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Abstract

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Keywords

geopolymer, concrete, geogrid-confined, piles, concept, frp-pvc confined, core:, pervious, behaviour

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Geogrid-confined pervious geopolymer concrete piles with FRP-PVC-confined concrete core: concept and behaviour

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Abstract:

The use of fibre reinforced polymer (FRP) and polyvinyl chloride (PVC) as strengthening materials for piles was found to be a promising scheme, due to their high strength-to-weight ratio, high durability and high anti-corrosion ability. This study presents an experimental investigation of a new form of composite piles: geogrid-confined pervious geopolymer concrete piles (GPGCPs) with fibre reinforced polymer (FRP)-polyvinyl chloride (PVC)-confined concrete core (FPCC). The GPGCP with FPCC consists of a circular geogrid outer tube, a FRP-PVC-confined normal geopolymer concrete core, and pervious geopolymer concrete (PGC) filled in between. The reason for applying PGC into piles is to increase the rate of consolidation. The aim of using FPCC is to improve the compressive strength and ductility of the concrete. In this study, two groups of GPGCPs (without and with FPCC) were prepared and tested under axial compression. In each group, one layer, two layers, and three layers of geogrid were used to investigate the influence of the outer tube. The test results show that the FPCC can significantly improve the mechanical behaviour of the GPGCPs. In comparison with GPGCPs without FPCC, the maximum axial loads of GPGCPs with FPCC were higher, and the ductility was improved significantly.

Keywords: FRP; PVC; Geogrid; pervious geopolymer concrete

1. Introduction

The use of FRP has been increasing significantly in civil engineering applications, due to its high strength-to-weight ratio, high durability, high anti-corrosion ability, satisfactory fire endurance and bond strength [1-12]. The FRP materials can be used as various forms of lateral confinement to strengthen the concrete columns, including fibre reinforced polymer (FRP) tubes [1-4], FRP rings [13], FRP stirrups [14] and FRP helices [15], FRP grids [16, 17]. Also, the externally bonded FRP systems have been proven to be an effective way to strengthen the concrete beams or girders [18, 19]. In addition, for piles, which are constructed in a marine environment, confinement by FRP is superior to steel materials due to the higher durability and anti-corrosion ability [20-23]. The USA is reported to spend more than \$1 billion annually to maintain waterfront piling systems. Therefore, FRP materials are regarded as suitable alternatives in marine environments [24].

However, because the ultimate strain and ductility of FRP are relatively low, some researchers have tried to apply plastic pipes such as high-density polyethylene (HDPE) and polyvinyl chloride (PVC) to solve these problems. Kurt [25] first conducted experimental tests and theoretical analysis on commercially available PVC tubes containing a concrete core. The PVC tubes were found to increase the strength of the concrete core to approximately 3.2 times the pipe burst pressure. Toutanji and Saafi [26] proposed a new hybrid column system consisting of concrete-filled PVC tubes reinforced with external continuous impregnated FRP hoops at various spacings. The results showed that using PVC-FRP tubes is an effective confinement technique that can significantly increase both the strength and failure strains of concrete. Fakharifard and Chen [27] investigated the mechanical behaviour of FRP-confined, PVC-confined and FRP-PVC-confined concrete. The results showed that the ultimate strength and ultimate strain of FRP-PVC-confined concrete specimens are larger than those of concrete columns confined by FRP alone or PVC alone.

Buildings, structures and highway facilities are sometimes constructed on saturated soft soil areas. To meet the requirements of allowable settlements and avoid failures, ground-improvement technologies are applied. The use of permeable granular piles is one of the common methods and includes sand compaction piles, stone columns, and rammed aggregate piers. The permeable granular piles can improve the load carrying capacity and reduce the settlement, which is similar to concrete, steel and wooden piles. However, the permeable granular piles have some advantages over concrete, steel and wooden piles. The permeable granular piles can increase the rate of consolidation and reduce the liquefaction potential caused by seismic and traffic loading [28-33]. The fast consolidation rate can effectively improve the elastic modulus of soft soil between piles, and reduce the differential deformation between the top surface of piles and soft soil. However, the strength and stiffness of permeable granular piles are relatively low, and their suitability in soft soils is limited.

To solve this problem, two improvement methods can be applied. One method is to apply pervious concrete piles as alternatives to granular piles. Pervious concrete piles can provide higher stiffness and strength than those of granular piles without reducing the permeability [34, 35]. The compressive strength of pervious concrete piles has been proven to be more than 10 times the compressive strength of granular piles with similar permeability [34]. The other method is to use geogrid and geotextile to encase the granular materials. These artificial materials have been proven to significantly improve the strength of permeable granular piles without affecting the drainage capacity. Recently, geogrid was used to confine and strengthen concrete members. Concrete columns confined with geogrids have been found to provide higher deformation capacity than unconfined concrete specimens [17, 36]. Until now, no studies combined the advantages of pervious concrete and artificial materials to improve the performance of piles.

Recently, geopolymers have gained a significant attention because the geopolymer binder can replace ordinary Portland cement (OPC) and substantially reduce the emission of CO₂ [37].

Several studies have investigated the performance of pervious geopolymer concrete (PGC) [38, 39]. It has been proven that the properties of PGC are similar to those of conventional pervious concrete.

In the present study, a new composite confinement system named geogrid-confined pervious geopolymer concrete piles (GPGCPs) is proposed. The aim of this new pile form is to combine the advantages of FRP, PVC, and PGC. The tests were divided into two groups: GPGCPs with and without FPR-PVC-confined concrete core (FPCC). The FPCC was made by filling the normal geopolymer concrete (NGC) into FPR-PVC tubes. One layer, two layers and three layers of geogrid tubes were used to investigate the effect of the amount of geogrid on the axial compressive behaviour of the concrete specimens. Here, the rationale for the new pile form and the testing results are presented to characterise the expected advantages.

2. Experimental Programme

2.1. Preliminary Tests

The preliminary tests examined the properties of four pairs of samples, including one pair of plain PGCs, one pair of plain NGCs, one pair of FRP-PVC tubes, and one pair of FPCCs. The aim of plain NGC and PGC tests is to obtain the axial stress-axial strain curves and the ductility of the reference samples. The diameter of NGC samples and PGC samples was 150 mm and their height was 300 mm. The aim of FRP-PVC tube and FPCC tests is to obtain their contribution to the vertical load carrying capacity of GPGCPs. The inner diameter and outer diameter of PVC tubes were 76 mm and 84 mm, respectively. The height of PVC tubes was 325 mm. The inner diameter and outer diameter of FRP-PVC tubes were 76 mm and 86 mm, respectively. The height of FRP-PVC tubes was 325 mm. The FPCC samples were made by filling NGC into FRP-PVC tubes. One strain gauge was used for each FPCC sample to obtain the hoop strain during the axial compression test. This

strain gauge was attached transversely onto the mid-height of the outside surface. The details of FPCC are shown in Fig. 1.

The four pairs of samples are named as follows: (a) P denotes plain PGC; (b) N denotes plain NGC; (c) FP denotes FRP-PVC tubes; (d) FPCC denotes the FRP-PVC-confined NGC; (e) the last numbers 1 or 2 are used to distinguish between the two nominally identical specimens. The details of preliminary tests are summarised in Table 1.

2.2. Test Matrix

A total of 12 GPGCP specimens were cast and tested under axial compression. All specimens were 160 mm in diameter and 325 mm in height. These specimens were divided into two groups. The six specimens of the first group did not have FPCC (shown in Fig. 2(a)). The six specimens of the second group had FPCC (shown in Fig. 2(b)). To ensure more representative results, two nominally identical specimens for each specimen configuration were tested.

The labelling of each specimen is named as follows: (a) GG denotes GPGCPs without FPCC, and the number afterwards denotes the number of geogrid layers (one, two and three layers); (b) GGC denotes GPGCPs with FPCC, and the number that follows denotes the number of geogrid layers (one, two and three layers); (c) the last number 1 or 2 is used to distinguish between the two nominally identical specimens. For example, Specimen GG2-2 represents the second of the two identical GPGCPs without FPCC that were confined with two layers of geogrid. The details of the GPGCPs specimens are summarized in Table 2. The detailed section configurations are shown in Fig. 3.

2.3. Preparation of Specimens

2.3.1. Geopolymer Concrete

The mix proportions for PGC and normal geopolymer concrete (NGC) are shown in Table 3. All mixes were conducted under ambient conditions (23 ± 2 °C). In this study, ground granulated blast furnace slag (GGBFS) and Class F fly ash (FA) were used as an aluminosilicate source. The GGBFS was supplied by the Australasian Slag Association, Australia [40], and the Class F FA was provided by the Eraring Power Station, Australia [41]. The FA was classified as Class F according to AS 3582.1 [42]. The alkaline activator was made by blending sodium hydroxide solution with sodium silicate solution. The concentration of sodium hydroxide solution was kept constant (14 M) for all mixtures. Sodium silicate solution was purchased from a local commercial supplier. The mass ratio of SiO_2 to Na_2O of the sodium silicate was 2.02 with a chemical composition of 29.6% SiO_2 and 14.7% Na_2O . The sodium hydroxide solution and sodium silicate solution were mixed together for 1 hour before being mixed with the aluminosilicate materials. The size of the coarse aggregates ranged from 5 mm to 12 mm.

The mixing procedures for the normal and PGC samples were the same. The geopolymer concrete was prepared by mixing the dry materials with the alkaline activator in a Lightburn 65 litre mixer. The dry materials were dry mixed for 2 min, then the alkaline activator was added to the mix and mixed for 1 min. Then, the additional water was poured into the mixer and mixed for another 2 min. After mixing, the fresh concrete was cast layer by layer into the moulds, and the depth of every layer was approximately 50 mm. Each layer was vibrated for 10 s on a vibration table. The specimens were left in the laboratory at an ambient condition for 24 hours, then covered by hessian clothes to prevent losing moisture from the concrete.

2.3.2. Geogrid tubes

For GPGCPs, to provide lateral confinement to the concrete specimens, the geogrid was formed into tubular shapes, and the ends were held with plastic ties. To ensure that the geogrid would not be loosened or slid under axial load, the geogrid was overlapped at an approximate length of 80

mm. To maintain the same dimensions of all concrete cores, the inner diameter of the geogrid tubes was kept at 160 mm (not including the thickness of the geogrid tube). The height of geogrid tubes was 325 mm.

2.3.3. FRP-PVC tubes

The PVC tubes were wrapped by GFRP to form FRP-PVC tubes using the wet layup method. The inner diameter and outer diameter of PVC tubes were 76 mm and 84 mm, respectively. The height of PVC tubes was 325 mm. At first, the GFRP sheets were impregnated with a mixture of epoxy resin and hardener at a ratio of 3:1. After that, the impregnated GFRP sheets were wrapped on the PVC tubes in five layers with an overlapping length of 150 mm. The FRP-PVC tubes were then cured in the laboratory for 1 day. These FRP-PVC tubes were used as the mould for casting the NGC. The outer diameter and height of FRP-PVC tubes were 86 mm and 325 mm, respectively. The inner diameter was same as that of PVC tubes (76 mm). The detailed section configuration of FPCC is shown in Fig. 2(a).

2.4. Material Properties

2.4.1. Geopolymer Concrete

The compressive strength of the plain pervious and normal geopolymer concrete was determined according to AS 1012.9 [43]. Three NGC cylinders and three PGC cylinders with 100 mm diameter and 200 mm height were tested to determine the 28-day compressive strength. Because the two groups of geogrid-confined pervious geopolymer concrete piles (GPGCPs) were produced in different batches, the compressive strengths of the PGC in the two groups of tests were different. However, the difference was minimal. The average 28-day compressive strength of PGC for GPGCPs without FPCC was 22.1 MPa; that of GPGCPs with FPCC was 21.7 MPa. The average 28-day compressive strength of NGC for GPGCPs with FPCC was 49.2 MPa. The permeability of

PGC were obtained by using the method proposed by Aoki et al. [44], and its average value was 9.1 mm/s.

2.4.2. Geogrid

The uniaxial geogrid (shown in Fig. 4(a)) were used as the outer confinement material. The inner dimensions of its square openings are 25 mm $\times$ 25 mm, which was measured by a standard ruler with an accuracy of 0.5 mm. The geogrid was manufactured from glass fibre with a bitumen coating. The uniaxial geogrid can resist corrosion and has a large tensile rupture strain. The widths of the transverse ribs and the longitudinal ribs were measured by a digital Vernier calliper, which are 10 mm and 6 mm, respectively. The thicknesses of the transverse rib and the longitudinal ribs were measured by a digital Vernier calliper, which are 1.5 mm and 1.0 mm, respectively. The mechanical properties of the uniaxial geogrid were determined by applying the ASTM D6637-M15 standard [45]. The length of each test transverse rib between the testing machine clamps was 150 mm, which was measured by a standard ruler with an accuracy of 0.5 mm. Five single transverse geogrid ribs were prepared and tested by using 100 kN Instron testing machine at the High Bay Laboratory, University of Wollongong, Australia. The tensile testing rate was 5 mm/min. The tensile load-axial strain curves of the geogrid are depicted in Fig. 4(b). The average tensile ultimate load of geogrid ribs was 2.8 kN and the average ultimate tensile strain was 9.2%.

2.4.3. Fibre reinforced polymer

In this study, glass fibre reinforced polymer (GFRP) was chosen as the reinforcement material. The GFRP sheets were formed from bidirectional glass fibre and had a nominal thickness of 0.15 mm. The GFRP sheets were first impregnated with a mixture of epoxy resin and hardener at a ratio of 3:1. Then, these sheets were cured in the laboratory for 1 day. Flat coupon tests were conducted according to ASTM D7565 [46]. The width of the test coupon was 25 mm and the length measured between the testing machine clamps was 200 mm. The longitudinal strain was measured by using

3 strain gauges on the two sides of the test coupon (shown in Fig. 5). The aforementioned 100 kN Instron testing machine was used to test the coupons at a loading rate of 2 mm/min. Five coupons were tested. The test results showed that the average tensile strength, elastic modulus and ultimate tensile strain were 621.5 MPa, 33.4 GPa and 1.86%, respectively, as shown in Table 4.

2.4.4. PVC

The commercial PVC pipes were used in this study, with an outside diameter of 84 mm and a thickness of 4 mm. The tensile tests of PVC coupons were conducted based on ASTM D638 [47]. Five dog-bone coupons (Fig. 6(a)) were cut from the PVC tube. The width of the narrow section of the dog-bone coupon was 13 mm and the length between the testing machine clamps was 115 mm. The tensile tests were carried out using the aforementioned 100 kN Instron testing machine at a loading rate of 5 mm/min. Figure 6(b) represents the tensile load-axial strain behaviour of the PVC coupon tests. The average tensile strength, fracture stress, elastic modulus and ultimate tensile strain were 51.2 MPa, 42.7 MPa, 1.58 GPa and 55.4%, respectively, as shown in Table 4. The PVC coupon specimens were found to exhibit much higher ductility than the FRP coupons.

2.5. Instrumentation and testing procedure

All of the preliminary test samples and GPGCP specimens were tested by using the Denison 5000 kN testing machine in the High Bay Laboratory at the University of Wollongong, Australia. The GPGCP specimens were capped with high-strength plaster at the top and bottom ends to ensure that the axial compression load was evenly applied onto the specimens. The axial deformation were measured using two Linear Variable Differential Transformers (LVDTs), which were fixed at the opposite corners between the loading and supporting steel plates. All the specimens were axially loaded up to 50 mm displacement at a rate of 1 mm/min.

For each FRP-PVC confined concrete (FPCC) specimen, a strain gauge was attached transversely onto the mid-height of the outside surface (shown in Fig. 1). The purpose of this gauge was to obtain the hoop strain during the tests.

For each GPGCP without FPCC specimen, three strain gauges were used to monitor the hoop strain. All these strain gauges were attached transversely onto the outside surface of transverse ribs of geogrid tubes. One strain gauge was attached at the mid-height transverse rib of the geogrid tubes, and two other strain gauges were attached at the second top and second bottom transverse ribs of the geogrid tubes, respectively. Figure 3(a) clearly shows the locations of these strain gauges.

For each GPGCP with FPCC specimen, four strain gauges were used to monitor the hoop strain. Three of them were attached transversely onto the outside surface of transverse ribs of geogrid tubes (shown in Fig. 3(b)). Their positions are same as those of GPGCPs without FPCC specimens described above. Another strain gauge was attached at the mid-height of the outside surface of the FRP-PVC-confined concrete, shown in Fig. 1.

In general, the ultimate axial strains of confined concrete columns are no more than 5% [3, 48, 49] because after reaching this value of axial strain, the load carrying capacity of columns decreases significantly, and the small remaining load carrying capacity is not considered. Another reason is that such large deformation are not allowed for structural members in practice. However, for granular piles encased by geosynthetics in the laboratory or on site, the axial strain can reach as high as 15% [50]. Therefore, in this study, to investigate if the GPGCPs can allow for such high deformation, the ultimate axial deformation was set to 50 mm, corresponding to an axial strain of 15.4%.

3. Test results and analysis

3.1. Preliminary tests

The preliminary test results are summarised in Table 5. The axial load-axial strain relationships of two plain PGC specimens and two plain NGC specimens are shown in Fig. 7. All plain PGC specimens and NGC specimens failed due to the crushing and spalling of concrete at the mid-height of the specimens.

To estimate the contribution of FRP-PVC tubes to the axial load carrying capacity, two specimens were axially loaded. Figure 8(a) shows that the two FRP-PVC tubes failed due to local elephant-foot buckling and rupture of the GFRP. The axial stress-axial strain relationships are illustrated in Fig. 8(b). The compression tests were stopped when the axial deformation reached 50 mm (the axial strain was 15.4%). The axial behaviour of the FRP-PVC tubes consisted of three branches. In the first branch, the axial stress increased rapidly to the peak stress. In the second branch, the FRP jacket ruptured, and the axial stress decreased sharply to a low level. After that, in the third branch, the axial stress was kept stable at a very low level.

Two FRP-PVC-confined concrete specimens were tested to determine their compressive behaviour, and their dimensions and section configurations are shown in Fig. 1. The final state of the FRP-PVC-confined concrete is shown in Fig. 9. The FRP jacket separated from the specimens, and the PVC tubes expanded and were distorted significantly. The axial stress-axial strain behaviour, shown in Fig. 10, is similar to that observed in a previous study conducted by Fakharifar and Chen [27]. The axial stress-axial strain curve consists of a parabolic first branch, a linear second branch and a descending third branch. The peak stress was reached at the end of the second branch when the GFRP ruptured. Then, the axial stress decreased rapidly and gradually stabilized. The compression tests were continued until the specimens reached an axial deformation of 50 mm (axial strain of 0.154). Figure 11 represents the axial strain versus hoop strain curves of two FPCCs from the beginning of the tests to the rupture of the GFPR jackets.

3.2. Final state of GPGCPs

The typical final state of the representative specimens after the tests is illustrated in Fig. 12 and Fig. 13. In this study, only two specimens failed due to the failure of the geogrid: GG1-1 and GG1-2, which were wrapped by one layer of geogrid without FPCC. One of the failed specimens is shown in Fig. 12(a). All other specimens reached an axial strain of 15.4% (corresponding to an axial deformation of 50 mm) without obvious failure. During the tests, for all GPGCPs, only a small number of coarse aggregates were squeezed out through the opening of the geogrid. Although the expansion of specimens was large, the geogrid tube held most of the coarse aggregates in the pervious geopolymer concrete.

Further examination of the final states of the GPGCPs without FPCC specimens indicated that the upper half of GG2 specimens expanded more due to the non-uniform dilation of the PGC, as shown in Fig. 12(b). However, the GG3 specimens expanded uniformly, and no prominent local bulging was observed, as shown in Fig. 12(c). For the final states of the GPGCPs with FPCC, the GGC1 specimens expanded at the mid-height of the specimen, as shown in Fig. 13(a). The specimens of GGC2 and GGC3 expanded uniformly over the entire height (shown in Fig 13(b)(c)), which were similar to GG3 specimens.

3.3. Mechanical behaviour of GPGCPs without FPCC during the axial compression test

3.3.1. Axial stress-axial strain behaviour

The key testing results of GPGCPs without FPCC are summarized in Table 6. The detailed axial stress-axial strain behaviour of the three pairs of GPGCPs without FPCC specimens is depicted in Fig. 14. All specimens showed a similar mechanical performance. The axial stress-axial strain behaviour of these specimens of GPGCPs without FPCC can be divided into three branches. In the first branch, their axial stress increased rapidly to the peak stress. It was found that their peak stress was not enhanced and was very close to the compressive strength of the unconfined PGC. The axial strains at the peak stress were also close to the axial strains of the unconfined PGC. In other words,

the geogrid tubes alone did not enhance the compressive strength of the PGC. This is because the confinement effect provided by geogrid tubes was small due to the low elastic modulus and large openings of the geogrid tubes.

In the second branch, after the peak stress, the axial stress decreased sharply. Even though the lateral expansion of the concrete became larger at this stage, the confinement effect provided by the geogrid was not significant because the tensile elastic modulus of the geogrid was relatively low and the openings were large. During the test, although the coarse aggregates did not spall from the openings of geogrid, these aggregates were somewhat ejected.

In the third branch, after the pronounced reduction in the axial stress, the GPGCPs without FPCC lost approximately 70% to 80% of the load carrying capacity. However, after an axial strain of 0.04, the axial stress stabilized without any significant change because with increasing the axial strain, the hoop strain became more pronounced, and the confining pressure provided by the geogrid became much higher. When the confining pressure achieved a certain level, the axial stress of the confined concrete was held steady even though the axial strain was large. However, Specimens GG1-1 and GG1-2, which had only one layer of geogrid, failed before reaching an axial strain of 15.4%.

The different layers of the geogrid had a significant influence on the mechanical behaviour in the third stage only. With additional layers of geogrid, the stable axial stress level increased. It is estimated that the stable axial stress increased by approximately 1.2 MPa for each additional layer of geogrid.

3.3.2. Axial-hoop strain behaviour

Figure 15 shows the axial-hoop strain curves of GPGCPs without FPCC. The axial strains were obtained from the measurement of the linear variable differential transformer (LVDT), while the

hoop strains were averaged from three strain gauges attached at the geogrid ribs. When the axial strain was smaller than 0.01, the differences of the three pairs of specimens were very close. However, when the axial strain increased from 0.01 to the end, the difference between the specimens became increasingly significant. With additional geogrid layers, the rate of increase of the hoop strain decreased considerably. Here, more layers of geogrid resulted in more confining pressure, leading to smaller hoop strain.

3.4. Mechanical behaviour of GPGCPs with FPCC during the axial compression test

3.4.1. Axial stress-axial strain behaviour

The key test results of this part are summarized in Table 7. The detailed axial stress-axial strain behaviour of the three pairs of GPGCPs with FPCC specimens is depicted in Fig. 16. All specimens in this part exhibited similar mechanical performance, and the most notable result was the presence of two peak axial stresses.

The axial behaviour of these specimens consisted of four branches. In the first branch, the axial stress of GPGCPs with FPCC increased rapidly to the first peak axial stress, which is similar to GPGCPs without FPCC. The axial strains at the first peak axial stress were close to the axial strain at the peak axial stress of unconfined PGC (around 0.002). During the tests, when the specimens reached the first peak axial stress, the outer PGC cracked. In other words, at that moment, the outer PGC reached its peak axial stress.

In the second branch, after the first peak stress, the axial stress decreased rapidly. At the end of this branch, the axial stress of all specimens decreased by approximately 5 MPa, and the axial strain reached approximately 0.007.

In the third branch, the axial stress increased by approximately 5 MPa and reached the second peak axial stress. At the end of the third branch, the axial strain reached approximately 0.025, at which

point the GFPR jacket ruptured, as confirmed by audible cracking of the GFRP jackets. The reason for the increase in the axial stress is that with increasing axial strain, the PFCC provided more load carrying capacity. This observation can be confirmed by the test results of FPCC alone in Fig. 10, showing that the axial stress increased until the axial strain reached approximately 0.025. It can be found from the Table 7 that the values of second peak axial stress were slightly smaller than the first ones. Therefore, the first peak axial stress were the maximum axial stress.

In the last branch, after the second peak axial stress, the axial stress decreased until the end of the compression tests. The rate of decrease reduced with increasing the axial strain. As the axial strain exceeded 0.14, the axial stress stabilized because when the hoop strain became more pronounced, the confining pressure provided by the geogrid was sufficient to stop the decrease in axial stress. The final average axial stress were approximately 25% to 40% of the nominal maximum average axial stress. Only in the fourth branch, the different numbers of geogrid layers had a prominent influence on the mechanical behaviour. With additional geogrid layers, the axial stress was higher. This trend is similar to the trend of GPGCPs with FPCC.

3.4.2. Hoop strain distribution

The axial-hoop strain curves of geogrid tubes in GPGCPs with FPCC are shown in Fig. 17. The measurement methods are same as the measurement methods used for GPGCPs without FPCC. The shape of the curves and the development trends are same as those of GPGCPs without FPCC. The final average hoop strain of Specimens GGC2 and GGC3 decreased by 31.7% and 51.6%, respectively, relative to the final average hoop strain of Specimens GGC1. In other words, the use of additional geogrid layers can significantly reduce the hoop strain.

In addition, the axial-hoop strain relationships of FPCC placed in the middle of GPGCPs were also obtained using strain gauges attached at the mid-height of the FPCC (shown in Fig. 18). The test results of hoop strain terminate at the point where GFRP jackets rupture. The testing results of

FPCC alone are also included in Fig. 18. There were no prominent differences between these sets of results. In other words, the outer PGC and geogrid tubes have almost no influence on the mechanical behaviour of FPCC, possibly because the confinement effect provided by the outer PGC and geogrid tube is marginal.

3.5. Comparison between two groups of GPGCPs

3.5.1. Axial stress-axial strain relationships

Figure 19(a), Figure 20(a) and Figure 21(a) show the comparisons between the axial stress-axial strain curves of GPGCPs with and without FPCC with the same number of geogrid layers. In each figure, the first peak axial stress (maximum axial stress) of all GPGCPs with FPCC were approximately 20% higher than the peak axial stress of all GPGCPs without FPCC. In addition, in each figure, the final axial stress of GPGCPs with FPCC were approximately 45% higher than the final axial stress of GPGCPs without FPCC. Thus, using FPCC has the advantage of increasing the axial load carrying capacity.

3.5.2. Hoop-axial strain

The comparisons between the axial-hoop strain curves of GPGCPs with and without FPCC with the same number of geogrid layers are shown in Fig. 19(b), Fig. 20(b) and Fig. 21(b). It is found that in each figure, the hoop strains of GPGCPs without FPCC was slightly higher than those of GPGCPs with FPCC. However, the use of FPCC has no significant effect on the hoop strain.

3.5.3. Ductility

The ductility of concrete members is regarded as one of the most important design aspects. The ductility calculation method proposed by Park [51] was adopted in this study, as shown in Equation (1):

$$\mu_{\varepsilon} = \frac{\varepsilon_u}{\varepsilon_y} \quad (1)$$

where μ_{ε} is the specimen ductility. In general, ε_u is defined as the specimen strain at 85% of the maximum stress at the descending branch (for unconfined specimens) or is equal to the ultimate strain (for FRP-confined concrete, it is the strain at the point of FRP rupture). The ε_y is the yield strain, which is the axial strain at the yield stress.

In this study, the definition of ε_u proposed by Wang et al. [36] was adopted, which defines ε_u as the axial strain at 50% of the maximum stress at descending branch, which is shown in Fig. 22. This is because this definition can more clearly demonstrate the differences in ductility for the concrete confined by the geogrid.

The yield strain ε_y was the strain at the yield stress. Here, the yield stress was obtained based on the method proposed by Park [51]. The yield stress corresponds to the intersection point between a horizontal line drawn from the first peak axial stress and the straight line passing through the origin and the point representing 75% of the first peak axial stress. There were three different types of stress-strain curves were observed in this study, and the positions of yield strain are shown in Fig. 22.

The ductility results of this study are summarized in Table 8. Compared with those of plain PGCs, the ductility results for Specimens GG1, GG2 and GG3 were improved by 1.7%, 11.3% and 20.9%, respectively. In other words, the geogrid alone cannot effectively enhance the ductility of pervious concrete piles. However, when the FPCC was included, the ductility of the piles increased significantly. The ductility results for Specimens GGC1, GGC2 and GGC3 were 22 times, 25 times and 30 times the ductility results for plain PGC, respectively. In addition, with additional geogrid layers, the ductility results of both groups of GPGCPs increased.

4. Conclusions

This study presents and explains the results of axial compression tests on geogrid-confined pervious geopolymer concrete piles (GPGCPs) with and without FRP-PVC-confined concrete core (FPCC). The axial stress-axial strain behaviour, axial-hoop strain behaviour and final state of the specimens have been discussed. According to the test results and discussions presented above, the following conclusions can be drawn:

1. For GPGCPs without FPCC, in comparison with plain PGC, geogrid tubes alone cannot effectively improve the maximum axial stress and axial strain at the maximum axial stress. This is because the confinement effect provided by the geogrid is relatively low. Only after the maximum axial stress, the effects of the geogrid tubes became clear. When the number of geogrid layers increased, the axial stress increased slightly. However, the effects of geogrid on hoop strain was significant. When the number of geogrid layers increased, the hoop strain decreased significantly.
2. For the GPGCPs with FPCC, two peak axial stresses appeared, and the values of these two peak axial stresses were close. The first peak axial stress, which was the maximum axial stress, was approximately 20% higher than the maximum axial stress of GPGCPs without FPCC. The effects of geogrid on axial stress of GPGCPs with FPCC were similar to those of GPGCPs without FPCC. Only after the second peak stress the effects of geogrid tubes became clear. When the number of geogrid layers increased, the axial stresss of GPGCPs with FPCC increased.
3. The incorporation of FPCC into GPGCPs did not have a significant influence on the hoop strain of the outer geogrid tubes. The effects of geogrid on hoop strain of GPGCPs with FPCC is similar to those of GPGCPs without FPCC. When the number of geogrid layers increased, the

hoop strain of GPGCPs with FPCC decreased significantly. When the number of geogrid layers was same for GPGCPs with and without FPCC, their hoop strains were close.

4. All GPGCPs can bear a large axial strain due to the high ductility of the geogrid. In comparison with the ductility of plain PGC, the ductility of GPGCPs without FPCC was not effectively enhanced. However, when the FPCC was applied, the ductility of the GPGCPs with FPCC improved significantly.

The focus of this paper is to present the new idea of GPGCPs with FPCC, and present the results of experimental work. The future investigation of analytical and numerical models about this new type pile is ongoing at the University of Wollongong, Australia.

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List of Figures

- Fig. 1. Details of FPCC samples (units: mm; SG: strain gauge).
- Fig. 2. GPGCPs without and with FPCC: (a) GPGCPs without FPCC; (b) GPGCPs with FPCC.
- Fig. 3. Details of GPGCP specimens (units: mm; SG: strain gauge): (a) GG1, GG2, GG3; (b) GGC1, GGC2, GGC3.
- Fig. 4. Geogrid used in this study: (a) uniaxial geogrid; (b) Tensile load-axial strain curves of the geogrid coupons.
- Fig. 5. Tensile test of the GFRP coupons.
- Fig. 6. PVC coupons used in this study: (a) dog-bone PVC coupon; (b) tensile load-axial strain curves of the PVC coupons.
- Fig. 7. Axial load-axial strain curves of PGC and NGC.
- Fig. 8. FRP-PVC tubes: (a) failure mode of the FRP-PVC; (b) axial load-axial strain curves.
- Fig. 9. Failure mode of the FRP-PVC-confined concrete.
- Fig. 10. Axial stress-axial strain curves of FPCC.
- Fig. 11. Axial-hoop strain responses of FPCC.
- Fig. 12. Final state of the representative GPGCPs without FPCC: (a) GG1; (b) GG2; (c) GG3.
- Fig. 13. Final state of the representative GPGCPs with FPCC: (a) GGC1; (b) GGC2; (c) GGC3.
- Fig. 14. Axial stress-axial strain curves of GPGCPs without FPCC.
- Fig. 15. Axial-hoop strain responses of GPGCPs without FPCC.
- Fig. 16. Axial stress-axial strain curves of GPGCPs with FPCC
- Fig. 17. Axial-hoop strain responses of GPGCPs with FPCC
- Fig. 18. Axial-hoop strain responses of FPCC placed in GPGCPs.
- Fig. 19. Comparisons between GG1 and GGC1.
- Fig. 20. Comparisons between GG2 and GGC2.
- Fig. 21. Comparisons between GG3 and GGC3.

591 Fig. 22. Definition of ε_u and ε_y : (a) Definition of ε_u and ε_y for unconfined concrete; (b) Definition
592 of ε_u and ε_y for GPGCPs without FPCC; (c) Definition of ε_u and ε_y for GPGCPs with FPCC
593

Fig. 1. Details of FPCC samples (units: mm; SG: strain gauge).

(a)

(b)

Fig. 2. GPGCPs without and with FPCC: (a) GPGCPs without FPCC; (b) GPGCPs with FPCC.

Fig. 3. Details of GPGCP specimens (units: mm; SG: strain gauge): (a) GG1, GG2, GG3; (b) GGC1, GGC2, GGC3.

(a)

(b)

Fig. 4. Geogrid used in this study: (a) Uniaxial geogrid; (b) Tensile load-axial strain curves of the geogrid coupons.

Fig. 5. Tensile test of the GFRP coupons.

(a)

(b)

Fig. 6. PVC coupons used in this study: (a) Dog-bone PVC coupon; (b) Tensile load-axial strain curves of the PVC coupons.

Fig. 7. Axial load-axial strain curves of PGC and NGC.

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(b)

Fig. 8. FRP-PVC tubes: (a) Failure mode of the FRP-PVC hollow tube; (b) Axial load-axial strain curves.

628

629

630

Fig. 9. Failure mode of the FRP-PVC-confined concrete.

Fig. 10. Axial stress-axial strain curves of the FPCCs.

Fig. 11. Axial-hoop strain responses of the FPCCs.

Fig. 12. Final state of the representative GPGCPs without FPCC: (a) GG1-1;
(b) GG2-1; (c) GG3-2.

Fig. 13. Final state of the representative GPGCPs with FPCC: (a) GGC1-2; (b) GGC2-2; (c) GGC3-1.

Fig. 14. Axial stress-axial strain curves of GPGCPs without FPCC.

Fig. 15. Axial-hoop strain responses of GPGCPs without FPCC.

Fig. 16. Axial stress-axial strain curves of GPGCPs with FPCC.

Fig. 17. Axial-hoop strain responses of GPGCPs with FPCC.

Fig. 18. Axial-hoop strain responses of FPCC placed in GPGCPs.

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Fig. 19. Comparisons between GG1 and GGC1.

Fig. 20. Comparisons between GG2 and GGC2.

Fig. 21. Comparisons between GG3 and GGC3.

Fig. 22. Definition of ϵ_u and ϵ_y : (a) Definition of ϵ_u and ϵ_y for unconfined concrete; (b) Definition of ϵ_u and ϵ_y for GPGCPs without FPCC; (c) Definition of ϵ_u and ϵ_y for GPGCPs with FPCC.

687 **List of Tables**

688 Table 1. Details of the preliminary test samples.

689 Table 2. Details of the GPGCP specimens.

690 Table 3. Mix proportions of PGC and NGC.

691 Table 4. Average material properties of GFRP sheets and PVC dog-bone specimens.

692 Table 5. Results of the preliminary tests.

693 Table 6. Test results of GPGCPs without FPCC.

694 Table 7. Test results of GPGCPs with FPCC.

695 Table 8. Ductility results.

696

697

Table 1. Details of the preliminary test samples.

Label	Specimen Type	Outer Diameter (mm)	Inner Diameter (mm)
P-1	Plain PGC	150	-
P-2		150	-
N-1	Plain NGC	150	-
N-2		150	-
FP-1	FRP-PVC tubes	86	76
FP-2		86	76
FPCC-1	FRP-PVC-confined concrete (FPCC)	86	-
FPCC-2		86	-

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PGC: Pervious Geopolymer Concrete.
NGC: Normal Geopolymer Concrete.

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Table 2. Details of the GPGCP specimens.

Label	Specimen Type	Diameter of the PGC (or NGC for plain NGC samples)	Number of Geogrid Layers	FPCC
GG1-1	GPGCPs without FPCC	160	1	-
GG1-2		160	1	-
GG2-1		160	2	-
GG2-2		160	2	-
GG3-1		160	3	-
GG3-2		160	3	-
GGC1-1	GPGCPs with FPCC	160	1	1
GGC1-2		160	1	1
GGC2-1		160	2	1
GGC2-2		160	2	1
GGC3-1		160	3	1
GGC3-2		160	3	1

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GPGCP: Geogrid-confined Pervious Geopolymer Concrete Pile.
 FPCC: FRP-PVC-confined Concrete.

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Table 3. Mix proportions of PGC and NGC.

Mix	Coarse Aggregates (kg/m ³)	Sand (kg/m ³)	Slag (kg/m ³)	Fly Ash (kg/m ³)	Sodium Hydroxide Solution (kg/m ³)	Sodium Silicate Solution (kg/m ³)	Additional Water (kg/m ³)
PGC	1558	-	122.0	183.0	50.8	101.7	45.8
NGC	1039	675	160.0	240.0	66.7	133.3	60.0

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Table 4. Average material properties of GFRP sheets and PVC dog-bone specimens.

Type	Thickness (mm/ply)	Tensile Strength (MPa)	Fracture Stress (MPa)	Elastic Modulus (GPa)	Ultimate Tensile Strain (%)
GFRP	0.15	621.5	-	33.4	1.86
PVC specimen	4	51.2	42.7	1.58	55.4

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Table 5. Results of the preliminary tests.

Specimen Label	Maximum Load (kN)	Maximum Axial stress (MPa)	Axial Strain at Maximum Load	Hoop Strain at FRP Rupture
P-1	373	21.1	0.0021	-
P-2	388	22.0	0.0019	-
N-1	871	49.3	0.0023	-
N-2	836	47.3	0.0021	-
FP-1	45.1	-	0.0232	-
FP-2	44.9	-	0.0215	-
FPCC-1	457	78.2	0.0264	-0.0180
FPCC-2	437	75.2	0.0255	-0.0183

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Table 6. Test results of GPGCPs without FPCC.

Specimen Label	Peak Axial Stress (MPa)	Axial Strain at Peak Stress	Final Axial Stress (MPa)	Final Lateral Strain	Final Axial Strain
GG1-1	21.8	0.0021	4.8	-0.083	0.113
GG1-2	20.9	0.0019	4.0	-0.072	0.096
GG2-1	21.9	0.0020	5.8	-0.064	0.154
GG2-2	22.0	0.0021	5.7	-0.059	0.154
GG3-1	22.1	0.0022	6.7	-0.041	0.154
GG3-2	22.6	0.0021	7.2	-0.045	0.154

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Table 7. Test results of GPGCPs with FPCC

Specimen Label	First Peak Stress (MPa)	Axial Strain at First Peak Stress	Second Peak Axial Stress (MPa)	Axial Strain at Second Peak Stress	Final Axial stress (MPa)	Final Lateral Strain	Final Axial Strain
GGC1-1	26.2	0.0021	25.2	0.0237	7.1	-0.080	0.154
GGC1-2	26.5	0.0022	24.6	0.0249	6.3	-0.081	0.154
GGC2-1	26.6	0.0024	26.2	0.0238	8.0	-0.054	0.154
GGC2-2	26.2	0.0026	25.7	0.0254	8.9	-0.056	0.154
GGC3-1	26.4	0.0021	26.2	0.0243	9.2	-0.038	0.154
GGC3-2	25.9	0.0022	25.7	0.0256	10.0	-0.040	0.154

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Table 8. Ductility results.

Specimen Label	Yield Strain (ϵ_y)	ϵ_u	Ductility	Average Ductility
P-1	0.00193	0.0035	1.81	1.77
P-2	0.00192	0.0033	1.72	
GG1-1	0.00200	0.0037	1.85	1.80
GG1-2	0.00189	0.0033	1.75	
GG2-1	0.00194	0.0039	2.01	1.97
GG2-2	0.00202	0.0039	1.93	
GG3-1	0.00199	0.0041	2.06	2.14
GG3-2	0.00198	0.0044	2.22	
GGC1-1	0.00126	0.0479	38.0	38.2
GGC1-2	0.00128	0.0491	38.4	
GGC2-1	0.00134	0.0596	44.5	44.9
GGC2-2	0.00132	0.0597	45.2	
GGC3-1	0.00136	0.0649	47.7	53.6
GGC3-2	0.00132	0.0784	59.4	

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