

Experimental Study on the Buckling Behaviour of Reinforcing Bars in FRP-confined RC Columns

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SUMMARY:

Fibre-reinforced polymer (FRP) confining jackets offer an attractive solution for the seismic retrofit of reinforced concrete (RC) columns. For the accurate prediction of the strength and ductility of FRP-confined RC columns, it is necessary to understand the interaction between the FRP jacket and the RC column at all deformation levels under seismic loading. This paper presents an experimental study into the influence of FRP confinement on the buckling behaviour of longitudinal steel reinforcing bars in RC columns under monotonic axial compression. Test results of six FRP-confined RC columns and six FRP-confined plain concrete columns (control specimens) are presented with particular attention to the evolution of strains in the longitudinal steel reinforcing bars and the FRP jacket as the load increases. The test results clearly indicate that significant interaction exists between the FRP confinement and the buckling of longitudinal steel bars, and that this interaction should be carefully considered when formulating constitutive laws for both the reinforcing bars and the FRP-confined concrete in theoretical modelling. The work presented in this paper represents the first attempt at understanding the buckling behaviour of longitudinal steel bars in plastic hinge zones of FRP-confined RC columns under seismic loading.

Keywords: FRP, Confinement, RC columns, Buckling, Interaction

1. INTRODUCTION

Fibre-reinforced polymer (FRP) confining jackets offer an attractive solution for the seismic retrofit of reinforced concrete (RC) columns. The FRP jacket has two important roles to play in the seismic retrofit of RC columns. The first role is to enhance the strength and ductility of concrete. As has been demonstrated by extensive existing work, both the strength and ductility of concrete can be significantly improved by a sufficiently stiff FRP jacket (Teng et al. 2002; Hollaway and Teng 2008). The second role is to reduce or eliminate the possibility of buckling of longitudinal steel reinforcing bars (Ilki et al. 2006; Teng et al. 2002; Hollaway and Teng 2008); such buckling of steel bars has been frequently observed in column tests and may lead to the ultimate failure of conventional RC columns.

Compared to the large amount of research work available on the behaviour of FRP-confined plain concrete columns (e.g. Lam and Teng 2003; Jiang and Teng 2007; Dai et al. 2011), existing work on the use of FRP jackets to prevent the buckling of steel reinforcing bars is very limited (Tastani et al. 2006; Bournas et al. 2007; Sato and Ko 2007; Sato and Ko 2008; Pellegrino and Modena 2010; Bournas and Triantafillou 2011) although this has been an important issue in un-strengthened RC columns and has been extensively studied from various perspectives (e.g. Bae et al. 2005; Bresler and Gilbert 1961; Mander et al. 1988; Mau and Elmabsout 1989; Mau 1990; Dodd and Restrepo 1995; Gomes and Appleton 1997; Pantazopoulou 1998; Bayrak and Sheikh 2001; Dhakal and Maekawa 2002a, b and c; Berry and Eberhard 2005; Brown et al. 2007; Zong and Kunnath 2008; Kunnath and Mohle 2009; Urmson and Mander 2012).

In FRP-confined RC columns, the confined concrete provides much stronger support to the steel reinforcing bars than in a conventional RC column where the concrete can easily spall off. Therefore, the buckling of steel reinforcing bars in an FRP-confined RC column is generally postponed to a

higher strain level although its occurrence may not be entirely eliminated by the FRP jacket (Tastani et al. 2006; Bournas and Triantafillou 2011). In the meantime, the growth of the lateral buckling (more precisely post-buckling) deformation of steel reinforcing bars may lead to additional strains in the FRP jacket, causing its premature rupture (Tastani and Pantazopoulou 2004). This interaction between the steel reinforcing bars and the FRP jacket through the concrete is an important mechanism governing the behaviour of FRP-confined RC columns.

To the best knowledge of the authors, no theoretical model has been developed to predict the interaction between the inelastic buckling of steel reinforcing bars and the confining action of the FRP jacket through the concrete between them. For the accurate prediction of the strength and ductility of FRP-confined RC columns under seismic loading, it is necessary to understand this interaction at all deformation levels. In order to achieve an in-depth understanding of this interaction mechanism in FRP-confined RC columns, systematic research including both laboratory tests and theoretical modelling is being conducted at The Hong Kong Polytechnic University. This paper presents the results of an experimental study on the buckling of steel reinforcing bars in FRP-confined RC columns under axial compression

2. EXPERIMENTAL PROGRAM

2.1. Test Specimens and Parameters

In this study, six FRP-confined RC columns and six FRP-confined plain concrete columns were prepared and tested under concentric axial compression. All the column specimens had the same dimensions (i.e. 200 mm in diameter and 500 mm in height). Due to the limited capacity of the concrete mixer, these 12 specimens were prepared as three separate batches. Each batch consisted of 2 FRP-confined RC columns, 2 FRP-confined plain concrete columns as control specimens, and 4 standard plain concrete cylinders (152 mm in diameter and 305 mm in height) for determining the compressive strength of concrete. The test parameters included the type of FRP [i.e. Polyethylene Terephthalate (PET) FRP and Carbon FRP(CFRP)] and the number of plies (layers) in the FRP jacket (i.e. 1 and 2 plies). Carbon and PET FRPs were chosen for comparison because they possess significantly different material properties. CFRP has a high strength, a high elastic modulus but a relatively low rupture strain (i.e. usually less than 1.5%). PET FRP is a new material with a large rupture strain (LRS) (usually larger than 5%) but a relatively low elastic modulus. PET FRP, being a recycled material, offers a more ductile, cost-effective and environmentally friendly seismic retrofit solution (Anggawidjaja et al. 2006; Dai et al. 2011; Dai and Ueda 2012). The typical stress-strain curves of PET FRP and CFRP are illustrated in Fig. 1a for comparison. Table 1 summarizes the key information of all the specimens. For each specimen configuration, two identical specimens were prepared to understand experimental scatters.

Each RC column had 4 ϕ 20 ribbed longitudinal steel reinforcing bars, leading to a longitudinal reinforcement ratio (ρ_l) of around 0.04. This high reinforcement ratio should not be an issue of concern as these column specimens were designed to allow steel bar buckling to be effectively studied rather than to achieve typical RC columns. Indeed, by using such a high reinforcement ratio, it was hoped that the contribution of steel reinforcing bars to the overall load-carrying capacity could be more accurately isolated from the test results. The longitudinal steel bars were extended to both the top and bottom ends of each RC column to ensure that the four longitudinal bars would directly and simultaneously receive loading from the beginning of the load process. The tensile stress-strain curve of the longitudinal steel reinforcing bars is shown in Fig. 1b. Smooth round bars of ϕ 10 mm in diameter were used as the transverse steel reinforcement (steel ties), and their concrete cover depth was 20 mm. Table 2 presents the mechanical properties of the longitudinal and transverse steel reinforcing bars.

It has already been established by existing research that if the unsupported length of longitudinal steel bars (between ties) exceeds 6D-8D, where D is the diameter of longitudinal steel bars, buckling of

longitudinal steel bars is likely to occur (Mau 1990; Priestley et al. 1996; Pantazopoulou 1998; Tastani et al. 2006;). In this study, a tie spacing of 20D (i.e. 400 mm) was used; this spacing, chosen after trial tests on three specimens with a tie spacing of 300 mm, is generally a little larger than is frequently met even in old structures. Nevertheless, such a large tie spacing can be encountered in old structures (Bournas and Triantafillou 2011) and was adopted to ensure that buckling of longitudinal steel bars would likely occur even in columns with reasonably strong FRP confinement.

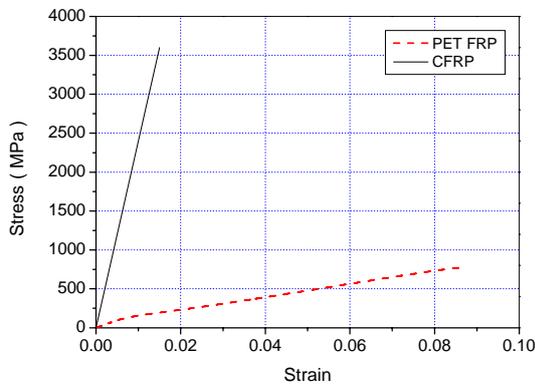
Table 1. Test Specimens and Summary of Key Test Results

Batch No.	Specimen name	Concrete strength (MPa)	Type of FRP	Number of plies of FRP	Nominal thickness of FRP (mm)	Rupture strain of FRP (%)	Ultimate axial deformation (mm)	Ultimate axial load (kN)
1	CFRP-RC-1-a ^a	38.0	CFRP	1	0.167	0.549	1.57	1866
	CFRP-RC-1-b					0.880	1.96	2006
	CFRP-PC-1-a					0.764	1.82	1452
	CFRP-PC-1-b					0.840	2.07	1384
2	PET-RC-1-a	37.2	PET-600	1	0.841	7.135	8.01	1675
	PET-RC-1-b					6.436	8.25	1621
	PET-PC-1-a					5.692	7.45	1065
	PET-PC-1-b					7.651	8.01	1169
3	PET-RC-2-a	35.0	PET-600	2	1.682	7.668	12.90	2353
	PET-RC-2-b					6.878	12.14	2335
	PET-PC-2-a					8.086	13.30	1776
	PET-PC-2-b					8.225	14.42	1824

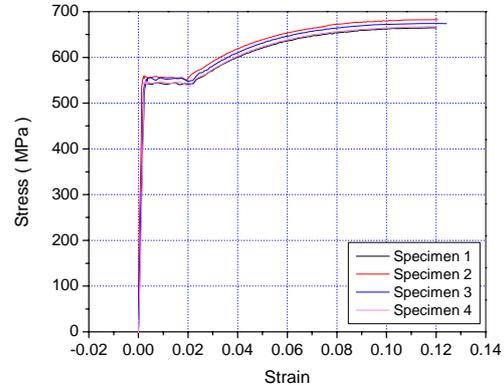
a: W-X-Y-Z: W = type of FRP; X = RC for reinforced concrete and PC for plain concrete; Y = number of plies; Z = specimen index.

Fig. 2 shows a photo of the steel cage; it is seen that two steel ties were used near each end (at 20 mm from the end) to restrain the lateral movement of the longitudinal steel bars, to avoid the yielding of the steel ties, and to provide a well-confined zone for the dispersion of axial loading into the middle region of the column. To guide the longitudinal steel bars to develop the largest lateral buckling deformation at the mid-height of the specimens, approximately 15% of the steel bar cross-section was removed over a mid-height zone of 30 mm.

The FRP jackets of all column specimens were formed via the wet layup process where fiber sheets were impregnated and wrapped around the column with an epoxy resin. An overlapping length of 300 mm (nearly half the perimeter of the cross-section) was used in forming all the FRP jackets to ensure sufficient end anchorage. At each end of the column, a single-ply CFRP strip with a width of 80 mm and a nominal ply thickness of 0.34 mm was used to strengthen the column ends in order to avoid local column failure due to uneven compression. The FRP jackets were left for curing for at one week before the column was tested under axial compression.



(a) Tensile stress-strain curves of FRP composites



(b) Tensile stress-strain curves of longitudinal bars

Figure 1. Tensile stress-strain curves

Table 2. Mechanical Properties of Steel Reinforcing Bars

Bar type	Nominal d_b (mm)	d_b^* (mm)	f_y (MPa)	ϵ_y (%)	$f_{s,max}$ (MPa)	$\epsilon_{s,max}$ (%) ^{note}
Φ10	9.66	-	350.0	0.17	500.0	20.0
Φ20	19.13	18.50	540.0	0.24	682.0	12.0

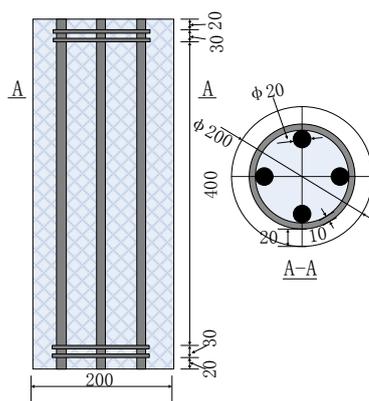
d_b = bar diameter; f_y = yield strength; ϵ_y = yield strain; $f_{s,max}$ = tensile strength; $\epsilon_{s,max}$ = strain corresponding to $f_{s,max}$; d_b^* = diameter of longitudinal steel reinforcing bars at reduced section.

Note: these recorded values are expected to be slightly smaller than the actual ones as the extensometer for measuring elongations of steel reinforcing bars was removed before final rupture.

2.2. Test Setup and Instrumentation

In order to measure the development of strains in longitudinal steel bars during the entire loading process, 5 pairs of strain gauges with a 5 mm gauge length were installed along each longitudinal steel bar at a uniform interval of 75 mm. The two gauges of a pair were placed at 180° apart around the bar cross-section (i.e. in the radial direction passing through the bar centre) to monitor its inner and outer strains to evaluate its curvature. On the steel tie nearest each of the two column ends, two strain gauges with a gauge length of 5 mm were installed on the outer surface (Figs. 2 and 3) to monitor whether they would reach yielding during the test. In addition, four hoop strain gauges with a gauge length of 20 mm were evenly distributed (i.e. at 90° intervals) on the FRP jacket at the mid-height of each column: one at the centre of the FRP overlapping zone and the other three outside the overlapping zone. Two additional sets of four evenly distributed hoop strain gauges each were installed at 75 mm below and above the mid-height section to capture hoop strain distributions of the FRP jacket away from the mid-height (Fig. 3). Moreover, four axial strain gauges with a gauge length of 20 mm were installed at the mid-height section and adjacent to the hoop strain gauges to measure axial strains at mid-height. The axial deformations of all the test columns were also measured using four linear variable displacement transducers (LVDTs) that were evenly distributed around the circumference and covered the mid-height region of 205 mm (Figs. 3 and 4a).

Prior to testing, all the specimens were capped with high strength gypsum at both ends to ensure uniform loading from the machine. All the compression tests were performed using a 5000 kN capacity servo-controlled MTS machine with displacement control at a constant rate of 0.4 mm/min. The specimens were monotonically loaded to failure (usually the rupture of the FRP jacket). During the test, all load, displacement and strain readings were automatically recorded using a data logger and stored in a computer.



(a) Dimensions

(b) Steel cage

(c) Steel cage with strain gauges

Figure 2. Specimen dimensions and reinforcement details.

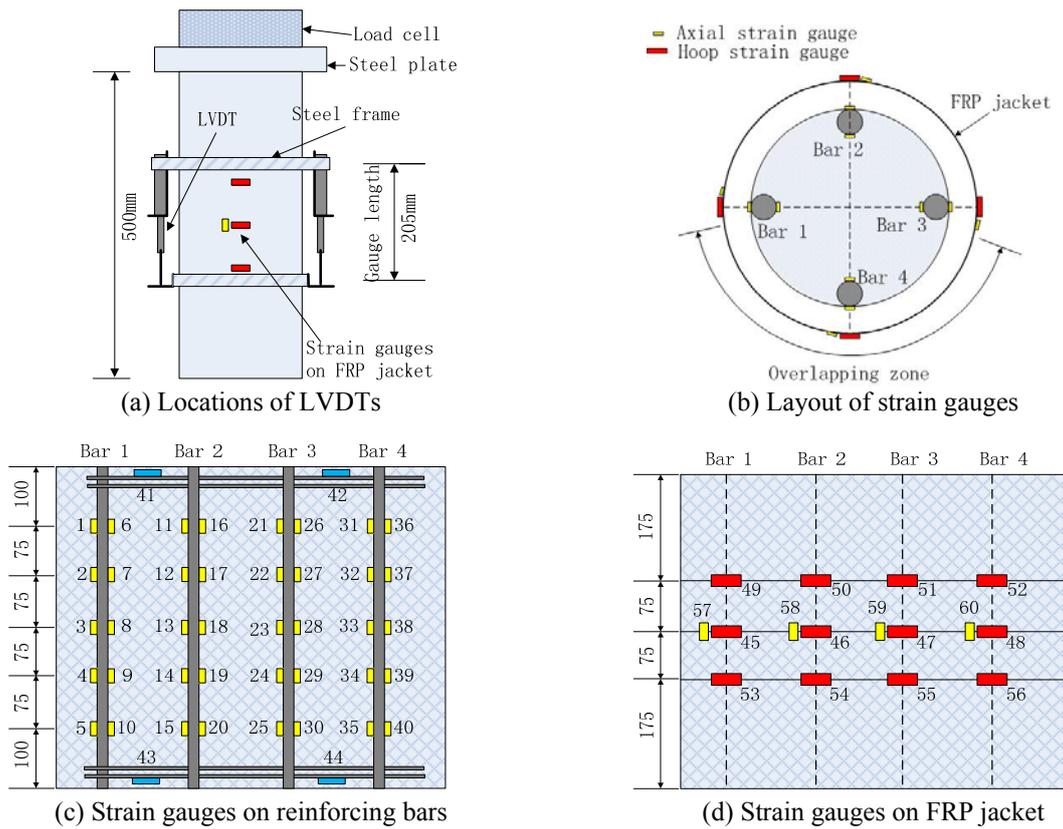


Figure 3. Test setup and instrumentation details.



Figure 4. Test setup and failure mode

3. TEST RESULTS

3.1. Failure Mode

Typical failures of the test specimens are shown in Figs. 4b to 4d. Both the FRP-confined reinforced and FRP-confined plain concrete columns failed in their mid-height region due to the hoop tensile rupture of the FRP jacket. The circumferential location of rupture was outside the overlapping zone. An explosive noise accompanied the failure of all the CFRP-confined specimens. The noise at the rupture of the FRP jacket in LRS FRP-confined columns was not as loud as that of CFRP-confined columns. The different amounts of axial shortening of a CFRP-confined column and a LRS

FRP-confined column at the ultimate state can be clearly seen in Fig. 4d. The specimen shown on the left of Fig. 4d was confined by a 1-ply CFRP jacket while the specimen on the right was confined with a 2-ply PET FRP jacket; these two FRP jackets sheets had similar confining stiffnesses (i.e. similar Et values, where E = hoop elastic modulus and t = nominal thickness of the FRP jacket). Obviously the shortening of the LRS FRP-confined column is much larger than that of the CFRP-confined column, indicating the much larger deformability of the former, as has been pointed out by previous researchers (Dai et al. 2011; Dai and Ueda 2012). After removing the external FRP jacket and the cover concrete, the deformed shape of the longitudinal steel bars was exposed (Fig. 4d). Apparently the longitudinal steel bars in the columns confined by LRS FRP buckled before FRP rupture and significant outward deformation had developed. By contrast, the lateral buckling deformation of longitudinal steel bars in CFRP-confined RC columns was invisible to the naked eye.

3.2 Load-Displacement Curves

The axial load-axial displacement curves of all the test specimens are shown in Figs. 5a to 5c for cases with a 1-ply CFRP jacket, with a 1-ply PET FRP jacket and with a 2-ply PET FRP jacket respectively. In these figures, the load shown is that from the load cell and the displacement is the averaging value from the four LVDTs. Figs. 5a to 5c show that all the curves have a similar bi-linear shape, with the slope of the second stage being dependent on the stiffness of the FRP jacket. In the second stage, the CFRP-confined RC and the plain concrete columns have nearly parallel stress-strain curves (Fig. 5a). This phenomenon indicates that the load carried by the longitudinal steel bars in the second stage of deformation varies little until the rupture of the CFRP jacket, which implies that the longitudinal steel bars had yielded in the second stage but did not suffer from much resistance degradation due to the development of lateral buckling deformations. That is, even though the tie spacing in the test zone and the diameter of longitudinal bars were chosen to allow the buckling of longitudinal bars to occur, the lateral buckling deformation, if any, were not significant enough at the deformation level when the CFRP jacket ruptured.

For the two RC columns confined with a 1-ply PET FRP jacket, the axial load-displacement curves have a nearly horizontal (in fact slightly descending) second portion; the axial load-displacement curves of the plain concrete columns confined with the same PET FRP jacket have a second portion that is initially slightly descending and then slightly ascending. The nearly horizontal shape for the second stage of the axial load-displacement curve of these columns is attributed to the relatively small confinement stiffness of the FRP jacket (Teng et al. 2009). However, the narrowing down of the difference between the load resisted by a PET FRP-confined plain column and the corresponding RC column as the axial deformation increases indicates a decrease in the contribution of longitudinal bars to the overall axial load carried by the column due to the progressive development of the buckling deformation of the longitudinal bars. A similar observation can be made about Fig. 5c.

Another phenomenon can also be observed in Fig. 5: the deformation capacities of FRP-confined RC columns and their plain concrete counterparts are nearly the same in Figs. 5a and 5b, however, in Fig. 5c, the PET FRP-confined RC columns have a smaller ultimate axial displacement than that of the FRP-confined plain concrete columns although in both cases the ultimate axial displacement is large (i.e. about 15 mm). This difference implies that the FRP jacket is less effective in the former case, which is consistent with Pellegrino and Modena's (2010) suggestion. This phenomenon can be explained by the buckling of longitudinal bars at large axial deformation levels as the buckling deformation may lead to premature rupture of the FRP jacket as discussed in the next section.

3.3 Effect of Buckling of Longitudinal Bars on Jacket Rupture

Table 1 presents the rupture strains of the external FRP jackets in all tested specimens. It should be mentioned that although special attention had been paid to minimize any possible eccentricity during the tests, the experimental strain values still exhibited a small extent of scatter. The existence of longitudinal steel bars was another factor for the scatter since one of the two longitudinal bars at the opposite sides of the columns buckled first during the loading process and led to early FRP rupture

around that location.

It can be seen in Table 1 that the average hoop rupture strain of CFRP jackets is about 0.75% for both plain concrete and RC columns while the average hoop rupture strain of PET FRP jackets is about 6.73% in both plain concrete and RC columns confined with a 1-ply PET FRP jacket. However, for the specimens with a 2-ply PET FRP jacket, the average hoop rupture strain of PET FRP jackets for the plain concrete columns is 8.16% while that for the RC columns 7.27%. That is, the longitudinal steel bars did not have a negative effect on the hoop rupture strain of the FRP jacket at a relatively small deformation level. However, when the axial deformation becomes larger, the rupture strain of the FRP jacket is smaller in RC columns than that in plain concrete columns due to the buckling of longitudinal bars. In other words, the buckling deformation of longitudinal bars led to locally elevated hoop strains in the FRP jacket and its premature rupture at a lower average hoop strain (average strain from the three strain gages). This effect has never been isolated and addressed in existing models for FRP-confined concrete in RC columns.

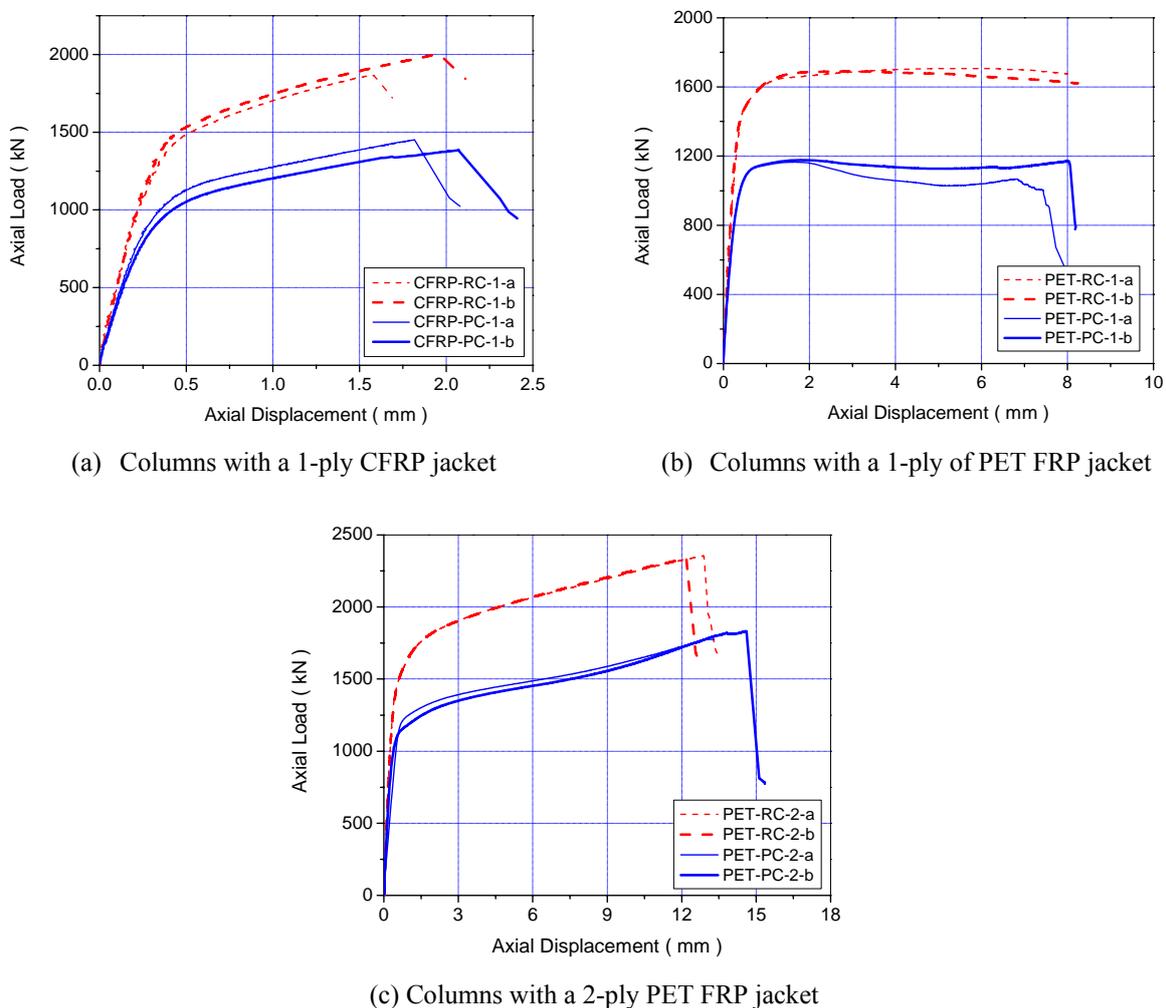
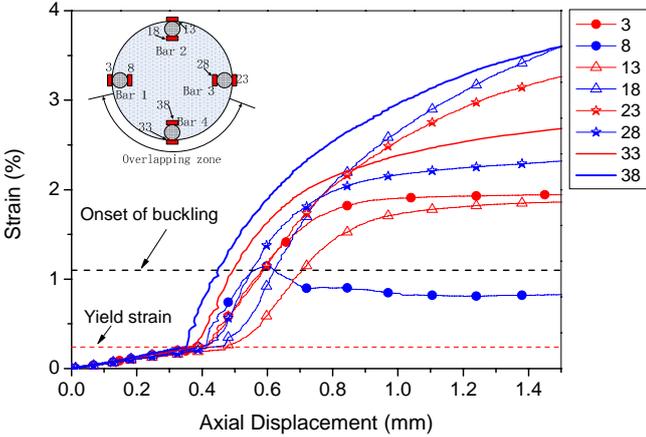


Figure 5. Load-displacement curves

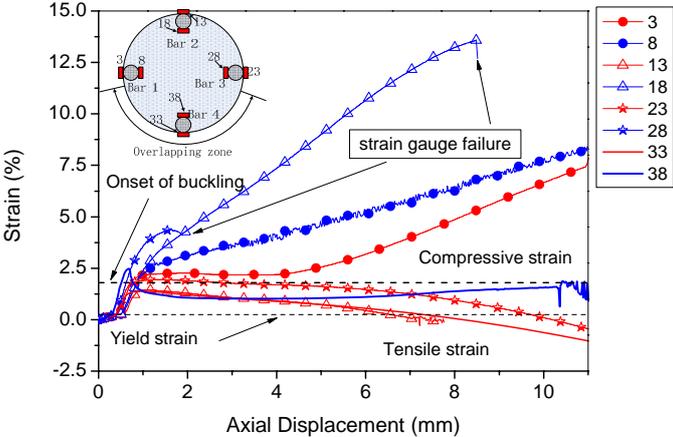
3.4 Strains in Longitudinal Steel Bars

A large number of strain gauges were employed in this study to monitor the evolution of strains in longitudinal steel bars during the entire loading process. Figs. 6a and 6b present the development of strains in longitudinal steel bars in RC columns confined with a 1-ply CFRP jacket and a 2-ply PET FRP jacket respectively for comparison purposes. A positive strain value in these figures indicates a

compressive strain. It should be noted that none of the steel ties (Fig. 2b) yielded based on strain gauge readings, implying that the lateral movements of longitudinal bars in the column end regions were negligible. It is clearly seen in Figs. 6a and 6b that the strains of longitudinal bars experienced two significant transition stages around the yield strain of the steel bars and around the critical strain at which the buckling of steel bars occurred. Here a longitudinal steel bar is assumed to have buckled when a significant strain difference in any pair of strain gauges on the two opposite sides of the bar was observed. As can be seen from Fig. 6a, although buckling deformations of the longitudinal steel bars in CFRP-confined RC columns were invisible to the naked eye as mentioned earlier, the strain readings revealed the onset of buckling of a longitudinal bar in a CFRP confined-RC column (refer to No. 3 and No. 8 strain gauges in Fig. 6a). For the RC columns confined with a 2-ply PET FRP jacket, significant strain differences can be observed in all strain gauge pairs on the longitudinal steel bars at the mid-height section of the column (Fig. 6b). It should be mentioned that one of the three longitudinal steel bars outside the FRP overlapping zone (Nos.1, 2 and 3 bars in Fig. 3d) always buckled first and the other bars then buckled subsequently. In the post-buckling stage, the strain of the more compressed side of the steel bars (Nos. 8, 18, 28 and 38 strain gauges in Fig. 6b) increases and that on the less compressed side (or tension side) (Nos. 3, 13, 23 and 33 strain gauges in Fig. 6b) decreases. At a sufficiently high axial deformation level, the strain on the less compressed side becomes negative (i.e. tensile) (Fig. 5b), indicating the development of a large curvature there and a significant loss of the axial resistance of the longitudinal steel bar.



(a) 1-ply CFRP-confined RC column (Specimen: CFRP-RC-1-b)



(b) 2-ply PET FRP-confined RC column (Specimen: PET-RC-2-a)

Figure 6. Evolution of strains in longitudinal steel bars

4. CONCLUSIONS

This paper has presented and interpreted the results from a series of monotonic axial compression tests on FRP-confined reinforced concrete (RC) and plain concrete columns to examine the buckling behaviour of longitudinal steel reinforcing bars in FRP-confined RC columns. Based on the experimental results and the associated discussions, the following conclusions can be made:

- (1) In all the test columns, the buckling of longitudinal steel reinforcing bars occurred before the rupture of the FRP jacket.
- (2) When the FRP jacket stiffness was kept the same, the buckling of longitudinal steel reinforcing bars was a more significant event in RC columns confined with a large-rupture-strain (LRS) FRP jacket than in RC columns confined a CFRP jacket.
- (3) A higher stiffness of the FRP jacket provides more effective confinement to the column and is thus more capable of restraining the steel bars against buckling. However, a higher jacket stiffness also leads to a higher axial deformation level of the column before jacket rupture; at such high deformation levels, the buckling deformation of the longitudinal steel reinforcing bars may cause the premature rupture failure of the FRP jacket, which needs to be properly considered in defining the ultimate state of the FRP jacket.
- (4) To closely predict the behaviour of FRP-confined RC columns, a reliable analytical model needs to be developed for the buckling behaviour of longitudinal steel reinforcing bars in FRP-confined concrete; in the meantime, such a model needs to consider the detrimental effect of the buckling process on the ultimate state of the FRP jacket.

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