1. Introduction

Characterized by “fat” hysteretic loops, ductile reinforced concrete columns possess great capabilities of energy dissipation. The current seismic design codes for concrete structures require structural components to survive a design-strong earthquake mainly by the large energy-dissipation capacity of the plastic hinges. However, has been observed in recent catastrophic seismic events such as the Chile earthquake in 2010 and the Japan and New Zealand earthquakes in 2011, most of the concrete components designed in accordance with the current codes survive strong earthquakes but are left with large residual deformations. As indicated in previous studies (Todd et al. 1994; Fujino et al. 2005; Ruiz-Garcia and Miranda 2010), the large residual deformations observed in ductile concrete structures make it difficult, if not impossible, to rehabilitate them. Therefore, increasing attention has been attracted to reduce the residual displacement of the structures for the immediate recovery of structural functionality after a strong earthquake. At present, two effective methods to reduce the residual deformation of concrete columns are (1) improving the hardening stiffness of the components (Kawashima et al. 1998; Pettinga et al. 2007; Fahmy et al. 2010; Guerrero et al. 2017), and (2) possessing self-centering capacity of the components (Mahin et al. 2006; Marriott et al. 2009; Qu et al. 2010; Pettinga et al. 2007; Guerrero et al. 2017). Another very effective method to reduce the residual drift is the construction and promotion of a new type of hybrid concrete structural system that consists of a resilient concrete frame and sets of energy-dissipation devices (Sun et al. 2013) as shown in Fig. 1(a). The resilient concrete frame possessing self-centering capability is intended to resist gravity and earthquake action, while the energy-dissipation devices are intended to dissipate most of the earthquake input energy. Thus, the drift-hardening concrete column with reduced residual drift is required to construct the resilient frame. Teran-Gilmore and Virto-Cambray (2009) and Guerrero et al. (2017) also proposed a similar concept. The schematic hysteretic responses of drift-hardening and ductile concrete columns are shown Figs. 1(b) and 1(c), respectively. In fact, drift-hardening capability of the column not only helps to reduce the residual deformation, but also is effective to reduce displacement demand under a strong earthquake (Ruiz-Garcia and Miranda 2003).

The main purpose of this paper is to put forward a simple method to make drift-hardening circular concrete columns with reduced residual deformation. For this purpose, non-prestressing PC strands (referred to as PC strands) are utilized as longitudinal reinforcement in the columns instead of deformed normal strength (NS) rebars. The proposed approach helps to simplify the con-
struction process compared with the traditional prestressed concrete columns. Thus, the potential capability of drift-hardening and construction benefit make it practical to apply the proposed columns in the concrete structures in high-intensity earthquake-prone regions.

The quasi-static experimental results on the seismic performance of five concrete columns reinforced by PC strands or NS rebars are presented. By this means, the following purposes are to be achieved: (1) validate the effectiveness of adopting PC strands as longitudinal rebars; (2) study the mechanical behavior and failure pattern of the proposed columns; (3) investigate the effects of partial confinement by bolted steel tube (BST) on the seismic performance of the proposed column; and (4) introduce a nonlinear analysis method considering the bond-slip effect of PC strands to evaluate the lateral response of the columns.

2. Review of previous research

Drift-hardening concrete structures with reduced residual displacement have been studied and can be traced back to the investigation by Priestley and Tao (1993). As the culmination of the 10-year research program “PRESSS” (Precast Seismic Structural Systems), Priestley et al. (1991, 1999) have revealed that the seismic behavior of the structure comprised of ductile frames and structural wall system jointed by utilizing unbonded post-tensioning (UPT) tendons was extremely satisfactory, with no significant strength loss in the frame direction despite the drift levels up to 4.5%.

Zatar and Mutsuyoshi (2002) employed UPT tendons in RC bridge piers to reduce the residual displacement of piers. The study showed that this method could significantly reduce the residual displacement, restrain cracking, and enhance shear strength of UPT piers after an earthquake. In addition, the UPT piers exhibited the advantage of lowering the maximum drift than the RC piers due to the self-centering response, despite the lower energy consumption of the UPT column.

In order to enhance the energy-dissipation capacity of the structural system with UPT columns, Marriott et al. (2009) conducted experimental research on UPT columns with external dissipaters. Minimal physical damage was observed for the post-tensioned systems, which exhibited very stable energy dissipation and self-centering properties.

Similar observations of the investigation on the UPT columns were also obtained by Billington and Yoon (2004), Mahin et al. (2006), Ou et al. (2010), and Song et al. (2015).

Another potential candidate to achieve the drift-hardening capability of concrete columns with reduced residual drift is the use of deformed high strength (HS) rebars as the longitudinal steel. It is realized by the longer elasticity of HS rebars than NS rebars, which also enables the columns to return to its original position at a larger drift than conventional ductile columns.

Watanabe and Osumi (1991) recommend a mixed use of HS rebars and NS rebars as the longitudinal reinforcement to give columns a high degree of protection against premature yielding under severe earthquakes. Iemura et al. (2004) and Cai et al. (2018) also presented a similar method to improve the drift-hardening capacity and reduce the residual drifts of columns.

Sun and Fukuhara (2005) conducted experiments on four one-bay and one-story high strength concrete frames (HSPCFs) with all the longitudinal reinforcements adopting HS rebars. The results showed that the application of HS rebars as longitudinal reinforcement could assure high ductility and reduce the residual story drift of the HSPCFs, while also ensuring the frame a drift-hardening capability till the drift of 2.0 to 2.5%, where the HS rebars began to yield. Similar conclusions were also obtained by Tavallali et al. (2011) by experimental research on the concrete beams and columns reinforced by HS rebars.

However, shear failure was observed in the columns reinforced by the conventional HS rebars at the drift of 3% and the drift-hardening capability was limited only to the drift of 2.5% due to the yielding of HS rebars (Sun et al. 2012). Therefore, Sun et al. (2012, 2013) proposed a method of utilizing low-bonded HS rebars instead of conventional HS rebars. Test results showed that the employment of low-bonded HS rebars as longitudinal reinforcements in the column not only circumvented the potential shear failure but also improved the drift-hardening capability of the columns until to the drift of 4% with a residual drift less than 0.5% even under high axial load. The low-bond strength further delayed the yielding of the HS rebars enabling the drift-hardening capability of columns to sustain up to a larger drift (Fu-
nato et al. 2012).

The use of PC strands, with identical features of low bond and high tensile yield strength (Ichiki et al. 2002), is a potential selection to achieve the drift-hardening capability of columns. Shim et al. (2008) experimentally investigated the seismic performance on prefabricated bridge columns with post-tensioned bonded threaded prestressing bars, and also investigated a combination of continuous mild reinforcing bars and the post-tensioned prestressing bars, and also investigated a combination of continuous mild reinforcing bars and the post-tensioned PC strands as longitudinal rebars (Shim et al. 2017). The test results showed that for columns with threaded prestressing bars, there was no effect of the prestress level on the self-centering capability at a larger drift. However, the columns with post-tensioned PC strands exhibited unreduced flexural strength up to 8% drift with significantly reduced residual deformation.

Yuan et al. (2018) utilized non-prestressing PC strands in the boundary elements of concrete wall and concluded that the non-prestressing PC strands were capable of ensuring the drift-hardening capability till 2.5% drift for concrete wall under low axial compression. From the literature review stated above, it can be concluded that the use of PC strands as longitudinal reinforcements is a potential solution for the concrete column to obtain drift-hardening capability at a large drift. In order to verify the effectiveness of this method, this paper carried out experimental studies on the circular concrete columns reinforced by PC strands. Furthermore, to enhance performance of the potential hinge region of the specimens, partial confinement by bolted steel tube at the bottom of the column has also been used and examined.

### 3. Experimental program

#### 3.1 Details of the specimens

Five concrete columns were fabricated as shown in Fig. 2. All test columns had a circular section with an outer diameter (D) of 300 mm. The shear span ratio (a/D) was specified as 3 and 4, where a was the height from the top surface of the foundation beam to the loading point. The axial load ratio (n) was designed as 0.15 and 0.35. Table 1 lists the details of the tested columns.

Four test columns were reinforced by the seven-wires PC strands with a nominal diameter of 15.2 mm (China GB/T 5224-2014, 2014) (labeled PC15.2) as longitudinal rebars. For comparison, one column utilized HRB400 (China GB/T 50010-2010, 2010) deformed rebars with a nominal diameter of 12 mm (labeled C12) as longitudinal rebars. Ten PC15.2 were used in the proposed columns. To keep each column had identical longitudinal reinforcement ratio (ρ_l) of 2.0%, twelve C12 rebars were adopted in the comparison column. To prevent premature slippage of the PC strands, each end of the PC strand was pressed into a 90 mm long extruded duct housing with an inner thread of 65 mm in length, and a steel bar of 70 mm long was then screwed into the remaining 25 mm of the extruded duct to fix to a 12 mm-thick steel plate by nuts (see Fig. 3).

Stirrups (C6) were adopted in all columns. To prevent the columns from shear failure, the calculated shear capacity (V_{su,ACI}) according to ACI code provisions (ACI 2014) were assured to exceed the predicted flexural capacity (V_{fu,1}) by nonlinear analysis of beam-column element using “OpenSees” computer software (devel-

### Table 1 Details of the tested columns.

| Series | Notation | f'_c (MPa) | n | a/D | Rebars | ρ_l (%) | Lateral confinement | V_{fu,1} (kN) | V_{su,ACI} (kN) |
|--------|----------|-------------|---|-----|--------|---------|---------------------|--------------|---------------|
| I      | NS3A15   | 28.2        | 0.15| 3   | 12-C12 | 1.92    | Hoop               | 1.48         | 96            |
|        | HS3A15   |             | 0.15| 3   | 10PC15.2 | 1.98 | Hoop             | 1.48         | 256          |
|        | HS3A35   |             | 0.35| 3   | 10PC15.2 | 1.98    | Hoop             | 1.48         | 254          |
| II     | HS4A15   | 19.2        | 0.15| 4   | 10PC15.2 | 1.98    | Hoop+BST          | 1.48         | 180          |
|        | HS4A15SP2|             | 0.15| 4   | 10PC15.2 | 1.98    | Hoop+BST         | 1.48         | 245          |

### Fig. 2 Reinforcement details and dimensions (in mm) of test columns.

![Fig. 2 Reinforcement details and dimensions (in mm) of test columns.](image-url)
oped by the Pacific Earthquake Engineering Research Centre, USA for carrying out earthquake engineering simulations, based on plane-remain-plane assumption as listed in Table 1. The simulation method in OpenSees to calculate \( V_{fu,1} \) will be introduced in Section 4.2. A spacing of 30 mm was determined for stirrups C6 and the corresponding steel ratio \( \rho \) was 1.48%.

To strengthen the potential region of the plastic hinge, BST with a height of 430 mm was attached for the specimens HS4A15SP2. The BST consisted of two pieces of semi-circular cold bent steel plates with a thickness of 1.87 mm was assembled by bolts and nuts. To provide reliable confinement, clamp plate with a thickness of 10 mm were applied at the bolted joints. A 10 mm gap from the top surface of the concrete base to the bottom of the BST was set to make the BST play a lateral restraint effect only. It should be noted that the spacing of stirrups C6 within the BST was changed into 97.5 mm with steel ratio of 2.96% including the BST.

All the details of the specimens are shown in Figs. 2 to 4 and Table 1. The tensile properties of the steel used are presented in Table 2.

### 3.2 Test instrumentations and apparatus

The specimens were tested under cyclic lateral loading while subjected to constant axial load. The test instrumentations and apparatus are shown in Fig. 5. Steel reaction frames and a concrete reaction wall formed the reaction system for actuators. The lower loading (foundation) beam (500 mm × 400 mm × 1200 mm) of the specimen was fixed to the strong floor by steel beams and screws, which was also restrained by four constrained displacement bolts to prevent movement of the specimen.

A constant axial load was imposed on the top of the loading stub (300 mm × 400 mm × 500 mm) by an MTS servo-controlled hydraulic actuator of 1500 kN capacity, which was installed to the top of the steel reaction frames. Then, cyclically reversed lateral load was applied by another MTS servo-controlled hydraulic actuator (with a capacity of 500 kN) anchored to the reaction wall. The reversed cyclic lateral loading was controlled by the drift \( R \) of the specimen and the drift \( R \) was calculated by dividing the measured tip lateral displacement \( \Delta \) by the shear span \( a \).

**Figure 6** shows the targeted displacement program. Two cycles were adopted at every drift level unless the drift \( R \) beyond 2%, and one cycle afterward until the drift reached 6%. Meanwhile, the increment of drift was 0.25% before the drift reached 1%, while successively, 0.5% from 1% to 4%, and then 1% until the drift approached 6%. Every cyclic unloading was controlled by force and terminated at the force of zero.

### Table 2 Tensile properties of the steel used.

| Notation | \( f_y \) (10^6 Pa) | \( \varepsilon_y \) (%) | \( f_{su} \) (10^6 Pa) | \( \varepsilon_{su} \) (%) | \( \Phi \) (%) | \( E_s \) (10^6 Pa) |
|----------|---------------------|------------------------|------------------------|------------------------|-------------|-------------------|
| C6       | 399.1               | 0.21                   | 574.1                  | 13.6                   | 20.6        | 2.06 \times 10^5  |
| C12      | 429.8               | 0.22                   | 540.4                  | 15.7                   | 21.3        | 1.98 \times 10^5  |
| PC15.2   | 1728.6              | 1.05                   | 1932.8                 | 7.9                    | 9.2         | 2.03 \times 10^5  |
| BST      | 326.2               | 0.16                   | 445.1                  | 21.8                   | 32.5        | 2.18 \times 10^5  |

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**Fig. 3** The anchorage for PC strands.

**Fig. 4** BST for confinement.

**Fig. 5** Loading apparatus.
The arrangement of the displacement transducers (DTs) and strain gages is shown in Fig. 7. To measure lateral drift, displacement transducers (DTs) noted as “H1”, “H2” and “H3” were installed at the height of 300 mm, 600 mm (from top surface of the foundation) and loading point respectively. To measure rotations of cross sections, a pair of DTs (noted as “VE1” and “VW1”) was used at height of 100 mm and pairs of DTs noted as “VE2” and “VW2”, “VE3” and “VW3”, “VE4” and “VW4” were placed at corresponding height of “H1”, “H2” or “H3” of the column. Besides, eight pairs of strain gages were placed on the two outermost rebars to measure the strain of the longitudinal rebars.

### 4. Experimental results and discussion

#### 4.1 Overall observations

Table 3 summarizes the primary experimental and calculated results. Figure 8 displays the crack and spalling-off pattern of the tested columns at 6% drift. The first flexural crack commenced at the drift of 0.5% in specimen HS3A35, while for all the others at 0.25% drift. With increasing drift, specimen NS3A15 developed many cracks within the range of 200 mm from the top of the foundation, while for specimens with PC strands, only one or two dominate cracks within the height range of 150 mm from the top of the foundation were observed. Especially, HS4A15SP2 only had one dominate crack in the gap between the bottom of the BST and the foundation. Until the termination, no obvious shear crack was observed in all the specimens.
Except for the specimen HS4A15SP2, the concrete cover of the other specimens began to spall off at 2% drift. After the test, spalling-off of the specimens NS3A15, HS3A15 and HS4A15 was within height of 150 mm, while 200 mm for HS3A35. During loading of 6% drift, the stirrup of HS3A35 at 90 mm height fractured. As shown in Fig. 8(b), concrete shell of specimen HS4A15SP2 confined by BST was slightly damaged after taking off the steel plates.

4.2 Experimental lateral force-drift responses

The measured lateral force-drift responses of the specimens are presented in Fig. 9. The dotted lines (referred to as the “calculated line”) were the lateral capability lines predicted by fiber element modeling in OpenSees based on plane-remain-plane assumption, which considered the $P$-$\Delta$ effect. To obtain the “calculated line”, Nonlinear Beam Column element (Neuenhofer and Filippou 1997) was used to simulate the test column. The lateral force-drift response of the column was obtained by integration of the moment-curvature response of the fiber section along the element, which adopted five Gauss-Lobatto’s integration points. Concrete01 in OpenSees was employed to model the concrete (see Fig. 10), in which Mander’s model (Mander et al. 1988) was applied to consider the confinement by hoops or BST on core concrete. The critical parameters to define the Concrete01 model of cover and core concrete are listed in Table 4, and Steel02 developed by Menegotto and Pinto (1973) was adopted to simulate the steel rebars as shown in Fig. 11 and the parameters are listed in Table 5.

The NS3A15 exhibited typical characteristics of ductile components with ideal “fat” hysteresis curves. Comparatively, the lateral-resistant capability of HS3A15, HS4A15 and HS4A15SP2 reinforced by PC strands increased up to 6% drift with a “slender” hysteresis curve. While HS3A35 showed a drift-hardening capacity up to 5% drift, indicating that concrete columns with PC strands had an excellent lateral-resistant capacity even under high axial load ratio.

From the comparison between the “calculated line” and experimental results as shown in Fig. 9, it demonstrated that the calculation method based on the plane-remain-plane assumption could evaluate the lateral capacity of the columns with NS rebars very well, but it would significantly overestimate the capacity of the proposed columns. So, a new calculation method considering the bond-slip effect is demanded for specimens with PC strands, which will be introduced in the following Section.

To exhibit the influence of various parameters, Fig. 12 compares the envelope curves of the base moment-drift...
relationships on average (in push and pull directions), in which $P-\Delta$ effect has been eliminated. After the drift reached 0.75%, the flexural capacities of NS3A15 was greater than that of HS3A15, indicating that the bond-slip began to have an evident effect around 0.75% drift. When the drift was greater than 1.5%, the flexural capacity of HS3A15 became greater than that of NS3A15. This phenomenon was caused by the fact that C12 rebars had yielded around 1.5% drift, but the PC strands did not

yield. Compared with HS4A15, HS4A15SP2 showed greater drift-hardening stiffness, indicating that the use of BST as partial constraint was helpful to improve the lateral-resistant capacity of columns. Due to higher axial load ratio, HS3A35 exhibited a greater initial stiffness but weaker drift-hardening stiffness than that of HS3A15.

![Fig. 9 Lateral force-drift responses.](image)

![Fig. 10 The model of Concrete01.](image)

![Fig. 11 The model of Steel02.](image)
4.3 Measured residual drifts

The measured residual drifts versus the peak drift on average (in push and pull directions) are shown in Fig. 13. At the drift of 2%, the residual drifts of the proposed columns were all less than 0.5%, while NS3A15 had larger residual drift.

The residual drift of NS3A15 showed faster growth than HS3A15 after 1% drift. In addition, HS3A35 displayed greater residual drift than HS3A15 due to the severer $P$-$\Delta$ effect and concrete damage. Comparing with HS4A15, HS4A15SP2 exhibited less residual drift after 3.5% drift, suggesting that use of BST had an active impact on reduce of residual deformation at large drift.

4.4 Measured strains for longitudinal reinforcement

The measured strains “Exp” of the longitudinal C12 rebars and PC strands located at height of 15 mm or 165 mm from the top of the concrete base are depicted in Fig. 14, and the location of the adopted strain is also shown. The black horizontal dotted lines display the yield strain of the longitudinal bars, and the red dotted curves represent the calculated strains, which will be discussed in the next section. The C12 rebars reached tensile yield strain at the drift between 0.75% and 1%. However, the PC strands did not yield during the loading process inferred from the trend of strain growth. With the increasing drift, the increase of the resistance capacity of the column caused by the increasing stress of the PC strands was enough to compensate for the decrease of the resistance capacity caused by the spalling-off of the concrete cover and the $P$-$\Delta$ effect, so that the column exhibited drift-hardening behavior. At the same time, the elastic restoring force of the PC strands obviously helped to reduce the residual displacement of the column.

The increase of tensile strains of PC strands slowed down around 0.75% drift due to the occurrence of slip. Its compressive strain no longer increased between 0.75% and 1.5% drift, even the tensile strain appeared. This indicated that the PC strands produced a certain untwisting, but the untwisting would be restrained by surrounding concrete and hoops.
To reflect further the behavior of the longitudinal reinforcement, Fig. 15 displays the profile of the strains along the height of the longitudinal reinforcement at various specific drifts. Digital missing as a result of the damages of the strain gauge at a certain height has occurred, and the dashed line in Fig. 15 is used to connect the measured strains at the adjacent height to exhibit the strain development trend along the height of the column. Strain development of C12 rebars was quite different from that of the PC strands, especially at a large drift. Because strains of C12 bars evidently grew near the foundation, while that of PC strands developed steadily especially after 1.5% drift. In addition, due to the low-bonded slip, stress redistribution along the height of the rebars promoted a more uniform strain distribution of PC strands.

4.5 Measured maximum and residual crack widths

Figure 16 illustrates the measured maximum widths. As shown in the figures, PC strands had no positive impact on the reduction of the maximum widths, but only one or two dominating horizontal cracks in the specimen with PC strands were observed as shown in Fig. 8. The dominating cracks concentrated mainly within the range of 150 mm from the top of the foundation. The fact that the above phenomenon was the result of low bond strength had been verified (Pandey and Mutsuyoshi 2005).

Figure 17 shows the measured residual crack widths. The residual crack width is the residual width of maximum crack. The residual crack width of NS3A15 was narrower than that of HS3A15. However, the NS3A15 exhibited an obvious trend of faster residual crack development than the HS3A15 after the drift of 1.5%. Therefore, although the experimental investigation in this paper is limited to the drift of 2%, it can be predicted that the column reinforced by PC strands is likely to achieve a smaller residual crack width than conventional RC columns at a large drift due to the plastic development of NS rebars, which can be confirmed by the experimental research by Yuan et al. (2018).

The ratio of residual crack width to maximum crack width of specimens HS3A15 and NS3A15 (R/M) is il-
4.6 Measured rotations for different sections

To obtain lateral deformations caused by section rotations, DT couples are allocated as shown in Fig. 7. According to the recorded displacements, section rotations can be calculated by Eq. (1).

\[ \theta_i = \frac{(|VE_i| + |VW_i|)}{L_i} \]

where \( i \) refers to sections numbered 1, 2, 3 or 4 from bottom to top, and \( \theta_i \) is the rotation of the \( i \)-th cross section. \( VE_i \) and \( VW_i \) are the displacements recorded by the east and west DT respectively. \( L_i \) is the distance between the probes of two DTs.

The calculation results suggested a strong linear relationship between \( \theta_i \) and drift \( R \) as listed in Table 6, and the results of specimen NS3A15 as an example was imaged in Fig. 19. Due to the crack concentrated on the column root, the \( \theta_1 \) of the specimen HS4A15SP2 was significantly greater than that of the rest specimens. Besides, comparing with NS3A15, the \( \theta_1 \) of specimens HS4A15 and HS3A15 were greater, which probably caused by slip of PC strands. As for \( \theta_2 \), the \( \theta_2 \) of the specimen HS3A15 with a shear span ratio of 3.0 was larger than that of the specimen HS4A15 with 4.0.

Above all, the values of \( \theta_1/\theta_2 \) were all approaching to 1. It is reasonable to consider that the deformation of the columns is mainly concentrated within the height range of \( 1D \), while the rest parts just rock rigidly. Thus, the assumption (3) in Section 5.1 is regarded as rational.

4.7 Equivalent viscous damping

The measured equivalent viscous damping (EVD) according to the method by Chopra (1995) is shown in Fig. 20. The specimens with NS rebars exhibited greater energy dissipation capacity after the yield of C12 bars than the others by PC strands, and the EVDs of them kept increasing. However, for the specimens with PC strands, the EVDs maintained almost constant after 1% drift, and the EVDs of specimens with axial load ratio of 0.15 were about 10%, while about 14% for axial load ratio of 0.35.

5. Lateral response evaluation

In view of the low bond strength between concrete and PC strands, the conventional calculation method based on plane-remain-plane assumption was no longer appropriate for evaluating the flexural strength of the proposed columns. To assess the lateral response of the columns with PC strands, an effective analytical method

### Table 6 Measured rotations for specimens.

| Notation | NS3A15 | HS3A15 | HS4A15 | HS4A15SP2 |
|----------|--------|--------|--------|-----------|
| \( \theta_1/R \) | 0.496  | 0.571  | 0.638  | 0.938     |
| \( \theta_2/R \) | 0.925  | 0.967  | 0.848  | 0.957     |
| \( \theta_3/R \) | -      | -      | 0.974  | 1.035     |
| \( \theta_4/R \) | 1.013  | 0.968  | 0.916  | 0.986     |

![Fig. 18 Ratio of residual crack width to maximum crack width.](image1)

![Fig. 19 Measured rotations for the specimen NS3A15.](image2)

![Fig. 20 Measured equivalent viscous damping.](image3)
put forward by Sun et al. (2006) [cited by Cai (2014, p. 97)], was introduced and employed. The method developed by Sun et al. (2006) [cited by Cai (2014, p. 97)] is a fiber spring element (FSE) method to evaluate the bond-slip effect of low bond HS bars on hysteretic behavior, which refers to the method suggested by Tada and Takeda (1985).

5.1 Numerical analytical procedures

The following assumptions are made to predict the lateral force versus drift relationship according to Sun et al. (2006) [cited by Cai (2014, p. 97)], and Tada and Takeda (1985): (1) concrete does not resist tensile stress; (2) the concrete section follows the plane-remain-plane assumption; (3) lateral deformation of the specimens is mainly contributed by the flexural rotation concentrated within the plastic hinge zone with a length of 1\(D\); (4) the stress and strain of the PC strands are uniformly distributed within the plastic hinge zone; (5) the constitutive law of the concrete defined by Sun and Sakino (2000) is applied in this model, as shown in Fig. 21; (6) an idealized bilinear hardening stress-strain model is adopted for the material model of the PC strands, as shown in Fig. 22; (7) the bond-slip model (see Fig. 23) developed by Funato et al. (2012) is applied to consider the slip effects along the entire longitudinal reinforcements in the anchorage and rocking zones.

As shown in Fig. 24, the specimen is divided into three zones, namely anchorage zone, hinge zone, and rocking zone. Thus, the elongation of the rebar is regarded as three parts. According to assumption (2), for a given curvature \(\phi_R\) within the hinge zone, the strain of concrete section can be calculated if the depth \(c\) of the neutral axis (see Fig. 25) is known, and then the behavior of the concrete can be determined. For PC strands, the total elongation equals to the sum of the elongation in three zones, see Eq. (2).

\[
\Delta_l = \Delta_l_{\text{anc}} + \Delta_l_{\text{hin}} + \Delta_l_{\text{roc}}
\]

where \(\Delta_l\) is the total elongation of the \(j\)-th row of PC strands, which is equal to the elongation of concrete fiber at the location of PC strands, as shown in Fig. 24. \(\Delta_l_{\text{anc}}, \Delta_l_{\text{hin}}\) and \(\Delta_l_{\text{roc}}\) represent the elongation of PC strands in the anchorage zone, plastic hinge zone and rocking zone, respectively (see Fig. 26). If the stress of the PC strands within the hinge zone is known, the \(\Delta_l_{\text{hin}}\) can be calculated on the basis of assumption (4). According to assumptions (3) the concrete in anchorage and rocking zone be deemed rigid, the \(\Delta_l_{\text{anc}}\) and \(\Delta_l_{\text{roc}}\) can be obtained by calculating the slip.

Depending on the stress transmitting range, two different boundary conditions are imposed on the PC strands in the anchorage zone and rocking zone as shown in Fig. 26. If the stress can be transmitted to the fixing steel plate, the slip at the end of the PC strands must be zero (End S = 0). If not, both the slip and the stress should be zero (Non-end S = 0 and \(f = 0\)) at the end of stress transfer. Taking the calculation process of the slip of rebar in the anchorage zone as an example, the iterative steps are as follows:

1. Discretize the anchor zone into \(m\) segments;
2. For the first segment, given the initial stress \(f_{\text{hin}}\) and an initial slip \(\Delta_{l\text{anc}}\), the stress \(f_k\) and the slip \(S_k\) of the \(k\)-th segments can be calculated by Eqs. (3) and (4),

![Fig. 21 The concrete model of Sun and Sakino (2000).](image1)

![Fig. 22 The model of PC strands.](image2)

![Fig. 23 The model of bond-slip.](image3)

![Fig. 24 Divided zones of specimen.](image4)
respectively;
(3) When \( k \neq m \), if \( f_k = 0 \), it needs to judge whether \( S_k = 0 \), and vice versa. If the boundary condition is not satisfied, the initial slip \( \Delta U_{\text{unc}} \) should be re-assigned, and the process returns to step 2). When \( k = m \), it needs to judge whether \( S_m = 0 \) is satisfied, if not, the initial slip \( \Delta U_{\text{unc}} \) is re-given, and the process also returns to step 2);
(4) If the boundary condition is met, the slip \( \Delta U_{\text{unc}} \) corresponding to the initial stress \( f_{\text{hin}} \) is obtained.

\[
f_k = f_{k-1} - \tau_k (S_{k-1}) c_k x / A_k
\]
\[
S_k = S_{k-1} - \varepsilon_k (f_{k+1}) x
\]

where \( f_k \), \( S_k \), \( \tau_k \) and \( \varepsilon_k \) represents the stress, the slip, bond strength and strain of the rebar in the \( k \)-th segment, respectively; \( c_k \) is the circumference of the rebar, calculated according to the nominal diameter of the rebar, and \( x \) is the length of the \( k \)-th segment; \( A_k \) represents the cross-sectional area of the rebar.

Based on the elongation calculation described above, if the assumed stress of PC strands within the hinge zone can satisfy Eq. (2), then the stress is the real stress of PC strands.

Finally, the lateral response of the member can be determined by the following steps:

5.2 Analysis results and discussion
The specimens reinforced by PC strands, the calculated results of two kinds of bond strengths (\( \tau = 2 \) MPa and \( \tau = 5 \) MPa) are presented. The 2 MPa is the measured results from the experiments on the bond strength of the PC strands (Ichiki et al. 2002), while the 5 MPa is adopted for comparison.

Figure 27 illustrates comparisons between the calculated and experimental envelope curves, in which the “Exp” represents the experimental results, the “Pro” refers to the results calculated by the proposed method and the “Ope” represents the results calculated by using OpenSees based on the plane-remain-plane assumption. Comparisons indicated that the higher bond strength was intended to contribute to a greater lateral resistance capacity. For the specimens reinforced by PC strands, “\( \tau = 2 \) MPa” was suitable as the value of the bond strength for prediction.

The calculated steel strains near the foundation of the column (see Fig. 14) and tensile strains along the height distribution at specific drift (see Fig. 28) were compared with the measured strains. In Fig. 28, the numbers after the signs of “Exp”, “Ope” or “Pro” indicate the percentage of the drift. It is very complicated to consider the untwisting behavior of the PC strand under compression in the analysis. Therefore, for the sake of simplicity, the compression behavior of PC strands was simplified to be consistent with the tensile behavior. As comparison, the measured strains along the height of column show a consistent development trend with that of the analytical model, but the measured strain of the PC strands generally shows a deviation from the analytical model. The difference between the measured and analytical strains was caused by the deviated measurement direction of the strain gauges from longitudinal axis due to the rotation of the stretched and elongated PC strands (Ichiki et al. 2002).

6. Conclusions
This paper put forward a simple approach to making
drift-hardening circular concrete columns by utilizing non-prestressing PC strands as the longitudinal reinforcement. By experimental and numerical studies on the seismic performance of proposed columns, the following conclusions can be summarized.

(1) It is effective to adopt PC strands with high strength and low bond as the longitudinal reinforcement to make drift-hardening circular concrete columns if the anchorage at both ends of the columns is sufficiently provided.

(2) When under axial load ratio of 0.15, the use of PC strands can enhance the drift-hardening capability of the columns with shear span ratio of 3 and 4 not less than 6% drift. Even under a high axial load ratio of 0.35, the proposed method also can maintain the drift-hardening capacity up to 5% drift for the column with the shear span ratio of 3.0.

(3) The residual drift of the proposed concrete columns has been left around 0.5% at the drift of 2%, which satisfies the certain recoverability limit by the FEMA-P58-1 (FEMA 2012) fairly well.

(4) A simple and effective nonlinear analysis method considering the bond-slip effect to predict the lateral response of the concrete columns has been presented. The maximum bond strength $\tau = 2$ MPa is recommended for the proposed concrete columns with the
concrete compressive strength of 25 to 35 MPa.

(5) Partial confinement by BST has been verified useful to enhance the drift-hardening capability of the proposed concrete columns. Besides, it is also an effective method to reduce the damage of concrete shell within the potential plastic hinge.

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References

ACI, (2014). “Building code requirements for structural concrete (ACI 318-14).” Farmington Hills, Michigan, USA: American Concrete Institute.

Billington, S. L. and Yoon, J. K., (2004). “Cyclic response of unbonded posttensioned precast columns with ductile fiber-reinforced concrete.” Journal of Bridge Engineering, 9(4), 353-363.

Cai, G., (2014). “Seismic performance and evaluation of resilient circular concrete columns.” Thesis (PhD). Kobe University.

Cai, Z. K., Wang, Z. and Yang, T. Y., (2018). “Experimental testing and modeling of precast segmental bridge columns with hybrid normal and high-strength steel rebars.” Construction and Building Materials, 166, 945-955.

China GB/T 5224-2014, (2014). “Steel strand for prestressed concrete.” Chinese National Standard.

China GB/T 50010-2010, (2010). “Code for design of concrete structures.” Chinese National Standard.

Chopra, A. K., (1995). “Dynamics of structure: theory and applications to earthquake engineering.” New Jersey, USA: Prentice-Hall.

Fahmy, M. F., Wu, Z., Wu, G. and Sun, Z., (2010). “Post-yield stiffnesses and residual deformations of RC bridge columns reinforced with ordinary rebars and steel fiber composite bars.” Engineering Structures, 32(9), 2969-2983.

FEMA, (2012). “Seismic performance assessment of buildings, volume 1 methodology (FEMA P-58-1).” Washington, USA: Applied Technology Council and Federal Emergency Management Agency.

Fujino, Y., Hashimoto, S. and Abe, M., (2005). “Damage analysis of Hanshin expressway viaducts during 1995 Kobe earthquake I: residual inclination of reinforced concrete piers.” Journal of Bridge Engineering, 10(1), 45-53.

Funato, Y., Sun, Y., Takeuchi, T. and Cai, G., (2012). “Modeling and application of bond behavior of ultra-high strength bars with spiraled grooves on the surface.” JCI Proceedings, 34(2), 157-162. (in Japanese)

Guerrero, H., Ruiz-Garcia, J. and Ji, T., (2017). “Residual displacement demands of conventional and dual oscillators subjected to earthquake ground motions characteristic of the soft soils of Mexico City.” Soil Dynamics and Earthquake Engineering, 98 (2017), 206-221.

Ichiki, T., Hosoi, K. and Nakatsuka, T., (2002). “Fundamental study on bond property between grout and prestressing steels.” In: Proceedings of the 1st fib Congress, Osaka, Japan 13-19 October 2002. Lausanne, Switzerland: Fédération Internationale du Béton, 1, 89-90.

Iemura, H., Takahashi, Y. and Sogabe, N., (2004). “Development of unbonded bar reinforced concrete structure.” In: Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, Canada 1-6 August 2004. Vancouver: WCEE Secretariat, Paper No. 1537.

Kawashima, K., MacRae, G. A., Hoshikuma, J. I. and Nagayako, K., (1998). “Residual displacement response spectrum.” Journal of Structural Engineering, 124(5), 523-530.

Mahin, S., Sakai, J. and Jeong, H., (2006). “Use of partially prestressed reinforced concrete columns to reduce post-earthquake residual displacements of bridges.” In: Fifth National Seismic Conference on Bridges and Highways, San Mateo, California September 18-20 2006. Buffalo, New York, USA: Multidisciplinary Center for Earthquake Engineering Research, Paper No. B25.

Mander, J. B., Priestley, M. J. N. and Park, R., (1988). “Theoretical stress-strain model for confined concrete.” Journal of Structural Engineering, 114(8), 1804-1826.

Marriott, D., Pampamin, S. and Palermo, A., (2009). “Quasi-static and pseudo-dynamic testing of unbonded post-tensioned rocking bridge piers with external replaceable dissipaters.” Earthquake Engineering and Structural dynamics, 38(3), 331-354.

Menegotto, M. and Pinto, P. E., (1973). “Method of analysis for cyclic loaded R. C. plane frame including changes in geometry and non-elastic behaviour of elements under combined normal force and bending.” In: Proceedings of IAEG symposium on resistance and ultimate deform ability of structures acted on by well defined repeated loads, Lisboa, Portugal 13-14 September 1973. Zurich: International Association for Bridge and Structural Engineering, 11, 15-22.

Neuenhofer, A. and Filippou, F. C., (1997). “Evaluation of nonlinear frame finite-element models.” Journal of Structural Engineering, 123(7), 958-966.

Ou, Y. C., Wang, P. H., Tsai, M. S., Chang, K. C. and Lee, G. C., (2010). “Large-scale experimental study
of precast segmental unbonded post-tensioned concrete bridge columns for seismic regions.” Journal of Structural Engineering, 136(3), 255-264.

Pandey, G. R. and Mutsuyoshi, H., (2005). “Seismic performance of reinforced concrete piers with bond-controlled reinforcements.” ACI Structural Journal, 102(2), 295-304.

Pettinga, D., Christopoulos, C., Pampanin, S. and Priestley, N., (2007). “Effectiveness of simple approaches in mitigating residual deformations in buildings.” Earthquake Engineering and Structural Dynamics, 36(12), 1763-1783.

Priestley, M. J. N. and Tao, J. R., (1993). “Seismic response of precast prestressed concrete frames with partially debonded tendons.” PCI Journal, 38(1), 58-69.

Priestley, M. J. N., (1991). “Overview of PRESS research program.” PCI Journal, 36(4), 50-57.

Priestley, M. J. N., Siritaran, S., Conley, J. R. and Pampania, S., (1999). “Preliminary results and conclusions from the PRESS five-story precast concrete test buildings.” PCI Journal, 44(6), 42-67.

Ruiz-Garcia, J. and Miranda, E., (2003). “Inelastic displacement ratios for evaluation of existing structures.” Earthquake Engineering and Structural Dynamics, 32(8), 1237-1258.

Ruiz-Garcia, J. and Miranda, E., (2010). “Probabilistic estimation of residual drift demands for seismic assessment of multi-story framed buildings.” Engineering Structures, 32(1), 11-20.

Shim, C. S., Chung, C. H. and Kim, H. H., (2008). “Experimental evaluation of seismic performance of precast segmental bridge piers with a circular solid section.” Engineering Structures, 30(12), 3782-3792.

Shim, C., Lee, S., Park, S. and Koem, C., (2017). “Experiments on prefabricated segmental bridge piers with continuous longitudinal reinforcing bars.” Engineering Structures, 132, 671-683.

Song, L. L., Guo, T., Gu, Y. and Cao, Z. L., (2015). “Experimental study of a self-centering prestressed concrete frame subassembly.” Engineering Structures, 88, 176-188.

Sun, Y., Cai, G. and Takeshi, T., (2013). “Seismic behavior and performance-based design of resilient concrete columns.” Applied Mechanics and Materials, 438, 1453-1460.

Sun, Y. and Fukuhara, T., (2005). “Development of high seismic performance concrete frames.” In: H. G. Russel Ed. Proceedings of the 7th International Symposium on the Utilization of High Strength/High-Performance Concrete (ACI SP-228), Washington, USA 20-24 June 2005. Farmington Hills, Michigan, USA: American Concrete Institute, 615-632.

Sun, Y., Fukuhara, T. and Kitajima, H., (2006). “Analytical study of cyclic response of concrete members made of high-strength materials.” In: Proceedings of the 8th U. S. National Conference on Earthquake Engineering, San Francisco 18-22 April 2006. Oakland, California, USA: Earthquake Engineering Research Institute, Paper No. 1581.

Sun, Y. and Sakino, K., (2000). “Simplified design method for ultimate capacities of circularly confined high-strength concrete columns.” ACI Special Publication SP-193, 571-585.

Sun, Y. and Takeuchi, T., (2013). “Fundamental study on seismic behavior of resilient concrete columns.” JCI Proceedings, 35(2), 1501-1506. (in Japanese)

Sun, Y., Takeuchi, T., Funato, Y. and Fujinaga, T., (2012). “Earthquake-resisting properties and evaluation of high performance concrete columns with low residual deformation.” In: Proceedings of the 15th World Conference on Earthquake Engineering, Lisbon, Portugal 24-28 September 2012. Lisboa: Sociedade Portuguesa de Engenharia Sismica, 1442-1451.

Tada, T. and Takeda, J., (1985). “Analysis of bond deterioration process in reinforced concrete member: (part 1) Analytical model, method and examples.” Journal of Structural and Construction Engineering (Transactions of AJS), 351, 22-30. (in Japanese)

Tavallali, H., Lepage, A., Rautenberg, J. and Pujol, S., (2011). “Cyclic response of concrete frame members reinforced with ultrahigh strength steel.” In: D. Ames, T. Droessler and M. Hoit Eds. Proceedings of Structures Congress 2011, Las Vegas 14-16 April 2011. Reston, Virginia, USA: American Society of Civil Engineers, 560-570.

Taran-Gilmore, A. and Virtor-Cambray, N., (2009). “Preliminary design of low-rise buildings stiffened with buckling-restrained braces by a displacement-based approach.” Earthquake Spectra, 25(1), 185-211.

Todd, D., Carino, N., Chung, R. M., Lew, H. S., Taylor, A. W., Walton, W. D., Cooper, J. D. and Nimis, R., (1994). “1994 Northridge Earthquake: Performance of structures, lifelines, and fire protection systems.” NIST SP-862, Gaithersburg, Maryland, USA: National Institute of Standards and Technology.

Watanabe, F. and Osumi, K., (1991). “Improvement of flexural behavior of reinforced concrete sections by combined use of different grade longitudinal bars.” ACI Structural Journal, 128, 927-940.

Yuan, W., Zhao, J., Sun, Y. and Zeng, L., (2018). “Experimental study on seismic behavior of concrete walls reinforced by PC strands.” Engineering Structures, 175, 577-590.

Zatar, W. A. and Mutsuyoshi, H., (2002). “Residual displacements of concrete bridge piers subjected to near field earthquakes.” ACI Structural Journal, 99(6), 740-749.
Notations (SI units):

- $a$ = shear span of the specimen.
- $a/D$ = shear span ratio of the specimen.
- $A_g$ = total area of the column section.
- $D$ = diameter of the specimen.
- $E_c$ = Young’s modulus of concrete.
- $E_s$ = Young’s modulus of steel.
- $f'_c$ = measured standard cylindrical concrete compressive strength.
- $f'_{cc}$ = compressive strength of core concrete.
- $f'_{res}$ = crushing strength of concrete.
- $f_y$ = yield strength or nominal yield of rebar and/or steel plate.
- $f_{su}$ = ultimate strength of rebar and/or steel plate.
- $n$ = axial load ratio [$= P(f'_c A_g)$].
- $P$ = applied axial load.
- $R$ = drift of the specimen ($\Delta/a$).
- $V$ = applied lateral load.
- $\Delta$ = measured lateral displacement of the loading point.
- $\varepsilon_0$ = compressive strain of plain concrete at $f'_c$.
- $\varepsilon_{cc}$ = compressive strain of core concrete at $f'_{cc}$.
- $\varepsilon_{20}$ = crushing strain of concrete at $f'_{res}$.
- $\varepsilon_k$ = strain of the rebar in the $k$-th segment.
- $\rho_{cr}$ = ultimate strain of rebar and/or steel plate.
- $\varepsilon_y$ = yield strain or nominal yield strain of rebar and/or steel plate.
- $\rho_l$ = steel ratio of the longitudinal reinforcement [$= A_d / A_g$].
- $\rho_v$ = volumetric ratio of hoops.
- $\phi$ = curvature of the plastic hinge region.
- $\Phi$ = elongation rate of steel.
- $\tau$ = the maximum bond strength between rebar and concrete.
- $\tau_k$ = bond strength of the rebar in the $k$-th segment.