. 9(PO3)). In addition, the appearance of cracks on the precast GPC specimen (PG4) also took longer than that on the precast OPC specimen (PO3). It indicates the tensile strength of concrete significantly affected the joint behaviours. Fig. 11(PG4) shows that only four minor cracks appeared on the CEP of Specimen PG4 at the peak load while various inclined cracks were distributed over the entire surface of CEP in Specimen PO3. This different failure mode could be attributed to the higher tensile strength (5.5 MPa) and brittleness of GPC as compared to OPC (3.8 MPa). High tensile strength of GPC concrete minimized the tensile crack development in the CEP of Specimens PG4. Therefore, the main failure mode was changed from the joint to the beam at the fixed-support as shown by the yellow curve in Fig. 9(PG4). After reaching the peak load, the inclined cracks continued to develop on the CEP surface of specimen PG4 until the end of the test. This performance is attributed to the strength hardening of longitudinal rebars. Therefore, the tensile stress in the longitudinal rebars still increased after achieving the maximum applied load. Furthermore, if the precast joints with OPC and GPC had the same compressive and tensile strength, there would be no significant difference between the failure mode and failure position of the two joints. Both precast joints might fail at the joint areas because tensile and shear cracks governed the main failure mode of these specimens. However, more brittle failure with more inclined cracks could be observed on the CEP of the precast specimen with GPC (PG4), compared to Specimen PO3 due to brittle characteristic of GPC material [12].

2.5.2. Hysteretic performance and energy dissipation capacity

Hysteretic response and energy dissipation capacity are the crucial characteristics to evaluate the performances of beam-column joints under seismic loads. A beam-column joint is considered having excellent energy dissipation capacity if the joint shows ductile behaviours without a considerable reduction of the effective stiffness and strength. The energy dissipation is calculated by the enclosed area (E_h) inside the hysteretic loop of each cycle [17]. The hysteretic responses and the energy dissipation capacities of all the specimens are shown in Figs. 14 and 15. Up to 1% DR, the energy dissipation capacity of all the specimens showed a similar trend and values since the response remains primarily in the elastic range. In general, the shape of the hysteresis loops of Specimens MO1 and MG2 was similar to each other while the two precast specimens revealed different hysteretic performances with less pinching observed in Specimen PG4 as compared to Specimen PO3. It should be noted that the pinching is associated with the considerable variations in the area of hysteresis loops. This observation could be explained that Specimen PG4 experienced fewer cracks than Specimen PO3 at the same DRs as shown in Fig. 11. As previously mentioned, the tensile strength of concrete considerably affects the crack development of these precast joints. Consequently, the use of high tensile strength concretes in Specimen PG4 limited the appearance and development of tensile cracks on the CEP. However, overall energy dissipation of Specimen PO3 was lower than that of Specimen PG4 from 1% to 3 % DR because the applied load per cycle of Specimen PO3 was lower than that of Specimen PG4. Therefore, the enclosed area (E_h) inside the hysteretic loop of each cycle of Specimen PO3 was lower than that of Specimen PG4. After reaching the maximum applied load at 3% DR, the failure modes of the two precast specimens were different. Specimen PO3 failed at the joint area whereas Specimen PG4 failed in the beam at the fixed-support. Wider flexural cracks were observed at the fixed-support of Specimen PG4, compared to the inclined cracks on CEP of Specimen PO3. Consequently, these wider flexural cracks combined with higher impact forces causing a sharp increase of energy dissipation of Specimen PG4 as compared to that of Specimen PG3.

Meanwhile, the monolithic joints revealed linear responses from the beginning of the test to 1% DR because most of the materials remained in elastic range in the initial stage. Therefore, less energy was dissipated in the early stage since only few minor cracks were formed. After 1% DR, the cracks on the beam and in the joint zone gradually developed causing the increase of the pinching on the hysteretic loop. Up to 3.5% DR, the energy dissipation of Specimens MO1 and MG2 was quite similar (see Fig. 15). However, when the DR increased from 3.5% to 6%, the energy dissipation of MO1 had a tendency to overcome that of MG2. This observation could be explained that the compressive and tensile strengths of GPC were higher than those of OPC. Therefore, the development of vertical cracks and crushing concrete at the fixed-support were minimized on Specimen MG2. More cracks on Specimen MO1 means it absorbed more energy than its counterpart.

As shown in Fig. 15, the energy dissipation of the dry joints (PO3 and PG4) was greater than that of the monolithic joint (MO1 and MG2) from 1% DR till the end of the test. For instance, the energy dissipation of Specimen PO3 was higher than that of Specimen MO1, approximately 62% at 3% DR. Fatter hysteretic loops of the dry joints, compared to the monolithic joints, caused the difference of energy dissipation between the two joint types. Above results proved that the proposed dry joints could be effectively applied in the earthquake-prone regions.

2.5.3. Envelope curves and maximum applied loads corresponding to DR

The envelope diagrams of the tested specimens were indicated in Fig. 16. It is noted that the envelope curve of Specimen PG4 is only up to 3.1% DR in push (+) direction and 3.5% in pull (-) direction because no data were recorded after achieving the peak load due to malfunction of the testing system. Overall, all the envelope curves in Fig. 16 were almost symmetrical in the push and pull directions due to the similar design of the top and bottom longitudinal rebars of the beams. However, the load-carrying capacity in the first direction of each cycle was slightly higher than that in the second direction. This phenomenon is attributed to the slight reduction of the applied load in the second direction due to damages in the specimens induced by the first cycle.

Two OPC Specimens (MO1 and PO3) had the same design with their counterparts GPC specimens (MG2 and PG4), respectively, except concrete. It is because the performances of GPC were the main parameter to be investigated in this study. The use of high strength GPC did not affect the shape of the envelope curve but affect the load-carrying capacity as shown in Fig. 16. For instance, Specimen PG4 achieved 54.6 kN at the peak load whereas the peak load of Specimen PO3 was 37.7 kN. This different load-carrying capacity was attributed to the different tensile and compressive strength of the two kinds of concrete. High tensile and compressive strengths of the GPC specimens improved the tensile crack resistance in the middle joint and the crushing of concrete at the fixed-support. Meanwhile, after reaching the peak load from 4% to 5% DR for Specimens MO1 and MG2, the applied load of these two specimens reached its plateau. The yielding and strength hardening of the rebars led to this favourable response which prevented the brittle failure of the beam-column joint and offered essential warnings before the joint could completely fail [29].

Table 6 summarizes the maximum applied loads with the specimens'DRs. In order to evaluate the ductility of the joint response, the DR is commonly utilized to compare between different beam-column joint types. The DR (R) is determined as follows:

$$R = D/l \tag{12}$$

where D and l are the vertical displacement and the beam length (l is 550 mm in this study). DR of the precast specimens is often lower than that of the monolithic specimens due to the discontinuity of the longitudinal rebars or the concrete between the beams and columns. For example, the DR of the monolithic joints usually achieves about 3% to 5% [50, 61] while precast joints only reach between 1.5% to 3% DR [24, 62]. This result explains why precast joints have not been popularly applied in earthquake-prone regions. In addition, standards require different DRs to be achieved for ductile moment-resisting frames. For instance, 2% DR is a requirement of ASCE 41-06 [63] while the requirement of CSA A23.3-07 [64] for structures built in seismic regions is 2.5% DR. The two precast specimens (PO3 and PG4) reached 3% DR while DR of the two monolithic specimens, namely MO1 and MG2, were 5% and 4%, respectively. Therefore, all the specimens using OPC and GPC could be effectively applied in seismic regions. It is noted that the DR of Specimen MG2 was lower than that of Specimen MO1, although they had the same design except the different properties of the concrete. This observation could be explained by two reasons: (1) GPC demonstrated very brittle failure at the peak load due to the material characteristics. (2) Specimen MG2 was cast by a high strength GPC (66.1 MPa). The brittle behaviours led to a low DR of Specimen MG2 as compared to Specimen MO1 (38.4 MPa) because concrete governed the main failure of the specimens.

Fig. 17 shows the maximum-load comparisons of the specimens. Overall, the proposed precast joints exhibited promising performances in DR and the maximum applied loads, compared to those of the existing research [24, 65, 66]. The peak loads of the two precast joints (PO3 and

PG4) were higher than both of the monolithic joints (MO1 and MG2), by approximately 26.6%. This excellent result might be attributed to the effects of the CEP. For failure at the fixed-support, the 550-mm cantilever beam of monolithic joints was longer than that of the precast joints (350 mm) whereas these beams had the identical cross-sections. Consequently, when the same load was applied, the bending moment at the fixed-support of monolithic joints was greater than that of the precast joints by approximately 57%. Therefore, it suggests that the utility of the CEP improves the maximum loads of these precast joints. For the failure in the joint areas, the maximum applied loads of the dry joints were determined by the height and thickness of the CEP. This study intentionally adjusted the thickness reported in the previous study [24]. As a result, the maximum applied loads of the precast joints dramatically enhanced when compared to those of the control monolithic joints. In addition, the peak load comparisons between the OPC group (MO1 and PO3) and the GPC group (MG2 and PG4) show that the peak load of the GPC group was 32% higher than that of the OPC group because the compressive strength (66.1 MPa) and the tensile strength (5.5 MPa) of the GPC group were higher than the compressive strength (38.4 MPa) and the tensile strength (3.8 MPa) of the OPC group. Therefore, it can be concluded that GPC could effectively replace OPC and meet performance requirements.

2.5.4. Ductility of joints

Ductility and drift ratio are two main parameters to evaluate the load-carrying capacity of structures without any considerable strength reductions. In this study, the ductility of beam-column joints is determined by the following equation [29, 67]:

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{13}$$

where Δ_u and Δ_y denote the ultimate and yield displacements of the joints at the ultimate and yielding load, respectively. The ultimate displacements of Specimens PO3 and PG4 were monitored at 90% post-maximum load and at the maximum load, respectively because of the limitation of equipment. Meantime, the ultimate displacements of the two monolithic joints were determined at 85% of the post-maximum load because the new equipment was used to replace the old one. For the yield displacement, there are numerous methods to determine this parameter. Fig. 18 shows the adopted method to define the ultimate and yield displacement of all the tested specimens.

All the specimens were designed so that concrete governs the failure modes such as concrete crushing, tensile cracks or shear cracks. As a result, the ductility of the specimen was governed by concrete rather than the CFRP bolts. The ductility comparisons of all the specimens are presented in Table 7. Specimen MO1 showed the highest ductility ($\mu = 2.4$) among the four specimens while the ductility of Specimens MG2, PO3, and PG4 was approximately 23%, 12%, and 50% lower than that of Specimen MO1, respectively. The ductility comparisons of the monolithic group and precast group present that the GPC specimens revealed lower ductility than the corresponding OPC specimens. For example, Specimens MO1 and MG2 had the same design. Also, the ultimate displacements of these two specimens were determined at the same 85% of the post-peak load. Nevertheless, the ductility of Specimen MG2 ($\mu = 1.8$) was lower than that of Specimen MO1 ($\mu = 2.4$). This reduction in the ductility of the GPC joint is attributed to the brittleness of high strength GPC. Meanwhile, the ductility of Specimen PO3 ($\mu = 2.1$) was quite close to that of the reference Specimen MO1 ($\mu = 2.4$). It is noted that the ultimate displacement of Specimens PO3 was determined at 90% of the post-peak loads while that of Specimen MO1 was 85% of the post-peak load. Therefore, it is expected that if the ultimate displacements of both specimens were monitored at the same 85% of the post-maximum loads, the ductility of Specimen PO3 might be similar or even greater than that of Specimen MO1. From the above analysis, the ductility of the specimens using the high strength of GPC should be improved in future research even though the current design still satisfies the requirements for the earthquake-prone region. Adding fibres in the mixture of the high strength GPC might be a potential solution to be considered for further investigations.

2.5.5. Effects of CFRP bolts on joint opening and stiffness

This study used two LVDTs to measure the joint opening of all the precast joints, as shown in Fig. 1. These LVDTs were set up in the horizontal direction on the top and bottom surfaces of the CEP. During the serviceability condition, the joint opening in precast beam-column joints is expected to close. However, when the applied load exceeds the serviceability condition (i.e., approximately 60-70% of the ultimate load [68]), the joint might open and then close after the load decreases. Joint openings of Specimens PO3 and PG4, which were measured by LVDT at the serviceability loads and maximum load, were around 0 mm and 1.6 mm, respectively. The marginal joint opening was attributed to the effects of low prestress levels in the bolts. The CFRP bolts were applied a prestress level of about 6 kN, as shown in Fig. 19. The torsion resistance of CFRP bolts was quite low so they were not prestressed to a high level. A high prestress level on CFRP bolts might lead to premature damage of the bolts. It is suggested that the CFRP bolts need to be prestressed to a higher level to minimize the joint opening. To resolve this issue, a new FRP bolts type and a new method to apply high prestress levels for FRP bolts are proposed and will be represented in another study.

The application of CFRP bolts in this precast joint type could also lead to lower stiffness during the initial stages (i.e., approximately 24%) compared to precast specimens with steel bolts [29]. This behaviour was attributed to the effects of elastic modulus than the prestress level on the bolts [29]. The elastic modulus of CFRP bolts (100 GPa) was lower than that of steel bolts (200 GPa). Therefore, the stiffness of the specimens with CFRP bolts was lower than that of the specimens with steel bolts. High prestress levels in bolts only result in a minor effect on the stiffness which was also reported in some other studies [69, 70]. In addition, the use of CFRP bolts to replace steel bolts in the proposed dry joints minimized a residual joint opening after resisting intensive load due to the linear behaviour of CFRP material.

2.6. Verification of predicted results

The results of the analytical model are verified with the experimental data in this section, including (1) the failure mode and (2) the maximum applied load. For the failure modes, it can be seen that there was a good correlation between the experimental observations and predictions of the proposed analytical model in Section 2.4. For instance, the proposed analytical model predicted that only Specimen PO3 could fail at the joint area while the other three specimens (MO1, MG2, and PG4) might fail in the beam at the fixed-support. This prediction coincided with the failure modes and failure position of all the specimens in the tests, as shown in Fig. 9 and Table 5.

Concerning the failure at the fixed-support, the proposed analytical model well predicted the maximum applied load of OPC Specimen MO1 with a variation of approximately 13% while the higher variation of 32% was observed on GPC Specimen MG2 due to the inaccuracy in estimating the moment capacity of the GPC beam (see more details in Section 2.4.2). For the failure at the joint area, the variation of the maximum applied load among analytical predictions and experimental tests was around 2-26% whereas only 1.3% variation of the horizontal shear strength was observed if the actual strain of the longitudinal rebars was adopted to determine T3. The above results proved that the proposed analytical model can predict the failure modes, peak load, and horizontal shear strength of the specimens. However, to improve the reliability of the proposed analytical model, it is necessary to use the numerical simulation to study some parameters which are too complicated to measure during experimental tests such as shear stress in CFRP bolts and the validity of the two assumptions in this analytical model. The proposed analytical model in this study can successfully offer the foundation for further studies.

3. Conclusions

This study conducted an experimental investigation on the structural performances of the ambient-cured GPC monolithic joints and the newly proposed dry joint using GPC and CFRP bolts under cyclic loading. A new analytical model to design GPC monolithic and GPC precast joints was also proposed. The results showed that GPC precast joint offered various advantages in terms of the load-carrying capacity and energy dissipation, compared to the monolithic joint. Nevertheless, GPC joints also revealed a reduction in the ductility due to brittle characteristic of GPC material. The following key points are drawn from the experimental results and theoretical predictions:

1. The proposed analytical model could well predict the main failure modes, failure positions, and failure load of both the monolithic and precast joints made of OPC and GPC. The analytical model predicted the horizontal shear strength of the precast specimen with a low variation of 1.3% while this variation of ACI 318-11 [40] model was 35%.
2. The crack development and failure mode of both OPC and GPC joints were similar. The tensile strength of concretes significantly affected several crucial parameters of the beam-column joints such as failure mode, load-carrying capacity, and energy dissipation.
3. The ductility of both GPC monolithic and GPC precast joints was lower than their counterparts OPC joints by approximately 22.9-42.8%. The GPC specimens showed brittle failure at the peak load.
4. Drift ratio of all the specimens was higher than 2.5%, which satisfied the requirements of ASCE 41-06 [63] and CSA A23.3-07 [64] standards for structures in seismic regions.

5. Both the GPC monolithic and GPC precast joints showed promising results in the indices of the peak load and energy dissipation compared to the corresponding OPC monolithic and OPC precast joints, respectively.
6. The application of CFRP bolts in the precast joints could minimize the residual joint opening after resisting intensive load. However, it showed an approximately 24% lower stiffness during the initial stages in the tests, compared to the precast specimens with steel bolts due to the lower elastic modulus of CFRP than steel.
7. CFRP bolts could replace steel bolts in the proposed dry joints to resolve corrosion problem while they still meet the design requirements of shear resistance and ductility for the dry joints.

In conclusion, this study suggests potential solutions for three main problems in the construction sector. Firstly, GPC could effectively replace OPC to reduce environmental pollution owing to reuse of industrial wastes. Secondly, the corrosion of the connecting elements in the conventional dry joints could be effectively mitigated by the application of CFRP bolts and plates. Finally, the new proposed dry joint type met the requirements for application in the earthquake-prone regions.

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Author contribution

Tuan T. Ngo: Data curation, Formal analysis, Investigation, Methodology, Visualization, Roles/Writing - original draft.

Tung T. Tran: Supporting the experimental process, Resources.

Thong M. Pham: Project administration, Supervision, Writing - review & editing.

Hong Hao: Funding acquisition, Supervision, Project administration, Writing - review & editing.

Compliance with ethical standards

Conflict of interest

The authors declare that they have no conflict of interest.

References

[1] Benhelal E, Zahedi G, Shamsaei E, Bahadori A. Global strategies and potentials to curb CO2 emissions in cement industry. J Cleaner Prod 2013;51:142-61.

[2] Castel A, Foster SJ, Ng T, Sanjayan JG, Gilbert RI. Creep and drying shrinkage of a blended slag and low calcium fly ash geopolymer Concrete. Mater Struct 2016;49:1619-28.

[3] Khan MZN, Shaikh FuA, Hao Y, Hao H. Synthesis of high strength ambient cured geopolymer composite by using low calcium fly ash. Constr Build Mater 2016;125:809-20.

[4] Khan MZN, Hao Y, Hao H, Shaikh FUA. Mechanical properties of ambient cured high strength hybrid steel and synthetic fibers reinforced geopolymer composites. Cem Concr Compos 2018;85:133-52.

[5] Ding Y, Dai J-G, Shi C-J. Mechanical properties of alkali-activated concrete: A state-of-the-art review. Constr Build Mater 2016;127:68-79.

[6] Wang J, Xie J, Wang C, Zhao J, Liu F, Fang C. Study on the optimum initial curing condition for fly ash and GGBS based geopolymer recycled aggregate concrete. Constr Build Mater 2020;247:118540.

[7] Rafeet A, Vinai R, Soutsos M, Sha W. Effects of slag substitution on physical and mechanical properties of fly ash-based alkali activated binders (AABs). Cem Concr Res 2019;122:118-35.

[8] Xie J, Wang J, Rao R, Wang C, Fang C. Effects of combined usage of GGBS and fly ash on workability and mechanical properties of alkali activated geopolymer concrete with recycled aggregate. Composites Part B 2019;164:179-90.

[9] Sumajouw MDJ, Rangan BV. Low-calcium fly ash-based geopolymer concrete: reinforced beams and columns, research report GC. Perth (Australia): Curtin University of Technology, 2006.

[10] Wallah S, Rangan BV. Low-calcium fly ash-based geopolymer concrete: long-term properties. Perth, WA , Australia: Curtin University, 2006.

[11] Noushini A, Aslani F, Castel A, Gilbert RI, Uy B, Foster S. Compressive stress-strain model for low-calcium fly ash-based geopolymer and heat-cured Portland cement concrete. Cem Concr Compos 2016;73:136-46.

[12] Thomas RJ, Peethamparan S. Alkali-activated concrete: engineering properties and stress–strain behavior. Constr Build Mater 2015;93:49-56.

[13] Nguyen KT, Ahn N, Le TA, Lee K. Theoretical and experimental study on mechanical properties and flexural strength of fly ash-geopolymer concrete. Constr Build Mater 2016;106:65-77.

[14] Visintin P, Mohamed Ali MS, Albitar M, Lucas W. Shear behaviour of geopolymer concrete beams without stirrups. Constr Build Mater 2017;148:10-21.
[15] Albitar M, Mohamed Ali MS, Visintin P. Experimental study on fly ash and lead smelter slag-based geopolymer concrete columns. Constr Build Mater 2017;141:104-12.
[16] Tran TT, Pham TM, Hao H. Rectangular stress-block parameters for fly-ash and slag based geopolymer concrete. Structures 2019;19:143-55.
[17] Ngo TT, Pham TM, Hao H. Ductile and dry exterior joints using CFRP bolts for moment-resisting frames. Structures 2020;28:668-84.
[18] Abbas AA, Syed Mohsin SM, Cotsovos DM. Seismic response of steel fibre reinforced concrete beam–column joints. Eng Struct 2014;59:261-83.
[19] Mady M, El-Ragaby A, El-Salakawy E. Seismic behavior of beam-column joints reinforced with GFRP bars and stirrups. J Compos Constr 2011;15:875-86.
[20] Hadi MNS, Tran TM. Seismic rehabilitation of reinforced concrete beam–column joints by bonding with concrete covers and wrapping with FRP composites. Mater Struct 2016;49:467-85.
[21] Paul RG, Melvin RR. Increased Joint Hoop Spacing in Type 2 Seismic Joints Using Fiber Reinforced Concrete. Structural Journal 1989;86.
[22] Wang G-L, Dai J-G, Bai Y-L. Seismic retrofit of exterior RC beam-column joints with bonded CFRP reinforcement: An experimental study. Compos Struct 2019;224:111018.
[23] Le-Trung K, Lee K, Lee J, Lee DH, Woo S. Experimental study of RC beam–column joints strengthened using CFRP composites. Composites Part B: Engineering 2010;41:76-85.
[24] Saqan EI. Evaluation of ductile beam-column connections for use in seismic- resistant precast frames. USA: University of Texas at Austin, 1995.
[25] Prabhakaran R, Razzaq Z, Devara S. Load and resistance factor design (LRFD) approach for bolted joints in pultruded composites. Compos Part B 1996;27:351-60.
[26] Lawler N, Polak MA. Development of FRP shear bolts for punching shear retrofit of reinforced concrete slabs. J Compos Constr 2010;15:591-601.
[27] Yunovich M, Thompson NG. Corrosion of highway bridges: Economic impact and control methodologies. Concr Int 2003;25:52-7.
[28] Kitane Y, Aref AJ, Lee GC. Static and fatigue testing of hybrid fiber-reinforced polymer-concrete bridge superstructure. J Compos Constr 2004;8:182-90.
[29] Ngo TT, Pham TM, Hao H. Effects of steel fibres and prestress levels on behaviour of newly proposed exterior dry joints using SFRC and CFRP bolts. Eng Struct 2020;205:110083.
[30] Maranan GB, Manalo AC, Benmokrane B, Karunasena W, Mendis P. Evaluation of the flexural strength and serviceability of geopolymer concrete beams reinforced with glass-fibre-reinforced polymer (GFRP) bars. Eng Struct 2015;101:529-41.
[31] Brooke N, Keyte L, South W, Megget L, Ingham J. Seismic performance of green concrete interior beam-column joints. Australian Structural Engineering Conference: Engineers Australia; 2005. p. 982.
[32] Datta M, Premkumar G. Comparative study of geopolymer concrete with steel fibers in beam column joint. IAEME Journals 2018;9:234-47.
[33] Raj SD, Ganesan N, Abraham R, Raju A. Behavior of geopolymer and conventional concrete beam column joints under reverse cyclic loading. Adv Concr Constr 2016;4:161-72.
[34] Saranya P, Nagarajan P, Shashikala AP. Behaviour of GGBS-dolomite geopolymer concrete beam-column joints under monotonic loading. Structures 2020;25:47-55.
[35] BS EN 1998-1-04. Eurocode 8: design of structures for earthquake resistance - Part 1: general rules, seismic actions and rules for buildings. BS EN 1998-1-04: European Committee for Standardization; 2004. p. 112-3.
[36] NZS 3101-06. Concrete structures standard - part 1: the design of concrete structures. NZS 3101-06. New Zealand, Wellington: Standards New Zealand; 2006. p. 137.

[37] AIJ-2010. Standard for structural calculation of reinforced concrete structures. AIJ-2010. Tokyo, Japan: Architectural Institute of Japan; 2010. p. 179-82.

[38] ACI 352R-02. Recommendations for design of beam-column connections in monolithic reinforced concrete structures. ACI 352R-02. Farmington Hills, MI: ACI (American Concrete Institute); 2002. p. 38.00.

[39] ACI 550R-96. Design recommendation for precast concrete structures. ACI 550R-96. Farmington Hills, MI: ACI (American Concrete Institute); 1996. p. 115-21.

[40] ACI 318-11. Building code requirements for structural concrete and commentary. ACI 318-11. Farmington Hills, MI 48331: ACI (American Concrete Institute); 2011. p. 503.

[41] Mukherjee A, Joshi M. FRPC reinforced concrete beam-column joints under cyclic excitation. Compos Struct 2005;70:185-99.

[42] Vidjeapriya R, Jaya KP. Experimental Study on Two Simple Mechanical Precast Beam-Column Connections under Reverse Cyclic Loading. J Perform Constr Facil 2013;27:402-14.

[43] Tran TT, Pham TM, Hao H. Experimental and analytical investigation on flexural behaviour of ambient cured geopolymer concrete beams reinforced with steel fibers. Eng Struct 2019;200:109707.

[44] Chem-supply Pty Ltd. https://www.chemsupply.com.au/. Australia2020.

[45] PQ-Australia Pty Ltd. https://www.pqcorp.com/. Australia2020.

[46] AS 1012.8.1-14. Method for making and curing concrete-compression and indirect tensile test specimens. AS 10128 1-14. Sydney, NSW: AS (Australian Standard); 2014.

[47] AS 1012.9.1-14. Methods of testing concrete-compressive strength tests-concrete, mortar and grout specimens. AS 101291-14. Sydney, NSW: AS (Australian Standard); 2014.

[48] Mahini SS, Ronagh HR. Web-bonded FRPs for relocation of plastic hinges away from the column face in exterior RC joints. Compos Struct 2011;93:2460-72.

[49] Hasaballa M, El-Salakawy E. Shear capacity of exterior beam-column joints reinforced with GFRP bars and stirrups. J Compos Constr 2015;20:04015047.

[50] Rahman R, Dirar S, Jemaa Y, Theofanous M, Elshafie M. Experimental Behavior and Design of Exterior Reinforced Concrete Beam-Column Joints Strengthened with Embedded Bars. J Compos Constr 2018;22:04018047.

[51] Grassl P, Wong HS, Buenfeld NR. Influence of aggregate size and volume fraction on shrinkage induced micro-cracking of concrete and mortar. Cem Concr Res 2010;40:85-93.

[52] J and R Metalwork Industry Pty Ltd. Quotation and Properties of CFRP bolts. China2018.

[53] GB/T 1447-05. Fiber-reinforced plastics composites-Determination of tensile properties. GB/T 1447-05. China: GB (Chinese Standard); 2005.

[54] Antonopoulos CP, Triantafillou TC. Experimental investigation of FRP-strengthened RC beam-column joints. J Compos Constr 2003;7:39-49.

[55] ACI 374.1-05. Acceptance criteria for moment frames based on structural testing and commentary. ACI 3741-05. Farmington Hills, MI: ACI (American Concrete Institute); 2005. p. 9.

[56] Karayannis CG. Mechanics of external RC beam-column joints with rectangular spiral shear reinforcement: experimental verification. Meccanica 2015;50:311-22.

[57] Hadi MNS, Tran TM. Retrofitting nonseismically detailed exterior beam–column joints using concrete covers together with CFRP jacket. Constr Build Mater 2014;63:161-73.

[58] Tran TM, Hadi MNS, Pham TM. A new empirical model for shear strength of reinforced concrete beam–column connections. Mag Concr Res 2014;66:514-30.

[59] Li J, Samali B, Ye L, Bakoss S. Behaviour of concrete beam–column connections reinforced with hybrid FRP sheet. Compos Struct 2002;57:357-65.

[60] Hwang S-J, Lee H-J. Strength prediction for discontinuity regions by softened strut-and-tie model. J Struct Eng 2002;128:1519-26.

[61] Ghomi SK, El-Salakawy E. Effect of joint shear stress on seismic behaviour of interior GFRP-RC beam-column joints. Eng Struct 2019;191:583-97.
[62] Jin K, Kitayama K, Song S, Kanemoto K-o. Shear Capacity of Precast Prestressed Concrete Beam-Column Joint Assembled by Unbonded Tendon. ACI Struct J 2017;114:51.
[63] ASCE 41-06. Seismic rehabilitation of existing buildings. ASCE 41-06. ASCE Reston, VA: ASCE; 2006.
[64] CSA A23.3-07. Design of concrete structures. CSA A233-07. Mississauga, Ontario: CSA (Canadian Standards Association); 2007.
[65] Hanaor A, Ben-Arroyo A. Prestressed bolting in precast concrete beam-column connection. Proceedings of the institution of civil engineers: Structures and buildings: Thomas Telford Services Ltd; 1998. p. 144-53.
[66] Palmieri L, Saqan E, French C, Kreger M. Ductile connections for precast concrete frame systems. ACI-Special Publication 1996;162-13:313-56.
[67] Park R. Evaluation of ductility of structures and structural assemblages from laboratory testing. Bulletin of the New Zealand national society for earthquake engineering 1989;22:155-66.
[68] Amr AA, Sami HR. Serviceability of Concrete Beams Prestressed by Carbon-Fiber-Reinforced-Plastic Bars. ACI Struct J 1997;94.
[69] Turmo J, Ramos G, Aparicio AC. FEM modelling of unbonded post-tensioned segmental beams with dry joints. Eng Struct 2006;28:1852-63.
[70] Turmo J, Ramos G, Aparicio ÁC. Shear behavior of unbonded post-tensioned segmental beams with dry joints. ACI Mater J 2006;103:409.

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Nomenclature

Notations

μ	ductility of joints	P_{max2}	applied load at the loading point of the beam with the failure in the middle zone
A_j	effective joint area	P_r	initial prestress forces of bolts
a_p	distance between the centroid of the top bolts and the extreme-top fibre of the concrete-end-plate	Q_1	applied load at 75% peak load
A_v	cross-section area of of stirrup legs	q_1	compressive stress in concrete on the left of the concrete-end-plate
b_b	width of beam	Q_2	applied load at 85% peak load
b_c	width of column	q_2	compressive stress in concrete on the right of the concrete-end-plate
b_j	effective joint width	Q_{max}	peak load
b_p	width of the concrete-end-plate	R	drift ratio
D	vertical displacement at the loading point	T_1	tensile force in top bolts
E_h	enclosed area inside the hysteretic loop of each cycle	T_{1a}	tensile force is caused by the applied load
f'_c	compressive strength of concrete	T_{1b}	tensile force is caused by the prestressing force
f_{ct}	tensile strength of concrete	T_2	tensile force in the bottom bolts
f_{yt}	yield strength of the stirrups	T_3	tensile force of the longitudinal rebars
H	height of the concrete-end-plate	T_4	compressive force of the rebars
H_c	height of the column	V_{ACI}	nominal shear strength of monolithic joint
h_{col}	effective joint depth	V_c	shear resistance of the concrete
h_p	thickness of the CEP	V_{c1}	shear force at column top
k_3	ratio that mentions the difference of cylinder and in-place strengths	V_{hs2}	horizontal shear force at Section S_2-S_2
l	beam length	V_{jh}	horizontal shear resistance in the middle zone of precast specimens
L_1	distance between the fixed-support and the loading point	V_s	shear resistance of the joint stirrups
L_b	distance between the loading point and the centroid of the column	V_{s2}	inclined shear force at Section S_2-S_2
M_{n1}	nominal moment strength of beams	β	an empirically derived function
n	number of stirrup legs	γ	effects of confinement in a joint
$P_{exp\text{-}max}$	load-carrying capacity from the experiment	γ_r	prestress level lost in bolts

P_{max}	load-carrying capacity	Δ_u	ultimate displacement
P_{max1}	applied load at the loading point of the beam with the failure at the fixed-support	Δ_y	yield displacement

Table 1

Mixture proportions of 1 m^3 GPC and OPC.

Materials	Unit	GPC	OPC
Sand	(kg/m^3)	630	534
7-mm coarse aggregate	(kg/m^3)	1100	1100
GGBFS	(kg/m^3)	160	-
FA	(kg/m^3)	240	-
Aa/binder ratio	-	0.6	-
Na_2SiO_3 solution	(kg/m^3)	172	-
12 M NaOH solution	(kg/m^3)	69	-
Na_2SiO_3/NaOH ratio	-	2.5	-
Coarse sand Gin Gin	(kg/m^3)	-	225
Cement	(kg/m^3)	-	400
Water	(L/m^3)	-	175
Plastiment BV35	(mL/m^3)	-	1600
Viscocrete 10	(mL/m^3)	-	1200
Viscoflow 15	(mL/m^3)	-	1200

Note: - = not applicable

Table 2

Chemical compositions of FA and GGBFS.

Composition (wt. %)	FA	GGBFS
Fe_2O_3	12.5	0.9
SiO_2	51.1	32.5
Al_2O_3	25.6	13.6
K_2O	0.7	0.35
CaO	4.3	41.2
MgO	1.5	5.1
MnO	0.15	0.25
Na_2O	0.8	0.3
P_2O_5	0.9	0.03
TiO_2	1.3	0.5
SO_3	0.24	3.2
Others	0.46	1.12
LOI[a]	0.6	1.1

Note: [a] Loss on ignition

Table 3

Rebar properties.

Diameters (mm)	f_y (MPa)	E_s (GPa)	Area (mm^2)	Notes
8	377	200	50	Spirals
10	560	200	78	Stirrups
16	597	200	201	Longitudinal rebars

Table 4

Details of CFRP bolts and nuts [52].

Names	Size	Tensile strength	Shear strength	Bending strength	Compressive strength	Ultimate load	Elastic modulus
	(mm)	(MPa)	(MPa)	(MPa)	(MPa)	(kN)	(GPa)
Bolts	φ20	≥ 850	≥ 160	480	760	≥ 267	100
Nuts	φ20	-	-	-	-	100	100

Note: - = not given

Table 5

The comparisons between the experimental and theoretical results of P_{max} and V_{jh}.

Names	Experimental results		Theoretical results (fixed-support)			Theoretical results (middle zone)						
	$P_{exp\text{-}max}$ (kN)	V_{jmax} (kN)	M_{n1} (kN.m)	P_{max1} (kN)	(%)	V_{ACI} (kN)	V_{c1}; T_1 (kN)	V_{hs2} (kN)	P_{max2} (kN)	(%)	V_{jh} (kN)	(%)
MO1	32.3	-	15.4	28.0	-13.3	216.9	23.2[a]	-	38.2	-	-	-
MG2	46.2	-	17.2	31.3	-32.3	284.6	44.4[a]	-	73.2	-	-	-
PO3	37.7	137.6	15.4	44.0	-	185.9	66.5[b]	64.9	37.0	-2	173.6	26
PG4	54.6	-	17.2	49.1	-10.0	-	100.7[b]	99.3	55.9	-	-	-
Ngo et al. [29]												
P4-S-SP-H	50.3	-	17.3	49.4	-1.7	185.9	107.7[b]	64.9	59.9	-	-	
P5-S-Sp	41.8	114.1	14.9	42.6	-	185.9	73.76[b]	64.9	41.0	-2	166.4	46
Saqan [24]												
DB-TC	499	*	337.8	422.3	-	-	708.3	271.5	367.1	26	*	*

Note: - = not applicable

* = lack of data

[a]Value of V_{c1}

[b]Value of T_1

Table 6

Maximum applied loads with corresponding DRs.

Names	Peak load (kN)		Increase (%)		Average (kN)	Increase (%)	DR at peak load (%)	
	Positive	Negative	Positive	Negative			Positive	Negative
MO1	25.8	32.3	-	-	29.1	-	5.0	5.0
MG2	30.7	46.2	18.7	43.3	38.4	32.3	4.0	4.0
PO3	35.8	37.7	38.6	16.9	36.8	26.6	3.0	3.0
PG4	54.6	42.7	111.2	32.2	48.6	67.3	3.1	3.6

Note: - = not applicable

Table 7

Ductility of tested specimens.

Names	Force	Q_{max}	Q_1	Δy	Q_2	Δu (85%)	$\mu=\Delta u/\Delta y$	Average (μ)	Decrease (%)
	Units	(kN)	(kN)	(mm)	(kN)	(mm)			
MO1	Positive	25.8	19.4	15.6	22.0	35.8	2.3	2.4	-
	Negative	32.3	24.2	14.5	27.4	35.6	2.5		
MG2	Positive	30.7	23.0	16.7	26.1	31.9	1.9	1.8	-22.9
	Negative	46.2	34.7	17.8	39.3	31.3	1.8		
PO3	Positive	35.8	26.9	12.2	32.2[c]	27.5[c]	2.3	2.1	-11.6
	Negative	37.7	28.3	11.1	34.0[c]	21.5[c]	1.9		
PG4	Positive	54.6	41.0	14.6	54.6[d]	17.3[d]	1.2	1.2	-50.0
	Negative	42.7	32.0	16.3	42.7[d]	19.6[d]	1.2		

Note:

[c]At 90% of the post-maximum applied load

[d]At the maximum applied load

\- = not applicable