

# **Effect of UFC Segments for Enhancing the Seismic Performance of Bridge Columns**

**K. Kawashima, S. Ichikawa**  
*Tokyo Institute of Technology, Japan*

**Mohamed ElGawady**  
*University of South Australia, Australia*

**T. Sasaki**  
*National Research Institute for Earth Science and Disaster Prevention, Japan*

**H. Matsuzaki**  
*Tokyo Institute of Technology, Japan*

**S Yamanobe**  
*Kajima Research Institute, Japan*



## **SUMMARY:**

This paper presents a bilateral cyclic loading experimental investigation of bridge columns having plastic hinges consisted of ultra-high performance steel fibre concrete (UFC). Two columns having different plastic hinge details were investigated. The first column, RC-UFC, consisted of two regions: a top region of 330 mm high constructed using normal strength concrete and a potential plastic hinge region 600 mm high constructed using lightly reinforced normal strength concrete encased in hollow-core UFC jacket. The second column, PC-UPC, constructed using unbonded post-tensioning and has geometry similar to column RC-UPC. However, the potential plastic hinge region was constructed using hollow-core UFC segments without the normal strength concrete core. Both columns were designed to have approximately the same nominal strength. The experimental tests showed that RC-UFC column was able to carry the applied axial load until a drift of 6% whereas the PC-UFC column was able to carry the applied axial load until a drift of 3.5%.

*Keywords: Segmental columns, bridges, high strength concrete, unbonded post-tensioning, self-centring*

## **1. INTRODUCTION**

Ultra high performance steel fibre reinforced concrete (UFC) represents a new class of cementitious material consisting of dense matrix and micro steel fiber. The excellent mechanical properties of UFC make it an ideal candidate for construction of bridge piers in regions of high seismic activities. UFC has compressive strength in the order of 200 MPa, a relatively high tensile strength in the order of 12 MPa, and strain hardening under tensile loading (Rossi 2002).

The manufacture cost of UFC is expensive to be used in the construction of a full bridge pier. However, it can be strategically used in the construction of critical regions such as potential plastic hinges to improve the strength and deformability of earthquake resistant piers.

Yamanobe et al. (2008) investigated the use of UFC jacket, having a wall thickness of approximately 20 mm, encased normal strength concrete in the potential plastic hinge regions of bridge piers. The UFC jacket consisted of eight segments, each having a height of 40 mm, while the normal strength core concrete encased in the UFC jacket was a continues element across the whole height of the pier. Two different details were used for the connection between the segments and the top normal strength concrete. The first detail did not use any vertical reinforcement within the segments and relied on the provided vertical reinforcement within the normal strength concrete core for flexural resistance. The second detail used 4 unbonded rebar as vertical reinforcement inside the UFC segments and extended inside the normal strength concrete atop the UFC jacket.

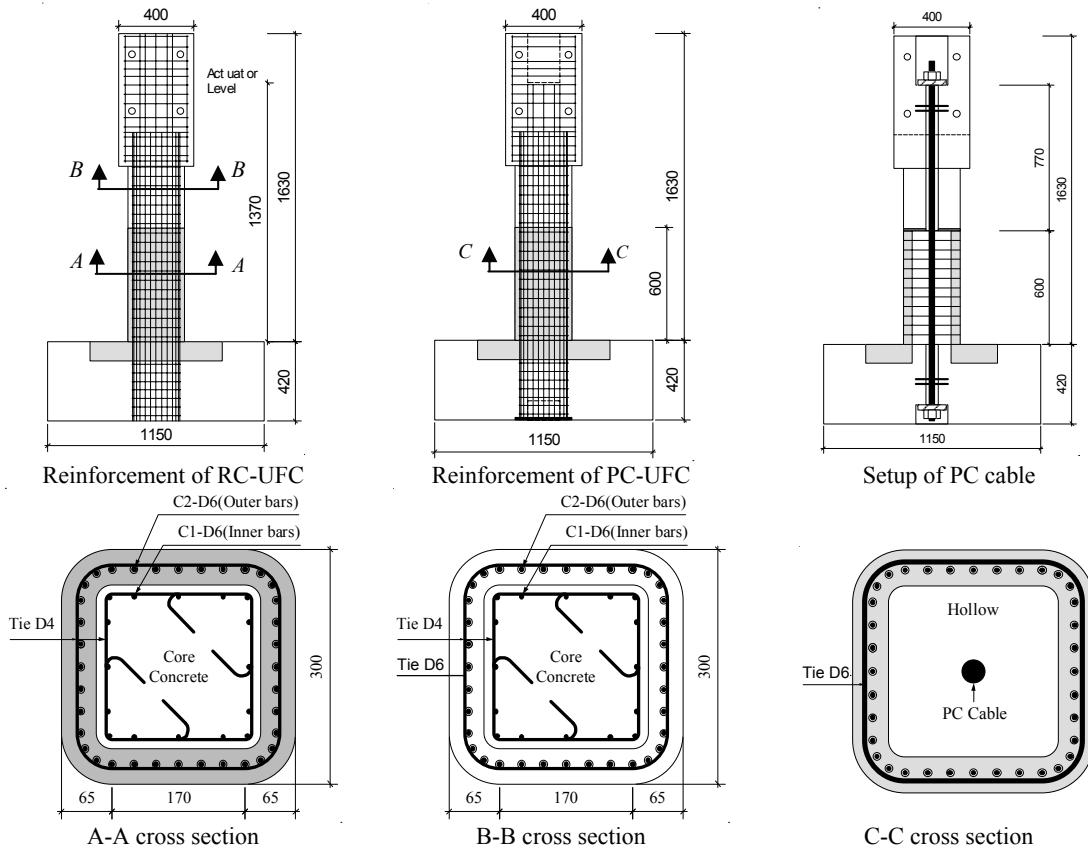
Recently, a resilient segmental precast post-tensioned pier system consisting of precast segments stacked over each other and connected by unbonded post-tensioning bars was developed (Chang et al. 2002, Hewes and Priestley 2002, Chou and Chen 2006, Marriott et al. 2009, Yamashita and Sanders 2009, ElGawady et al. 2010, ElGawady and Sha'lan 2011, ElGawady and Dawood 2012, and Dawood et al. 2012). The segments were reinforced with longitudinal rebar and post-tensioning bars or post-tensioning bars only, and confined using rebar ties, fibre reinforced polymer (FRP) tubes, and/or steel tubes. The segmental piers showed high self-centering capabilities compared to conventional reinforced concrete (RC) piers.

This manuscript presents the seismic behavior of two piers using UFC segments in the potential plastic hinge regions. The piers were subjected to constant axial load and bi-lateral loading with increasing levels of lateral displacements.

## 2. EXPERIMENTAL INVESTIGATION

### 2.1. Overview of the test specimens

This study investigates the cyclic behaviour of two different bridge piers having identical dimensions but different construction details (Fig.1). Each pier has a 300 mm square cross section and the corner of the cross section was rounded with a radius of 65 mm to reduce stresses concentration.



**Figure 1.** Configuration and dimensions of RC-UFC and PC-UFC columns (unit: mm)

The first pier, hereinafter referred to as RC-UFC, consisted of two regions: a top region of 330 mm high having a cross section of 300 mm x 300 mm and was constructed using normal strength concrete (Section B-B in Fig. 1). A bottom region of 600 mm high represents the potential plastic hinge region. The plastic hinge consists of a lightly reinforced normal strength concrete core having a cross section of 210 mm x 210 mm encased in a UFC jacket having a wall thickness of 45 mm so that the core concrete section carries the shear force at the plastic hinge (CF-UFC, Section A-A in Fig. 1).

The UFC jacket consists of twelve 50 mm high UPC segments. Each internal side of a UFC segment has 28 circular indentations to improve the cohesion between the core concrete and the UFC jacket (Fig. 2). Furthermore, a loading stub 700 mm high atop of the pier is used. The stub has a 400 mm square cross section and manufactured using normal strength concrete.



**Figure 2.** UFC segment

The second pier, hereinafter referred to as PC-UPC, has geometry similar to RC-UFC. However, the potential plastic hinge region was constructed using hollow-core UFC segments without the normal strength concrete core (Section C-C in Fig. 1). A 12 mm thick steel plate was inserted between the plastic hinge region and the RC region i.e. atop of the 600 mm high UFC region.

For both piers, the 100 mm below the footing surface with a cross section of 700 x 700 mm were constructed using UFC to avoid local crushing of the footing. The interface joints between the segments are dry joints.

For RC-UFC pier, 36 vertical mild rebar D 6 (6 mm diameter, C2-D6 in Fig. 1) were uniformly distributed around the perimeter of the pier cross section. The rebar were welded at the bottom of the foundation to a thick steel plate and extended 180 mm into the loading stub. The mild steel bars passed through 10 mm diameter PVC tube extending through the UFC segments (i.e. the rebar were unbonded in the potential plastic hinge region) and it was bonded through the top normal strength concrete region. Such unbonding of the rebar allows uniform distribution of the axial strains in the unbonded length of the rebar allowing the system to reach large displacements before yielding of the rebar. To ensure adequate flexural resistance of the core concrete, additional 16 mild rebar D 6 (C1-D6 in Fig. 1) were uniformly distributed around the perimeter of the core concrete cross section. This reinforcement extended all way through the foundation and was welded to a steel plate located inside the foundation. For shear reinforcement, 1 D 6 and 1 D 4 (4 mm diameter) @ 50 mm were used as shown in Fig. 1. Therefore, the shear resistance of the pier is provided by the lightly reinforced concrete core as well as the closed ties.

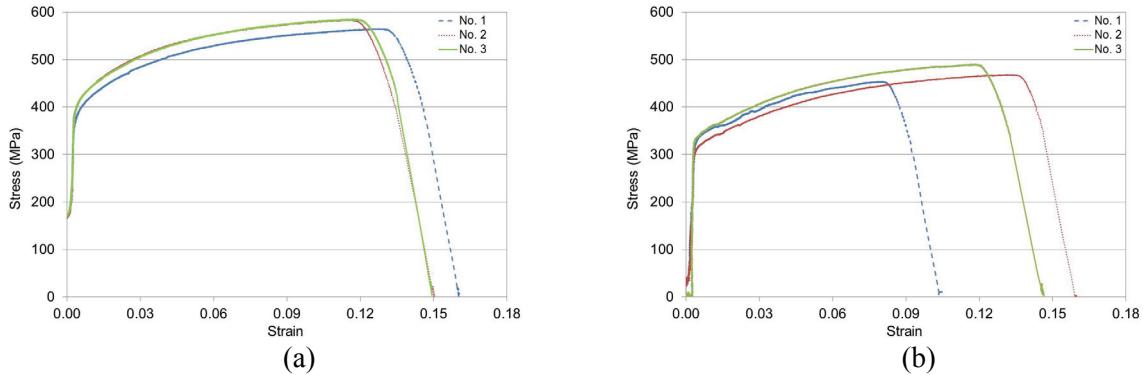
The reinforcement for the PC-UFC pier is similar to those of RC-UFC with three main differences. The first difference is that the PC-UFC was reinforced using a 28.5 mm diameter post-tensioning Dywidag rebar having unbonded length of 1690 mm. The Dywidag rebar passed through a 55 mm diameter polyvinyl chloride (PVC) duct located within the middle of the cross section of the pier in the top region. The second difference is that the core flexural reinforcement (C1-D6 in Fig. 1) were not extended in the bottom region of the pier and was welded to the 12 mm thick steel plate that was inserted at 600 mm from the base. The third difference is that for shear reinforcement, a D 6 (6 mm diameter) close ties @ 50 mm were used in the plastic hinge region. Therefore, the shear resistance of the pier is provided by the friction between segments and the applied post-tensioning force as well as the D 6 closed ties. For the top region, ties similar to those of RC-UFC pier were used.

## 2.2. Material characteristics

Normal strength concrete having a nominal compressive strength of 31 MPa and elastic modulus of 27.95 GPa was used for the concrete core and foundation construction while ultra-high strength steel fiber concrete (UFC) having a nominal compressive strength of 191 MPa and elastic modulus of 45.16

GPa was used for the construction of the segments. The UFC segments were supplied by a precast commercial company.

The steel rebar D6 (SD345) had yield strength of 391 MPa and E-modulus of 194 GPa. The rebar D4 (SD295) had yield strength of 318 MPa and E-modulus of 163 GPa. The post-tensioning rebar had yield strength of 1777 MPa, and E-modulus 187 GPa. The stress-strain curves for three different specimens of D4 and D6 are shown in Fig. 3.



**Figure 3.** Stress-strain curves for (a) SD 345 (D6) and (b) SD 295 (D4)

### 2.3. Loading pattern and test set-up

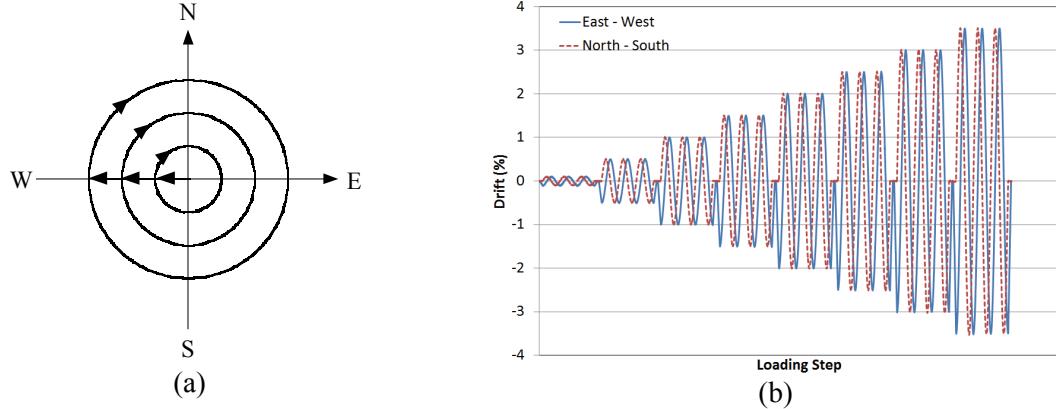
To simulate the dead load from a bridge superstructure, a constant vertical force of 86.1 kN corresponding to axial stresses of 0.97 MPa, and 1.88 MPa for solid and hollow cross-sections, respectively, was applied at the top of each column using a vertical actuator. The applied axial load was kept constant during testing. The columns were loaded using a displacement control protocol in circular orbits (Fig. 4a). A column was first loaded toward the west direction until a drift of 0.5%. From this point, the column was loaded three cycles by circular orbit having a diameter of 0.5% drift. Finally the column was unloaded in the west direction to its original position. This set of loading was repeated with a drift increment of 0.5% until failure occurred (Fig. 4b). The drift is defined as the lateral displacement at mid-height of the loading stub divided by 1370 mm which is the height from the top face of the foundation to mid-height of the stub. During loading the first cycle for specimen RC-UFC, the actuators malfunctioned and caused unintended drift of 1.2% in the southeast direction. This led to yielding of some rebar (C2-D6 in Fig. 2) located in the northwest, southwest, and southeast of the bottommost column cross section. However, no damage was observed in the UFC segments. After that, the specimen was unloaded and re-loaded in the intended circular orbits (Fig. 4a).

## 3. EXPERIMENTAL RESULTS AND DISCUSSION

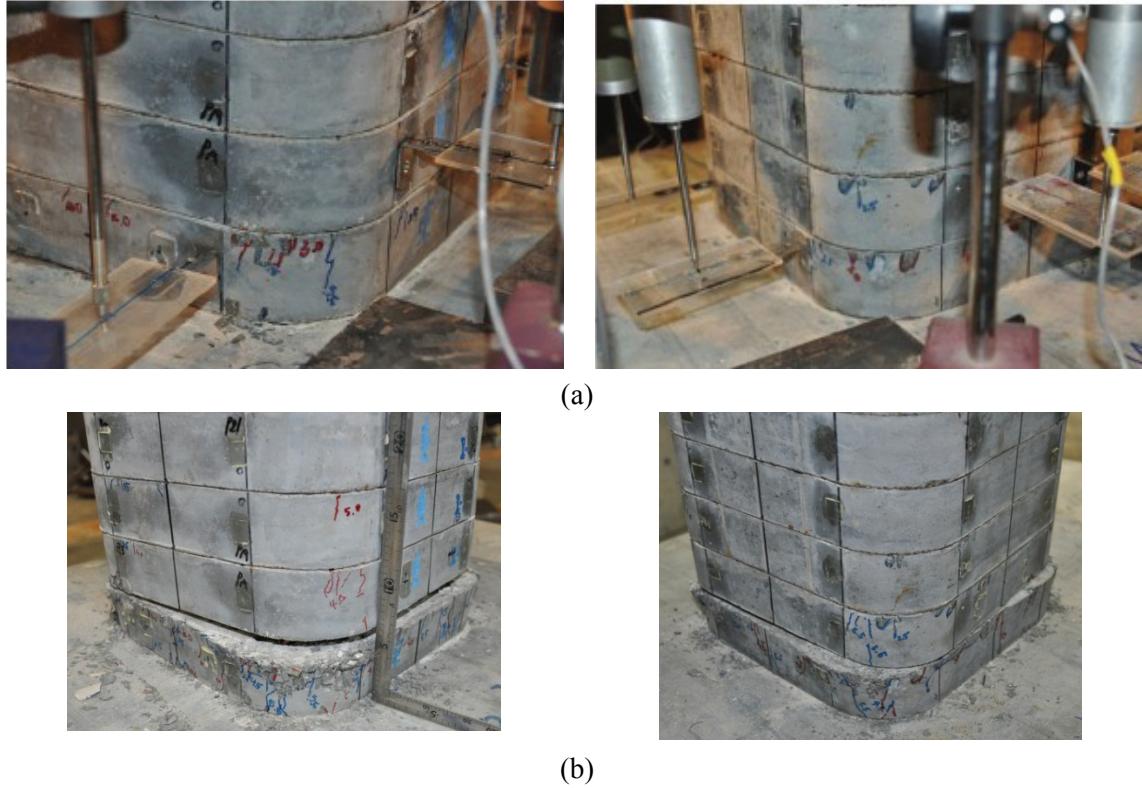
### 3.1. General behaviour

For RC-UFC column, no cracks were observed until loading to a drift of 1.5% where minor spalling occurred in the first (bottommost) segment at the toes. By loading to a drift of 2.5%, several vertical splitting cracks occurred in the first and second segments. The average length of the cracks was about 25 mm in each segment. Fig. 5 (a) shows the crack pattern of the test specimen at a drift of 3.5%. By drift of 4%, the column started to twist counter clockwise on top of the first segment. At this stage, more spalling and vertical cracking appeared in the first and second segments. However, the damage was very local and limited to the UFC jacket. Longitudinal rebar fracture started at drift of 5.5% and it was accompanied by a loud popping sound. Multiple rebar fractured when the column was subjected to a drift of 6% and the test was ended at a drift of 6.5% due to rupture of the rebar (Fig. 5b). By the test end, the column respond mainly in a rocking mode and the interface joint between the first and

second segment opened as large as 12 mm. Furthermore, there was neither significant spalling nor crushing of the UFC segments. Finally, the damage penetration into the concrete core was measured using metal ruler which was inserted in the interface joint between the 1<sup>st</sup> and 2<sup>nd</sup> segments. It was found that the damage extended 53 mm, 10 mm, and 34 mm inside the normal strength concrete core in the east, north, and south directions of the column, respectively.



**Figure 4.** Cyclic loading (a) circular orbits, and (b) East-West and North-South loading



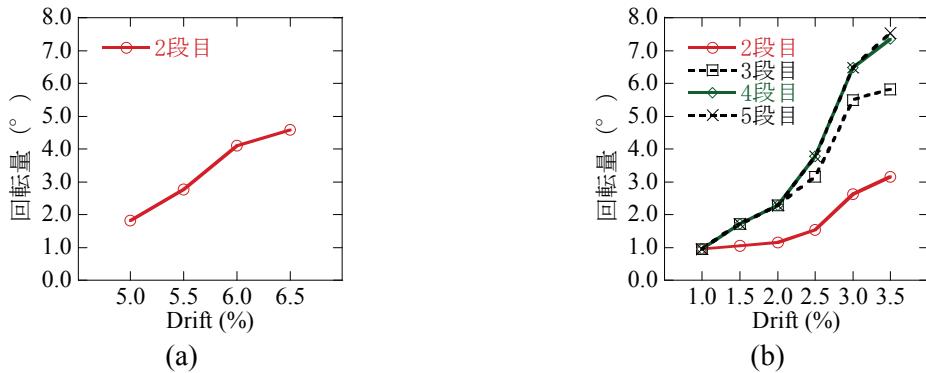
**Figure 5.** The southeast (left) and northwest (right) corners of specimen RC-UFC at drift (a) 3.5% and (b) 6.5%

For the PC-UFC, at 1% drift, limited spalling in the corner of the top of the 1<sup>st</sup> segment occurred. At drift 3%, a rumbling sound indicating that the interior surface of the UFC jacket started to spall and fall on the foundation was heard. At drift of 3.5%, damage progressed quickly in the UFC segments causing reduction in the strength and the test was stopped. Fig. 6 shows specimen PC-UFC at the test end. Specimen PC-UFC has more damage and twisting at the bottom segment compared to specimen RC-UFC as shown in Figs. 5(a) and 6.

Since the columns were subjected to circular orbits cyclic loading, some twisting in the segments occurred. In general, the twisting angle for the PC-UFC column was higher than that of column RC-UFC. For column RC-UFC, twisting took place at the 2<sup>nd</sup> interface joint (just above the 1<sup>st</sup> segment) while twisting took place at the 2<sup>nd</sup>, 3<sup>rd</sup>, 4<sup>th</sup>, and 5<sup>th</sup> interface joints of column PC-UFC. Fig. 7(a) shows the measured residual twisting angle at the 2<sup>nd</sup> segment of column RC-UFC. Although visual inspection showed that twisting started at a drift of 2.5%, the twisting angle measurements started at a drift of 5%. As shown in the figure, the twisting angle increased linearly with increasing the applied lateral drift and it reached an angle of 4.6° at drift of 6.5%. Fig. 7(b) shows also the residual twisting angle measured at the 2<sup>nd</sup>, 3<sup>rd</sup>, 4<sup>th</sup>, and 5<sup>th</sup> segments of columns PC-UFC. The measurements were relative to the 1<sup>st</sup> segment. As shown in the figure and unlike column RC-UFC, twisting took place at several interface joints. By the test end, the PC-UFC column had a permanent twisting angle of approximately 8°. The 2<sup>nd</sup> and 3<sup>rd</sup> segments contributed by approximately 40% (i.e. 3.2°) and 35% (2.8°) of this angle as shown in Fig. 7. This is significantly higher than the measured twisting angle for RC-UFC column where only 1.8° was measured at a drift of 5%. In the case of real earthquake such rotation should be smaller since the loading would not be in the same direction for several cycles.



**Figure 6.** The southeast (left) and northwest (right) corners of specimen PC-UFC at drift 3.5%



**Figure 7.** Measured residual twisting angles for different segments (a) RC-UFC column and (b) PC-UFC column

### 3.2. Hysteretic behaviour

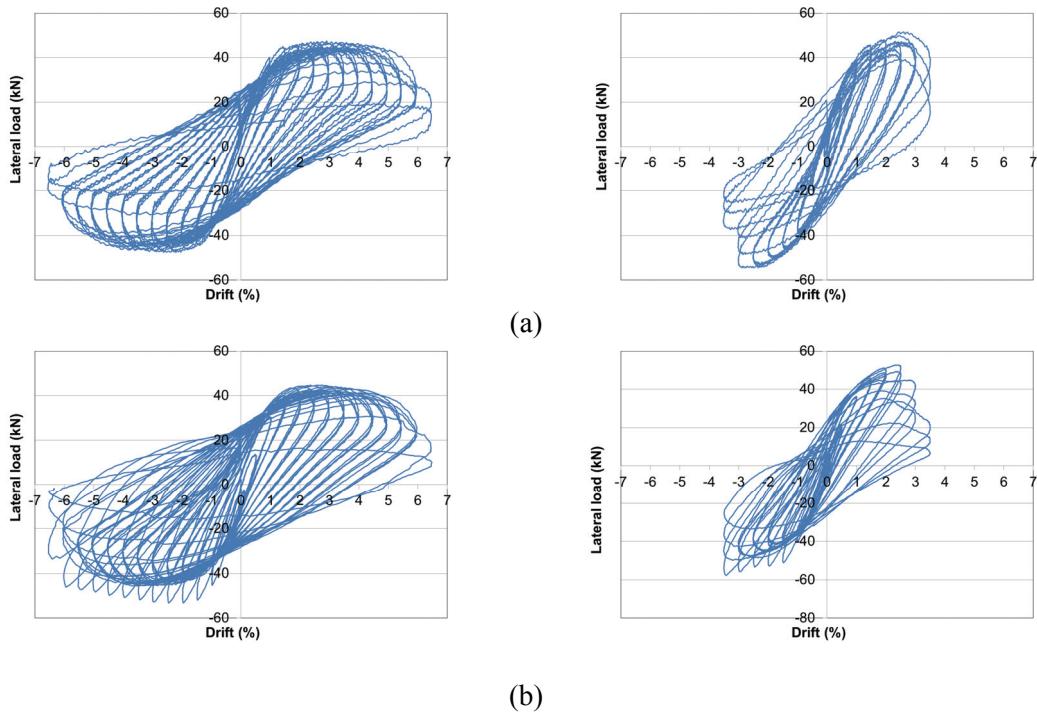
Fig. 8 shows the lateral load versus lateral displacement measured at the middle of the column stub for both specimens. For RC-UFC column and as shown in the figure, the peak strengths of the column were measured at a drift of 2.5% and they were 47.7 kN and 53.3 kN in the North-South and East-West directions, respectively. Beyond that the load started to gradually decrease due to minor spalling in the UFC jacket. At drift of 6.5% and due to multiple rupture of unbonded rebar the strength dropped to 19.4 kN and 16.1 kN in the North-South and East-West directions, respectively which represent a drop by 59% and 70% of the corresponding strengths, respectively.

For PC-UFC column and as shown in the figure, peak strength of 54.5 kN was measured at drift of

3.0% in the North-South. In the East-West direction, the peak strength was measured at a drift of 3.5% and it was 57.7 kN. These peak strengths represent 14% and 8% higher than those measured for the RC-UFC column. Beyond that the load suddenly decreased, due to crushing of the 1<sup>st</sup> segment and rupture in the unbonded rebar, to 25.2 kN and 13.0 kN in the North-South and East-West directions, respectively which represent a drop by 54% and 73% of the corresponding strengths, respectively.

### 3.3. Strain in the ties

The axial strains in the ties in the 1<sup>st</sup> and 4<sup>th</sup> segments are plotted versus the drift in the North-South direction and are shown in Fig. 9. As shown in the figure, for a given drift, the strains in column PC-UFC are generally higher than those measured in column RC-UFC. These higher strains indicate a different shear resistance mechanism. In column RC-UFC, the shear was resisted by the ties in the segments, ties in the core concrete, the concrete core, and the UFC jacket while in column PC-UFC the shear was resisted by the friction between segments and the post-tensioning force as well as the ties in the UFC segments and UFC jacket. Furthermore, the strains in the 1<sup>st</sup> segment of each column are higher than the strains in the 4<sup>th</sup> segment in the same column.



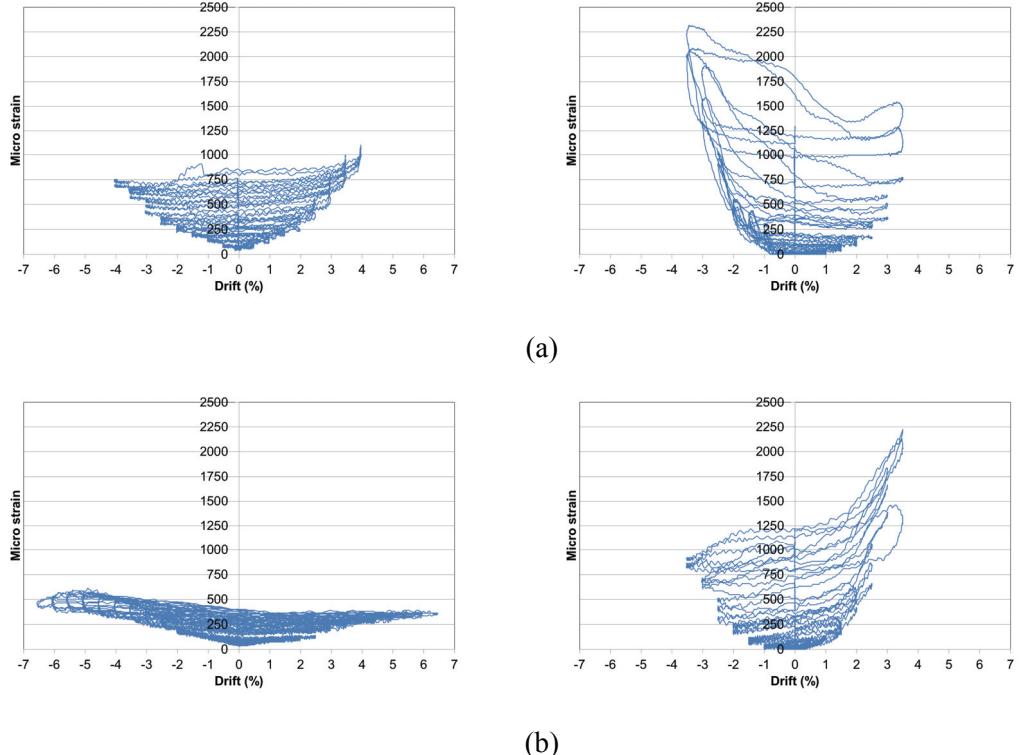
**Figure 8.** Hysteretic curves for specimens RC-UFC (left), and PC-UFC (right) in the (a) North-South , and (b) East-West loading directions

In general and as shown in the figure, the response was quite stable and the strains in the steel increased with increasing the applied lateral drift. The response was symmetric to certain extend in specimen RC-UFC while it was asymmetric in specimen PC-UFC due to torsion effects which took place in the PC-UFC column quite early. For RC-UFC, the strain in the 4<sup>th</sup> segment was small and reached a value of approximately 500 $\mu$  strain at drift of 6.5% while it reached a value of approximately 1000  $\mu$  strain at drift of 3.5% at the 1<sup>st</sup> segment where some local damage occurred. Beyond drift of 4%, the strain gauge at the first segment malfunctioned and the strains were not correctly measured. For the PC-UFC column, the strain in the ties reached high values exceeding the yield strains at the 1<sup>st</sup> and 4<sup>th</sup> segments at drift of approximately 3%.

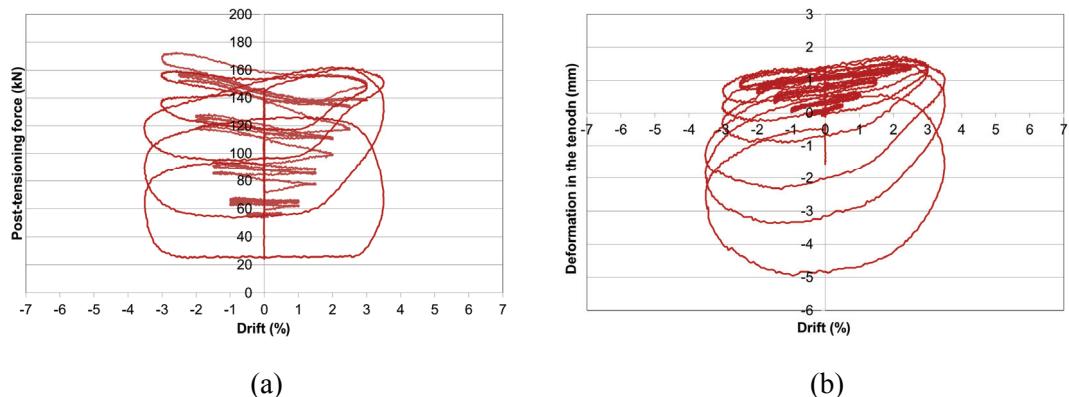
### 3.4. Post-tensioning force

In the PC-UFC column, as the applied lateral displacement increased, the amplitude of the openings of

the interface joints between segments increased. The post-tensioning rebar elongated once the interface joint opening extended to the centre of the column cross section. Increases in the post-tensioning bar length were uniformly distributed over the unbonded length, allowing the system to reach a drift of 3.5% without yielding the post-tensioning bar. Fig. 10 shows the measured post-tensioning force and axial deformation versus applied drift in the North-South direction. As shown in the figure, the ultimate forces in the post-tensioning bar reached 173 kN or 3.2 times the original post-tensioning force. This increase in the post-tensioning forces corresponds to a stretch of 1.7 mm in the post-tensioning bar as shown in Fig. 10(b). By increasing the applied drift to 3.5%, damage occurred at the 1<sup>st</sup> segment and the post-tensioning force decreased to 24 kN corresponding to decrease in the column height by 5.0 mm. Such decrease in the post-tensioning force led to losses in the column shear capacity, stiffness, and the restoring abilities as shown in Fig. 8.



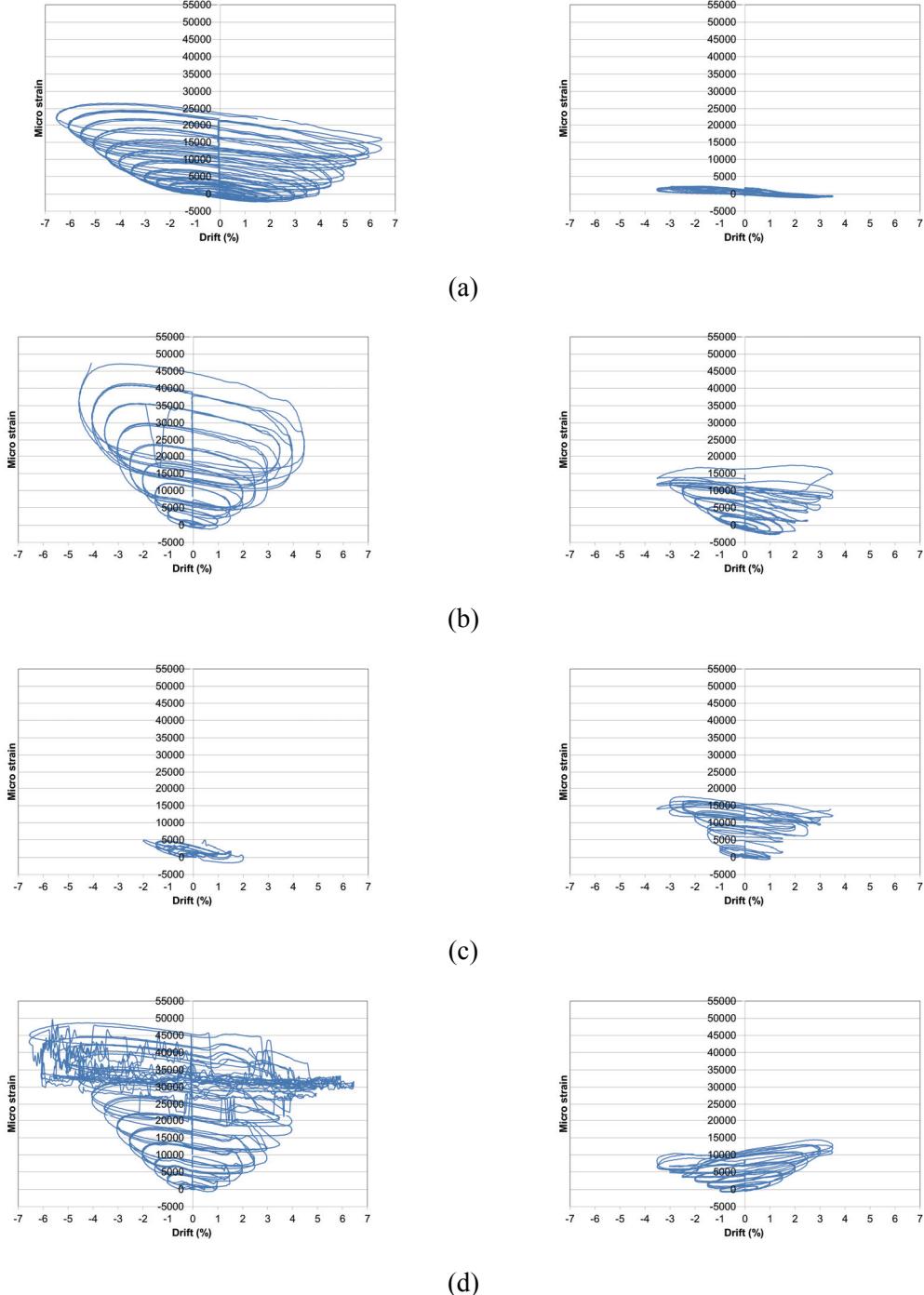
**Figure 9.** Drift versus strains in the ties for columns RC-UFC (left) and PC-UFC (right) at (a) 1<sup>st</sup> and (b) 4<sup>th</sup> segments



**Figure 10.** Loading history versus drift for the post-tensioning rebar (a) post-tensioning force, and (b) deformation in the rebar

### 3.5. Strain in the longitudinal unbonded rebar

The axial strains in the unbonded rebar (C2-D6 in Fig. 1) in the UFC segments in the northeaster corner at heights of 0, 142.5 mm, and 242.5 mm from the top of the foundation of each column are shown in Fig.11. For column RC-UFC, the strain gauges at heights of 142.5 mm and 242.5 mm malfunctioned at drifts of 4.5% and 2.0%, respectively. Shown also in the figure are the axial strains at height of 767.5 mm from the top surface of the foundation i.e. inside the normal strength concrete region.



**Figure 11.** Strains in the unbonded rebar in the RC-UFC column (left) and PC-UFC column (right) at (a) 0 mm, (b) 142.5 mm, (c) 242.5 mm, and (d) 767.5 mm

As shown in Fig. 11, all the rebar yielded quite early during the test. In addition, for a given drift, the

strains are significantly higher in the RC-UFC column compared to column PC-UFC. It is also of interest to note that despite that the rebar were unbonded, the axial strains are significantly different on the same rebar from one location to the other within the unbonded length. For example, in the PC-UFC column, the strains in the rebar at heights of 0 mm, 142.5 mm, 242.5 mm at drift of 3.5% are -780, 8020, and 12000  $\mu$  strains. Similar conclusion can be extended to the RC-UFC column although the malfunctions of some strain gauges. For example at 3.5% drift the strains at heights of 0, and 142.5 mm are 6380 and 27000  $\mu$  strains, respectively. Similarly at 2% drift, the strains are -917, 6087, and 1376  $\mu$  strains at heights of 0 mm, and 142.5, 242.5 mm, respectively

#### 4. CONCLUSIONS

This paper presents a bilateral cyclic loading experimental investigation of bridge columns having plastic hinges consisted of ultra-high performance steel fibre concrete (UFC). Two columns having different plastic hinge details were investigated. The first column, RC-UFC, consisted of two regions: a top region of 330 mm high constructed using normal strength concrete and a potential plastic hinge region 600 mm high constructed using lightly reinforced normal strength concrete encased in hollow-core UFC jacket. The second column, PC-UPC, constructed using unbonded post-tensioning and has geometry similar to column RC-UPC. However, the potential plastic hinge region was constructed using hollow-core UFC segments without the normal strength concrete core. Both columns were designed to have approximately the same nominal strength. The experimental tests showed that RC-UFC column was able to carry the applied axial load until a drift of 6.5% whereas the PC-UFC column was able to carry the applied axial load until a drift of 3.5%. In addition, the hole-core column PC-UFC suffered significant torsion which resulted in a residual twisting angle of 8° at a drift of 3.5% while the counterpart RC-UFC column suffered a twisting angle of 4.6° at drift of 6.5%.

#### REFERENCES

- Chang, K. C., Loh, C. H., Chiu, H. S., Hwang, J. S., Cheng, C. B., and Wang, J. C. (2002). Seismic behavior of precast segmental bridge columns and design methodology for applications in Taiwan. *Taiwan Area National Expressway Engineering Bureau*, Taipei, Taiwan in Chinese.
- Chou, C.-C., and Chen, Y.-C., (2006). Cyclic tests of post-tensioned precast CFT segmental bridge columns with unbonded strands. *J. Earthquake Engng. Struct. Dyn.* **35**: 159-175.
- Dawood, H., ElGawady, M., and Hewes, J., (2012-in press). Behavior of segmental precast post-tensioned bridge piers under lateral loads. *J. of Bridge Engineering ASCE*.
- ElGawady, M., Booker, A., Dawood, H. M. (2010). Seismic behavior of post-tensioned concrete filled fiber tubes. *J. of Composites for Construction ASCE* **14:5**, 616-628
- ElGawady, M., and Dawood, H., (2012). Analysis of segmental piers consisted of concrete filled FRP tubes. *Engineering Structures*, **38**, 142-152
- ElGawady, M., and Sha'lan, A., (2011). Seismic behavior of self-centering bridge bents. *Journal of Bridge Engineering ASCE* **16:3**, 328-339.
- Hewes, J. T., and Priestley N. (2002). Seismic design and performance of precast concrete segmental bridge columns. *Univ. of California at San Diego*, Report No. **SSRP-2001/25**.
- Marriott., D., PaMPanin, S., and Palermo, A., (2009). Quasi-static and pseudo-dynamic testing of unbonded post-tensioned rocking bridge piers with external replaceable dissipaters. *J. Earthquake Engng. Struct. Dyn.* **38**, 331-345.
- Rossi, P. (2000). Ultra-high performance fibre reinforced concretes (UHPFRC): an overview. *Proceedings pro015 : Fifth RILEM Symposium on Fibre-Reinforced Concretes (FRC) BEFIB*, Lyon, France
- Yamashita, R., and Sanders, D. H. (2009). Seismic performance of precast unbonded prestressed concrete columns. *ACI Structural Journal* **106: 6**, 821-830.
- Yamanobe, S., Sogabe, N., Iemura, H., and Takahashi, Y. (2008). Development of high-seismic-performance reinforcement concrete bridge pier with high-performance plastic hinge. *Proc. JSCE*, **64:2**, 317-332