Lateral cyclic behavior of bridge columns confined with pre-stressed shape memory alloy wires

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ABSTRACT
This study focuses on investigating the effects of the wrapping mode of shape memory alloy (SMA) spirals with a large pre-strain on the lateral cyclic behavior of the bridge columns. The seismic performance of RC columns confined with different NiTiNb SMA spirals spacing and wrapping heights is compared through a series of quasi-static tests. The confined columns are tested under compression and cyclic lateral loadings. The lateral cyclic behavior of confined columns is compared with that of the as-built column. The test results demonstrate that NiTiNb SMA with a large pre-strain significantly improves the ductility and energy dissipation capacity, and has a slight effect on the peak lateral strength of the bridge piers. The SMA spirals reduce the crushing concrete zone and delay the buckling of steel bars in spite of the crack range in the SMA specimens increasing compared with the control column. The reinforcement effect increases as the SMA spirals spacing decreasing, particularly in terms of ductility and energy dissipation. The damage pattern has a slight distinction in the specimens strengthened by different wrapping mode.

1. Introduction
The collapse of the Morandi Bridge in Italy affirmed that the majority of old bridges in the world have remained in service for a lengthy period (Zordan 2018). The capacity of bridge columns, particularly to resist strong seismic ground motions, deteriorates progressively as the service time increases. The repair and retrofitting of old bridges to enhance their seismic capacity has become an important issue. Numerous cases of bridge collapse under earthquakes have demonstrated that old bridge columns were built with short lap splices in the developed plastic hinge zone of columns and insufficient transverse reinforcement, resulting in inadequate ductility and shear capacity (Ma et al. 2017).

Strengthening the lateral concrete confinement in the plastic hinge zone of columns using passive and active confinement techniques is an effective method for enhancing the ductility and shear capacities of old bridges. Passive confinement methods, such as the use of internal transverse steel reinforcement, external steel jackets (Thermou, Katakalo, and Manos 2018;...
Harajli and Hantouche 2015; Nematzadeh et al. 2017), and fiber-reinforced polymer (FRP) jackets (Vuggumudi and Alagusundaramoorthy 2018; Shakir, Guan, and Jones 2016; Faustino, Frade, and Chastre 2016; Tastani et al. 2013), have been widely explored, and used in both new and existing structures. The confinement pressure is exerted by hoop stresses developing in the lateral reinforcement as a result of concrete lateral dilation under loading. Therefore, lateral reinforcement can only be achieved following lateral concrete dilation. This drawback is avoided in active confinement methods, owing to the active confining pressure applied on the concrete prior to loading.

Previous research has demonstrated that the effect of active confinement is superior to that of passive confinement (Ghadami and Nematzadeh 2018). In recent years, shape memory alloy (SMA) has been integrated into structures as an active, semi-active or passive component to reduce the risk of damage caused by ground motions owing to its shape memory effect (SME) and superelasticity (SE) (Song et al. 2000; Qian, Li, and Song 2016; Song et al. 2011). The unique mechanical properties of SME and SE make it easy for providing active confining pressure on reinforced concrete (RC) columns to increase the strength and ductility of concrete structures.

The confining effectiveness of SMAs in increasing the ductility of RC columns under uniaxial compressive load have been concentrated on strain-stress behavior and recovery stress. Andrawes and Shin (2008; Andrawes et al. 2010) studied the feasibility of using SMA rings to apply confining stress on concrete columns by analytical analysis and experimental test. They compared the stress-strain curves of SMA wires confined concrete cylinders under compression loading with unconfined and fiber-reinforced polymers (GFRP) wrapped cylinders. Choi et al. (2008, 2010a, 2013a, 2014) conducted lots of tests to research the recovery and residual stresses of SMA wires wrapped on concrete cylinders. Tran, Balandraud, and Destrebecq (2015) compared the failure modes and stress-strain diagrams of active and passive confinements by SMA wires through crush tests. In addition, the uniaxial cyclic stress-strain behavior of concrete cylinders confined by SMA wires was also investigated (Park et al. 2011; Chen and Andrawes 2017). The stress deterioration ratio and stress-strain envelopes were tested. These studies proposed new jacketing methods and revealed the working mechanism of SMA wire jackets as an active confining method under uniaxial compression loading. However, the bridge piers suffer bending loading as well as axial compression loading when the bridge is subjected to a strong ground motion. The stress state of piers under uniaxial compression loading is quite different from realistic seismic loadings. The bending behavior of bridge piers jacketed by SMA wires under complex loadings should be further evaluated.

A great research effort has been made to evaluate the flexural ductility and shear capacity of RC columns confined by SMA wires through quasi-static cyclic tests. Shin and Andrawes conducted a series of quasi-static experiments to investigate the efficacy (Shin and Andrawes 2010a, 2010b, 2011). These studies illustrated that SMA spirals have a significant increase in the flexural ductility and energy dissipation capability of RC columns compared to those of as-built columns and columns retrofitted with GFRP. They also analyzed the lateral cyclic behaviors of the columns under different confining pressures by numerical analyses (Andrawes et al. 2010; Shin and Andrawes 2014). Choi et al. (2010b, 2012) compared the effect of the NiTiNb and NiTi SMA wires on the flexural strength and ductility of the lap-spliced RC columns under axial load and lateral load. Choi et al. (2013b) also developed the fragility curves for RC columns confined by SMA wires using developed analytical models based on the experimental results. In these quasi-static tests, the SMA wires jackets increased. The jacket height was 400 mm and the pitch of the wires was 2.0 mm. required 250 m to wrap a column. They found that the SMA jackets do not provide RC columns with adequate lateral confinement if the volumetric ratio of SMA is small. Further research is required to verify this observation.

This study aims to research the effects of the pitches and wrapping heights of the SMA wires on the seismic performance of RC columns, to guide the design of retrofitting the bridges piers using SMA wires. For this purpose, the hysteretic curves of the specimens confined with different wire pitches were measured by quasi-static cyclic tests. The strength degradation and duality of these specimens were compared to assess the effectiveness of the SMA wire pitch and height.

2. Mechanical properties of NiTiNb SMA

The SME and SE of the SMA are attributed to the transformation of two crystals (martensite and austenite phases) in the alloy, which is governed by four temperatures, namely the martensite finish (Mf), martensite start (Ms), austenite start (As), and austenite finish (Af) temperatures. The four phase transition temperatures are not constant, and will be altered by the influence of stress (Tran, Balandraud, and Destrebecq 2015). The SME refers to the ability of the SMAs to recover their reference shapes, even after experiencing relatively large deformations (up to 8% strain) when heating above Af, as illustrated in Figure 1 (Vokoun, Kafka, and Hu 2003). If the deformation is confined, a high recovery stress will be generated in the alloy; this is the principle of SMA active confinement. Superelasticity indicates that the SMA is loaded to a relatively large strain (up to 8%) in the pure austenitic state and can be completely restored to its reference shape following unloading. However, once the strain exceeds 8%, the
alloy thermomechanical properties change (Brena and Schlick 2007) For example, as illustrated in Figure 2, when stretched to 16% strain in the pure austenite phase, the NiTiNb SMA wires cannot completely return to their reference shapes or even exhibit larger residual strain. The reason for this phenomenon is that the larger stress increases the alloy phase transition temperatures. This implies that the large increase in the phase transformation temperatures causes the alloy wires to no longer be completely in pure austenite phase. Thus, heating the alloy can still cause it to return to a certain strain, which is similar to the SME.

In this study, NiTiNb SMAs produced by Beijing Nonferrous Metals Co. Ltd were adopted. Their phase transition temperatures are As = −40°C, Af = −30°C, and Ms and Mf below −90°C, and at room temperature, the alloy is in its pure austenitic phase. Firstly, the SMA wires were tensioned to an initial strain of 16% (approximately 11% permanent strain) at room temperature by a tensioning device. Measurement and analysis carried out using differential scanning calorimetry indicated an increase in the transformation temperatures of the alloy wire after stretching, with As = 81°C, Af = 91°C, and Ms and Mf below −60°C. Thereafter, the pre-strained SMA wires were spirally wrapped in the plastic hinge zone, where confinement was desired. After anchoring the wires to keep them in place, the SMA spirals were heated by electrical power using a 23 A alternating current to achieve temperatures greater than 200°C. The deformed SMA spirals have a tendency to return to their original length, thus creating a hoop effect on the concrete at the constraint point, which forms the basis of active confinement pre-stressing.

3. Design of test specimens

3.1. Model description and design for reducing vibration measurement

To ensure structure design rationality, the test specimen size was determined according to the concrete column performance database prepared by several research institutions in Europe, America, Japan, and other countries. In this paper, five RC columns, named BP-01 to BP-05, with uniform dimensions and the same reinforcement were casted. As illustrated in Figure 3, the column height was 1050 mm, and its diameter was 300 mm with an 18 mm concrete cover. The dimensions of the footing were 1400 × 600 × 500 mm. The effective shear span of the lateral loading point was 900 mm from the bottom of the column, and the effective shear span ratio was 3 (with reference to the average shear span ratio of the pseudo-static test).

The concrete column specimens were made using a type of concrete C40, which indicates that the average compressive strength of a cylindrical sample of such class of concrete is 40 MPa. The C40 concrete is composed of the components water, cement, sand, and gravel with the weight portion 1:0.39:1.29:2.88. The maximum aggregate
size is 20 mm. Standard concrete cubes of 150 mm×150 mm×150 mm were poured and cured with the column specimens. The cubes compressive strength is 41.0 MPa under 28 days of outdoor curing conditions. The outdoor temperature was approximately 15°C during maintenance. Two types of steel bar, hot-rolled ribbed bars (HRB400) and hot-rolled plain steel bars (HPB300), were embedded in specimens to reinforce concrete as a tensioning device. For each type of steel bar, the number (e.g., 400 in HRB) denotes the yield strength used for design. More details about the mechanical properties of concrete and steel bars are list in Table 1.

For an improved reinforcement effect, the seismic performance of the reference specimen should have sufficient lifting space. The reinforcement followed the Guidelines for Seismic Design of Highway Bridges (JTG/T 002-01-2008), with 0.006 ≤ps≤ 0.04, where ρ is the longitudinal reinforcement ratio. Eight HRB400 longitudinal bars with a diameter of 12 mm were arranged evenly along the column circumference, of which the longitudinal reinforcement ratio (ρl) was 1.28%. HPB300 spiral stirrups with 8 mm diameter and 100 mm spacing were used. The volumetric stirrup ratio is 0.49%, which is slightly higher than the value in the guidelines.

The specifications of the strengthened specimens BP-01 to BP-05 are displayed in Table 1. The NiTiNb SMA wires with 16% pre-strain and tensioned at room temperature were adopted and their diameter was 1.9 mm. The SMA spiral used in each reinforced specimen consisted of segmental pre-strained wires connected by a U-shaped hoop. This was done because using a sufficient number of U-shaped hoops has proven to ensure that no slip occurs between two segments. From the contrast experiment, the recovery stress σ_{SMA} was observed to reach a value greater than 440 MPa when heating the alloy to 200°C by electric current. Referring to the study by Dommer and Andrawes (2012), a formula to determine the confining pressure using the active constraints of the SMA was proposed.

\[ p = \frac{2Aσ_{SMA}}{Ds} \]  

where \( A \) is the SMA wire cross-sectional area, \( D \) is the column diameter, and \( s \) is the SMA spiral spacing.

The confining pressures of SMA wires with different values for spacing can be obtained, as indicated in Table 2.

| Specimen | Volumetric ratio of SMA spiral (%) | Spiral spacing (mm) | Spiral height (mm) | Heating temperature (°C) | Active confining pressure P (MPa) |
|----------|---------------------------------|--------------------|-------------------|------------------------|-------------------------------|
| BP-01    | 0                               | 0                  | 0                 | 0                      | 0                             |
| BP-02    | 0.25                            | 15                 | 300               | 200                    | 0.55                          |
| BP-03    | 0.19                            | 20                 | 300               | 200                    | 0.42                          |
| BP-04    | 0.13                            | 30                 | 300               | 200                    | 0.28                          |
| BP-05    | 0.19                            | 20                 | 200               | 200                    | 0.42                          |
4. Test description and procedure

4.1. Test setup and instrumentation

Figure 4 illustrates the quasi-static test equipment and installed specimen. The specimen was fixed on the floor. The axial loading device is an electro-hydraulic servo actuator fixed on the top beam with sliding supports, which make the jack move along with the specimen in horizontal direction. A spherical hinge is fixed on the top of the specimen. The spherical hinge and sliding support make the axial load be in the gravity direction throughout the test. The electro-hydraulic servo actuator was hoisted on the top beam to avoid the influence of its gravity on the specimen. Therefore, the P-Delta ($P - \Delta$) effect is taken into consideration. A radial pressure sensor and displacement sensor set in the actuator were used to monitor the axial force and displacement at the loading point simultaneously, and they constituted the vertical acquisition system. An additional actuator and the same acquisition system were fixed on the reaction wall in the lateral direction. To ensure uniform axial pressure on the specimens, a spherical surface was set on the top of the columns. Moreover, the specimen footing was fixed on the ground with four vertical screws, and two screws were used in the lateral direction for connection to the reaction wall through a steel beam. All values measured by the sensors were recorded as raw data in real time by the computer, the recording frequency of which was 10 Hz.

The force acting on the key position of the specimen is reflected by the strain of the longitudinal reinforcement and stirrups in the plastic hinge zone. As indicated on the left of Figure 4, four strain gauges were arranged along the steel bar force direction, and their specification was $3 \times 2$ mm (grid length $\times$ grid width). The strain acquisition system was a Yangzhou Jingming wireless static strain acquisition system, the acquisition frequency of which was set to 10 Hz.

4.2. Loading scheme

The axial load on the top of the specimen was 284 kN, which was calculated based on the axial compression ratio mentioned above. It was applied on the specimen at a suitable rate prior to the test, and maintained constant during the test. Subsequently, a reverse cyclic lateral load was applied to the displacement control in a number of successive runs, in which the displacement amplitudes were set based on the calculated column heights. In each run, three full cycles of reverse lateral loading were applied. The cyclic loading regimen is illustrated in Figure 5 and Table 3. Loading for each specimen continued until either a load drops of $\pm 15\%$ of the maximum achieved load or obvious irreparable damage of the longitudinal bars and confined concrete was observed.

5. Test results and analysis

5.1. Test phenomena

The first lateral cracks started in the lower 50 mm of the control column specimen (BP-01) when the lateral drift ratio reached 0.44%. As the shear force on the column increased gradually, the cracks began to develop obliquely when the drift ratio reached 0.89%. After the drift ratio reached 1.33%, the number of cracks no longer increased significantly, and three to four main cracks was formed on the scope of the lower 0 to 400 mm of the column due to the longitudinal reinforcement entering the plastic stage. A vertical crack appeared at 2.22% drift ratio, and the cover concrete spalled off at 4.00% drift ratio. After the cover concrete spalling, the burden of core concrete and reinforcements to bear compressive stress increased as the lateral drift increased. Finally, when the drift ratio reached 5.78%, the failure of the column occurred owing to core-concrete crushing and longitudinal reinforcement buckling, thus the test was terminated.
Figure 5. Loading time history of lateral displacement.

For the columns retrofitted with SMA spirals (BP-02 to BP-05), they showed similar failure mode in general, but differed in detail. In general, the first lateral crack was all formed at 0.44% drift ratio. Four to five main cracks developed at 1.33% drift ratio, and the scope increased to 500 mm. Different from control column, additional lateral cracks at the column–footing interface of the four retrofitted columns were initiated at 0.89% drift ratio owing to the hoop action of SMA which increased the lateral stiffness of the column. Vertical cracks formed at 2.22% drift ratio propagated with increases in the lateral drift and connected the two lateral cracks at around 5.78% drift ratio. Specially, when the drift ratio reached 0.89% and 1.33%, the original horizontal cracks on BP-04 and BP-05 began to develop obliquely, while it was not obvious on BP-02 and BP-03 until the drift ratio reached 2.22%. Moreover, for BP-02, BP-03 and BP-04, the longitudinal reinforcements were fractured at 8.44% drift ratio, while it was occurred at 7.56% drift ratio for BP-04. The result was due to the fact that the high volume ratio of SMA helped the column resist shear more effectively, thus delaying and mitigating the damage.

Figure 6 illustrates the final damage pattern of each specimen, in which the curves with digits indicate the crack locations under the corresponding displacement amplitudes. As illustrated in Figure 6(a), a large zone of concrete falling-off existed in a range of 200 mm from the two sides at the bottom of the column, even exposing the reinforcement. However, owing to the hoop effect of the pre-stressed SMA, the final damage pattern of the strengthened columns, illustrated in Figure 6(b–e), was significantly lighter than that of the reference columns. Moreover, the crack range in the SMA specimens increased to almost 1.5 times that of the reference specimens, while the crack widths in the increased range were very small. This result was owing to the change in the lateral stiffness of the column plastic hinge zone by the pre-stressed SMA, which increased the scope of damage but decreases the damage degree. What’s more, the cracked cover concrete could share a burden of compressive loads and reduced the compressive stress of longitudinal reinforcements and core concrete in compression. This behavior delayed the crushing of core concrete and the buckling of reinforcements in compression.

5.2. Hysteresis curve and skeleton curve

Figure 7 presents the hysteretic curves of BP-01 to BP-05, where $F$ and $\Delta$ are lateral forces $F$ and displacements $\Delta$, respectively, and the drift ratio is the ratio of the lateral displacement $\Delta$ to effective height (900 mm) of column. From the figure, it can be observed that the curve of BP-01 exhibited a pinch phenomenon, which meant that its energy dissipation capacity and deformability were poor. The strength attenuation speed increased rapidly and the longitudinal bar was broken at a smaller displacement after reaching the ultimate strength. All of the above demonstrated its poor seismic

| Numbering | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
|-----------|---|---|---|---|---|---|---|---|---|----|----|
| Displacement amplitude (mm) | $\pm 2$ | $\pm 4$ | $\pm 8$ | $\pm 12$ | $\pm 20$ | $\pm 28$ | $\pm 36$ | $\pm 44$ | $\pm 52$ | $\pm 60$ | $\pm 68$ |
| Drift ratio (%) | $\pm 0.22$ | $\pm 0.44$ | $\pm 0.89$ | $\pm 1.33$ | $\pm 2.22$ | $\pm 3.11$ | $\pm 4.00$ | $\pm 4.89$ | $\pm 5.78$ | $\pm 6.67$ | $\pm 7.56$ |
| Loading rate (mm/min) | 6 | 10 | 14 | 20 | 28 | 36 | 44 | 52 | 66 | 76 | 86 |

Figure 6. Final damage patterns of test specimens: (a) BP-01; (b) BP-02; (c) BP-03; (d) BP-04; (e) BP-05.
However, the curves of the BP-02 to BP-05 specimens were fuller than the reference ones. In view of the ultimate strength, BP-02, BP-03, and BP-04 reinforced with three different spacing SMA wires exhibited decreasing trends, but the amplitude was lower. In order to better reflect and compare the behaviors of each specimen in different phases under cyclic loading, the skeleton curves are drawn, as illustrated in Figure 8. In general, the specimens were elastic at the initial loading stage, in which the lateral load increased linearly with displacement, and all the envelope curves essentially overlapped. The strengthening with the SMA wires contributed little to the initial specimen stiffness, which was listed in Table 3. With the increase in the lateral loading amplitude, cracks appeared and the curves tended to be gentle, indicating that the column stiffness began to decrease gradually. The curves began to separate at this phase. When the lateral load reached the maximum, the

Figure 7. Hysteretic curve for each specimen: (a) BP-01; (b) BP-02; (c) BP-03; (d) BP-04; (e) BP-05.
strength of BP-01 attenuated rapidly, while the other specimens exhibited an attenuation platform, exhibiting superior displacement ductility.

Figure 8(a) illustrates the influence of different spacings of the SMA wires on the skeleton curves of BP-02 to BP-04. Although the overall trends of the curves were very close, the curves of BP-02 enveloped those of BP-03 and then BP-04, which indicated that a smaller SMA spacing yielded a superior effect. While the skeleton curves of BP-02 and BP-03 were very close, the curve of BP-04 was further in the negative axis direction, and its limit displacement was less than those of the other two. Figure 8(b) illustrates the influence of different wrap heights of the SMA wires on the skeleton curves. In general, the skeleton curves of BP-03 and BP-05 were very close, but BP-05 was inferior to BP-03 in terms of the ultimate bearing capacity and ultimate displacement. The previous design indicated that the wrap heights were all greater than the column plastic hinge zone height. Therefore, the improvement of the skeleton curves was not obvious.

5.3. Ductility

Ductility is defined as the capability of a structural member to sustain inelastic deformations prior to failure, without a substantial strength loss. The ductility coefficient can be divided into the curvature, displacement, and angular ductility coefficients. The displacement and angular ductility coefficients are generally used to reflect the ductility of the component. Table 4 lists the ductility of each specimen averaged from its drift ratios. The displacement ductility is defined as the ratio of the ultimate displacement to the yield displacement. The yield displacement and ultimate displacement refer to the lateral displacements at 80% of ultimate load at the ascending and descending part of the skeleton curves, respectively (Tawfik, Badr, and ElZanaty 2014). The ultimate joint rotation is used to represent the angular ductility coefficient, which is the ultimate displacement divided by the calculation height of the specimen.

The initial stiffness of each specimen, as well as the maximum lateral load, was very close, while the ductility was quite different. In this paper, the initial stiffness was the slope between the origin and the peaks in load–displacement curves at 0.22% drift ratio, at which specimens were in the elastic stage. From the table, it is observed that the ductility coefficients of BP-02, BP-03, BP-04 and BP-05 were 1.29, 1.28, 1.14 and 1.25 times that of BP-01, respectively. Specimen BP-02 exhibited the highest ductility among all specimens, while BP-03 and BP-04, which had the same SMA spiral wrap heights but different spacings, were less ductile than BP-02. The superior ductility of specimen BP-02 may be attributed to the higher active confining stress provided by the more densely spaced SMA spirals. However, when comparing specimens BP-03 and BP-05, with the same spiral wrap spacing but different heights, it was observed that the higher wrap height exhibited superior ductility, although the improvement was very limited. Furthermore, the ductility of the strengthened specimens was significantly increased compared to that of the reference ones. Similar conclusions can be drawn in the analysis of the ultimate joint rotation.

| Specimen | Initial stiffness (kN/mm) | Maximum load (kN) | Yield displacement (mm) | Ultimate displacement (mm) | Ultimate joint rotation | Displacement ductility |
|----------|--------------------------|------------------|------------------------|---------------------------|------------------------|-----------------------|
| BP-01    | 18.28                    | 83.16            | 6.01                   | 41.11                     | 0.043                  | 6.45                  |
| BP-02    | 19.22                    | 87.19            | 7.68                   | 64.31                     | 0.067                  | 8.29                  |
| BP-03    | 18.83                    | 83.83            | 7.56                   | 63                        | 0.065                  | 8.24                  |
| BP-04    | 18.68                    | 82.44            | 7.84                   | 56.17                     | 0.061                  | 7.38                  |
| BP-05    | 18.26                    | 80.50            | 7.80                   | 62.69                     | 0.065                  | 8.08                  |
5.4. Strength degradation

Under the lateral cyclic load, the lateral strength and stiffness of the column decrease with an increase in the number of cycles due to the crack development in concrete and bending yield or even fracture of the reinforcement. The strength degradation of the component can be described by the strength degradation coefficient \( k \) and \( k' \). \( k \) is the ratio of the maximum peak load (first cyclic peak load) under a certain displacement amplitude to the maximum load under all displacement amplitudes. And \( k' \) is the ratio of the peak load of the following cycle loads to the first cyclic peak load under a certain displacement amplitude. \( k \) reflects the ability of the column to resist the cyclic action of different amplitudes of displacement, while \( k' \) reflects the ability to resist the same displacement amplitude. The closer the value is to 1, the smaller the strength deterioration of the component is. In the test, the lateral force reached the maximum value at 20 mm displacement amplitude (2.22% drift ratio), hence the strength attenuation coefficient started from this point, as illustrated in Figure 9. In Figure 9(b), \( k' \) of the third cyclic peak lateral load in the three cycles was analyzed due to its maximizing the strength degradation under the same displacement amplitude.

As illustrated in Figure 9, the degree of strength degradation for each specimen increased with the increase in the loading displacement amplitude. When drift ratio reached 4.00%, the strength of BP-01 decreased obviously, and its \( k \) and \( k' \) was 0.86 and 0.87, respectively, exhibiting the worst performance. The reason for this result was that under this drift ratio, the cover concrete spalling off resulted in the decrease of the compressive capacity of the control column, which reduced its lateral bearing capacity. However, whether from \( k \) or from \( k' \), the strength degradations of retrofitted columns were all no more than 5% at this point due to the hoop effect of SMA. Although the cover concrete partially cracked and crushed, it still participated in burdening compression under the hoops. In Figure 9(a), although the curves of retrofitted columns were very close in general, specimens with larger SMA volumetric ratio still presented smaller stiffness degradation. In Figure 9(b), it was obvious that BP-02 (0.25 volumetric ratio) and BP-03 (0.19 volumetric ratio) were superior to BP-04 (0.13 volumetric ratio) after 3.11% drift ratio, and BP-02 was superior to BP-03 after 6.67% drift ratio. Therefore, we can conclude that the larger the volumetric ratio of SMA spirals, the slower the degradation of column strength. However, the influence of the wrap height on the specimen strength degradations could almost be ignored due to the fact that the curves of BP-03 and BP-05 almost coincided illustrated in Figure 9(a), and the same thing happened in Figure 9(b).

5.5. Stiffness degradation

The calculation of the lateral effective stiffness \( K_e \) is essential to develop a new hysteretic model for RC columns retrofitted with SMA spirals in the future. It was estimated as the slope between the origin and the positive peaks in load–displacement curves at a given drift ratio. The average effective stiffness values of the three cycles under the amplitude shift were taken as the representative effective stiffness values, illustrated in Figure 10. It is observed from the curves that the stiffness degradations of all specimens obey a fast-to-slow change law and the overall stiffness degradation levels are very similar. Comparing the stiffness curves of each reinforced specimen, the difference between them was less than 5%. Although the effective stiffness of control specimen was slightly lower than that of reinforced specimens during the whole loading process, it was very limited. For example, this result was
Figure 10. Comparison of specimen stiffness degradations: (a) effective stiffness; (b) effective stiffness degradation with lateral drift.

more obvious before 2.22% drift ratio, while the initial stiffness of BP-02 (19.22 kN/mm) was only 7.25% higher than that of BP-01 (17.92 kN/mm). This may be due to the relatively large original stiffness of RC column, which makes the lifting effect of SMA spirals on its effective stiffness not obvious, and the difference between SMA spirals with different spacing and height was more difficult to reflect. Thus, the spacing and wrap height of the SMA spirals used in this paper had little influence on the stiffness degradation of the RC column. Conversely, owing to the less change on the column stiffness, the stress of the structure was almost unchanged, which would be conducive to the protection of other components of the structure. Moreover, it will benefit to the establishment of numerical modeling.

Previously, Brena and Schlick (2007) used displacement ductility to describe the relationship of the effective stiffness of the RC columns, while Choi et al. (2012) improved the equation by replacing displacement ductility with drift ratio to better fit his test data. The improved method suggested by Choi et al. (2012) to estimate effective stiffness as shown below

$$\frac{K_e}{K_0} = 1 \quad \text{for} \quad DR \leq 1 \quad (a)$$
$$\frac{K_e}{K_0} = \frac{1}{DR} \quad \text{for} \quad DR > 1 \quad (b)$$

where $K_0$ is effective stiffness $K_e$ at 1.0% drift ratio; and $DR$ is drift ratio of the system.

Although they concluded that Equation 2 was not in line with the stiffness attenuation after the failure point, but more applicable for elastic–perfectly-plastic systems, it also provides a way of thinking for this study. Generally, $DR$ is considered as the drift ratio of yield point, and $K_0$ is the corresponding yield stiffness. In this study, the yield points of BP-01 to BP-05 were 0.67%, 0.81%, 0.79%, 0.83% and 0.81% drift ratio, respectively, which were all in the range of 0.5% to 1% drift ratio. The similar results were found in the reference (Choi et al. 2012). Illustrated in Figure 10(a), the difference of stiffness curves between specimens decreases with the increase of lateral drift. Thus, 1% drift ratio was used as $DR$ in this paper, too. In Figure 10 (b), specimen BP-01 showed a slight deviation after 1% drift ratio and it crossed the reference curve, exhibiting the worst fitness. This is because the smaller yield stiffness of BP-01 led to a larger $K_e/K_0$ value. When the drift ratio exceeded 3.11%, the concrete fell off seriously and the lateral stiffness loses seriously, which finally led to the smaller ratio. For retrofitted specimens, the stiffness degradation curves showed a slight deviation especially after 4.89% drift ratio, where the concrete damaged seriously and the strength lost serious.

5.6. Accumulative energy dissipation

Hysteretic energy dissipation is an important index for the seismic performance evaluation of components, which reflects the ability to resist external energy dissipation from seismic loads. The parameters that measure the specimen hysteretic energy dissipation consist of the area contained in the hysteresis loop of the force-displacement hysteretic curve. A larger area results in a higher energy dissipation capacity of the specimen. Figure 11 illustrates the variation curves of the accumulative energy dissipation ($E_{hyp}$) for each specimen with the loading displacement.

The accumulative energy dissipation of all specimens followed the slow-to-fast trend. During the initial period, the column specimens were still in the elastic stage, during which the degree of damage was lighter, the specimen hysteretic energy was at a lower level, and the $E_{hyp}$ gradually increased. After entering the plastic stage, with the increase in the displacement amplitude and loading cycle frequency, the deformation degree of
the columns became increasingly obvious. During this process, because of the plastic deformation of the concrete, steel bars, and SMA spirals, the accumulative energy dissipation curve of the specimen began to grow steadily. To facilitate comparison, the damage point was defined as the displacement amplitude corresponding to the cyclic fatigue during the first fatigue fracture. The final values for the accumulative energy dissipation of BP-02, BP-03, BP-04, and BP-05 were 118.29, 114.00, 63.53, and 108.14 kNΔm, respectively. Compared to the BP-01 value of 40.13 kNΔm, these increased 2.95, 2.84, 1.58, and 2.70 times, respectively. The curve growth trend almost coincided with the different wrap heights. From Figure 11(a), it was observed that the trends of the accumulative energy dissipation curve for each specimen were almost identical, but the final values exhibited a difference, in that the final values of BP-02 and BP-03 were very close and greater than that of BP-04. According to Figure 11(b), the curve growth rate of BP-05 was slower than that of BP-03, but it was very limited.

5.7. Residual drift

The residual displacement is an important index for evaluating the postearthquake functionality in bridges, and can determine whether or not a bridge remains usable following an earthquake (Lee and Billington 2010). In this paper, the residual displacement Δr refers to the column deformation that is not restored when the lateral load at the column top decreases to zero, and its value is the coordinate abscissa value at the intersection of the hysteretic curve and lateral axis. After three cycles of loading displacement amplitudes, the three corresponding residual displacements were generated, and the mean values were taken as the residual displacements under the displacement amplitude, as illustrated in Figure 12.

When the lateral drift ratio was less than 0.89%, the specimens were essentially in the elastic stage, and their repositioning ability was strong. The residual displacement of each specimen was not more than 1.25 mm (0.14% drift ratio) during this stage, which could practically be ignored. After 1.33% drift ratio, the
residual displacement curve slope increased sharply due to the fact that the columns entered the yield stage. With the specimen entering plastic working state, the column lateral stiffness decreased, while the residual displacement increased markedly. As illustrated in Figure 12(a), the final residual displacement of BP-01 is 33.36 mm (residual drift ratio was 3.71%) under 4.89% drift ratio, and the corresponding values of BP-02, BP-03, and BP-04 are 29.40, 29.95, and 30.85 mm (3.27%, 3.33% and 3.43% drift ratio), respectively. SMA spirals induced stresses when concrete started dilating. The extra confinement pressure applied on the concrete restrained concrete dilation and delayed the damage of concrete. Therefore, the residual displacements of specimens strengthened by SMA spirals were obvious lower than that of the control specimen at the same displacement amplitude. Figure 12(a) shows that the wrap spacing of SMA has less effect on the residual displacement of specimens. Figure 6 reveals that the damage observer in the confined specimens at the end of the test were similar. The contribution of SMA spirals was in limiting concrete crushing. The crushed concrete size of the specimen with fine wrap SMA wires was smaller compared with the coarse wrapped specimens. Therefore, the confined columns exhibited a slight increase in residual displacement with the winding spacing increasing. On the other hand, the area of damage in confined columns was limited in the bottom compared to BP-01. The damage was not extended to the minimum wrap height, which caused the influence of the SMA spiral wrap height on the specimen residual displacement was small. The difference between BP-03 and BP-05 residual displacements was less than 1 mm (0.11% drift ratio) as presented in Figure 12(b).

6. Conclusions and discussion

This paper aimed to study the cyclic behavior of bridge columns actively confined with large-strain pre-stressed SMA spirals, which can be conveniently stretched at room temperature and stimulated by current. Quasi-static experimental tests using five specimens with different strengthened parameters were designed and carried out. It was confirmed that large-strain pre-stressed SMA spirals can provide effective restoring stress to reinforce piers.

Owing to the hoop effect, SMA spirals did not just effectively protect the concrete from crushing but also contributed to delaying progressive damage in the reinforcing bars under successive cyclic excitations. Considering the high price of SMA at present, it may be possible to reinforce the column together with FRP in practical application in order to further reduce the concrete damage at the plastic hinge of the column.

The deformability and accumulative energy dissipation of the column strengthened with SMA spirals are remarkably improved. The results of the specimens with smaller wrapping spacing (SMA volumetric ratio) are superior to those with relatively large spacing. For the strength degradation and residual displacement of the specimens, the strengthened specimens are obviously superior to the as-built column. However, the improvement of the specimens with different pitch spacing and wrapping height is very small. And SMA spirals contribute little to the lateral stiffness degradation of columns. This may be a good result. Owing to the less change on the column stiffness, the stress of the structure was almost unchanged, which would be conducive to the protection of other components of the structure.

It is obvious that as long as the winding range of SMA spirals covers the plastic hinge area of the column, the difference of columns seismic performance between different wrap heights is very small. Considering the influence of failure mode, the wrap height slightly higher than the height of plastic hinge zone in this paper is recommended. From the changing trends of deformability and accumulative energy dissipation, it seems that there is a suitable SMA wrapping spacing, and thus the ductility and energy dissipation of the bridge columns may be optimally improved, which requires further exploration by means of numerical method or experimental test. In addition, the dynamic characteristics of the SMA confined columns and ground motions are neglected in quasi-static tests, shaking table tests and nonlinear dynamic analysis are required to assess the seismic performance of the SMA jacketed bridge piers.

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Author contributions

Dr. Pan proposed the method and designed the experiments, Mr. Yue conducted the experiments, Mr. Hui analyzed the results, and Dr. Fan aided in writing and proofreading the paper. Professional editing and English corrections were also carried out.

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