Effect of the concrete cover thickness ratio on the post-yield stiffness of bridge columns with partially unbonded unstressed steel strands

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Abstract
A new self-centering concrete bridge column has been developed by the authors. The proposed bridge column uses unstressed partially unbonded seven-wire steel strands as elastic elements to reduce the residual displacement of the column after a strong earthquake. This research aimed to study the effect of concrete cover thickness ratio on the cyclic behavior of the proposed column. Four large-scale column specimens were tested using lateral cyclic loading. One column was the conventional concrete bridge column. The other three columns were the proposed self-centering bridge columns with varying concrete cover thickness ratios. Test results showed that partial unbonding effectively prevented the strands from yielding. The proposed columns showed post-yield stiffness ratios higher than the conventional column. The concrete cover thickness ratio did not significantly influence the hysteretic energy dissipation and the strain responses of longitudinal reinforcement. However, it had a significant impact on the post-yield stiffness ratio. The post-yield stiffness ratio of the proposed column tended to be inversely proportional to the concrete cover thickness ratio. A relationship was proposed between the concrete cover thickness ratio and the post-yield stiffness ratio for the preliminary design of the proposed column. Based on the relationship, the cover concrete thickness ratio should not exceed 5.1% to achieve a post-yield stiffness ratio of at least 5%, as recommended in the literature to control the residual displacement of a column.

Keywords: Columns, Near-fault ground motions, Residual displacement, Self-centering, Post-yield stiffness, Concrete cover

1 Introduction
Taiwan experiences frequent seismic activity and has a high population density. According to data released by the Central Geological Survey of the Ministry of Economic Affairs of Taiwan (Central Geological Survey 2019), approximately one-third of the population of Taiwan lives within 10 km of an active fault and is threatened by near-fault earthquakes. Earthquake observations have revealed that the ground motion displacement and velocity near a fault rupture zone are considerably different.
from those observed far from this zone (Somerville et al. 1997 and Abrahamson et al. 1998). In a near-fault region, a large displacement of the ground in one direction and high-speed pulses over a short time are typically observed (Kalkan et al. 2006). Structures cannot efficiently dissipate an earthquake’s energy by swinging, and conventional reinforced concrete (RC) bridge columns might exhibit large residual displacement after an earthquake, which substantially reduces the safety and serviceability of the bridge (Phan et al. 2006).

To reduce the residual displacement of bridge columns, the use of prestressing for the post-earthquake self-centering of bridge columns has been extensively studied. Hewes (2002) examined the post-tensioning of precast segmental columns. During an earthquake, the period of the column was increased by opening and closing the column segments. After the earthquake, self-centering was achieved through gravity load and unbonded post-tensioning. To reduce the damage to the plastic hinge region, the bottom segment of the column was confined by a steel plate. Chou et al. (2006) placed steel energy-dissipating devices at the base of post-tensioned concrete-filled steel tubular segmental columns to improve the energy dissipation behavior. Ou et al. (2007, 2010a and 2010b) developed precast segmental unbonded post-tensioned concrete bridge columns for regions with strong earthquakes and proposed to use energy dissipation steel bars passing through segmental interfaces to enhance energy dissipation. To avoid the premature fracturing of the energy dissipation bars because of the opening and closing of segmental interfaces during an earthquake, the bars were partially unbonded, or high-performance bars were used. Test results revealed that if the energy dissipation bars were bonded, they would fracture when the drift ratio of the column was 3%. If the energy dissipation bars were partially unbonded or high-performance bars were used, the fracture of the bars could be considerably delayed to a drift ratio of 6%, and the residual drift could be controlled to be within 1%. Test results showed that the bridge columns developed by Ou et al. were ductile, and had good energy dissipation and self-centering capability. Although prestressing has been proved to be an efficient tool for reducing the residual displacement, using this tool is challenging because of the high cost of prestressing operation and the issue of concrete creep due to prestressing.

Another method of reducing the residual displacement is to increase the post-yield stiffness ratio of bridge columns; that is, the ratio of the post-yield stiffness to the initial stiffness of the columns. Kawashima et al. (1998) found that if the post-yield stiffness ratio is higher than 5%, a bridge column exhibits a small residual displacement after a strong earthquake. Moreover, the residual displacement increases sharply as the post-yield stiffness ratio approaches zero. The post-yield stiffness ratio of conventional RC bridge columns is typically close to zero. To increase the post-yield stiffness ratio of bridge columns, Iemura et al. (2004 and 2006) and Yamanobe et al. (2008) added unbonded high-strength bars to RC bridge columns. The bars were high-strength and unbonded to the concrete to remain elastic during an earthquake. The post-yield stiffness ratio of the column was increased, and the residual displacement of the columns was reduced. Fahmy et al. (2010) wrapped the longitudinal steel reinforcement of RC bridge columns in fiber-reinforced polymer to improve the post-yield stiffness of the reinforcement and the column. Ibrahim et al. (2017) further wrapped the transverse steel reinforcement with fiber-reinforced polymer in addition to the longitudinal
reinforcement. Test results showed that using a longer unbonded length for the longitudinal reinforcement can decrease the residual displacement of the column.

Liu et al. (2021) used 1200-MPa ultra-high-strength (UHS) steel reinforcing bars with low bonding force to design a 2-bay, 2-story frame for a cyclic loading test. Test results showed that incorporating UHS reinforcing bars in frame columns increased the uniformity of the inter-story displacement distribution, preventing concentrated damage at the bottom of the columns. The residual displacement and residual crack width of the frame were effectively controlled in the study of Liu et al. (2021).

In our previous work (Ou et al. (2022)), a self-centering RC bridge column was proposed. ASTM 416 (2010) Grade 270 (1860-MPa) seven-wire steel strands were used as the elastic element to provide stiffness and self-centering capability to the column after yielding of the conventional longitudinal steel reinforcement during an earthquake. Although the yield strength of the steel strand was extremely high, the steel strand could still yield when the column was subjected to a large displacement. To prevent the yielding of the steel strand before a 5% drift ratio of the column was achieved, duct tape was used to wrap a certain length of the steel strand extending upward from the plastic hinge area of the column. The steel strand was not prestressed, which reduced the construction cost of the column compared to the post-tensioned self-centering bridge column. Moreover, concrete creep due to prestressing can be avoided. In this study, based on the self-centering column developed in Ou et al. (2022), the concrete cover thickness ratio of the proposed column was varied to explore the influence of the loss of concrete at the compressive side on the post-yield stiffness ratio of the column.

2 Experimental program
2.1 Specimen design
Four large-scale bridge columns with a scale factor of 0.5 were designed and tested. The nominal yield strength of the deformed bar used in the test was 420 MPa, the nominal compressive strength of the concrete was 35 MPa, and the nominal ultimate strength of the steel strand was 1860 MPa. In all columns, the longitudinal deformed bar design, transverse deformed bar design, axial compressive load, and section size and height of the column were identical; the columns only differed in their use of the steel strand and concrete cover thickness. Each column contained 12 D25 longitudinal deformed bars. The area ratio of the longitudinal deformed bar to the column section was 1.69%. The bar size and vertical spacing of the transverse reinforcement were D13 and 100 mm, respectively. The applied axial compressive load was 0.1 $A_gf'_c$, where $A_g$ and $f'_c$ are the gross cross-sectional area and concrete compressive strength of the column, respectively. The section size of the column was 600 mm x 600 mm. The height of the column was 4500 mm, and the height from the lateral load to the bottom of the column was 3050 mm. The four bridge columns were denoted as CCC, CSC40, CSC30, and CSC20. CCC was the control column and had a concrete cover thickness of 30 mm and no steel strand. A total of 12 seven-wire steel strands with a diameter of 15.2 mm were added to CSC40, CSC30, and CSC20 as the elastic elements of the columns. The area ratio of steel strands to the column section was 0.47%. To study the influence of different concrete cover thicknesses on the post-yield stiffness ratio, concrete cover thicknesses of 40, 30, and 20 mm were investigated for CSC40, CSC30, and CSC20, respectively. Note that the
concrete cover thickness of bridge columns typically ranges from 80 to 40 mm depending on the exposure conditions (ACI 2019). With a scale factor of 0.5, the range of the thickness was reduced to 40 to 20 mm for the columns of this research. The specified concrete cover ratio was calculated as the ratio of the specified concrete cover thickness to the specified effective depth as defined by Eq. (1). Its values were 7.5%, 5.5%, and 3.6% for CSC40, CSC30, and CSC20, respectively. The material properties are listed in Table 1, and design variables of the columns are listed in Table 2. Note that the test results of CCC and CSC20 have been reported in Ou et al. (2022).

In Table 1, \( \varepsilon_{yl}, \varepsilon_{ul}, f_{yl}, f_{ul} \) are the yield strain, ultimate strain, yield strength, and ultimate strength of the longitudinal deformed bar, respectively; \( \varepsilon_{py}, \varepsilon_{pu}, f_{py}, f_{pu} \) are the yield strain, ultimate strain, yield strength, and ultimate strength of the strand, respectively; \( f_{yh} \) is the yield strength of the transverse deformed bar; and \( f_{c} \) is the compressive strength of the concrete.

In Table 2, \( \rho_{lb} \) is the ratio of the area of the longitudinal deformed bar to the gross area of the column section; \( \rho_{lp} \) is the ratio of the area of the steel strand to the gross area of the column section; \( c_{c,D} \) is the specified concrete cover thickness; \( d_{L,D} \) is the specified distance from the extreme compression fiber to the centroid of the farthest layer of longitudinal tension reinforcement; and \( c_{L,D} \) is the specified concrete cover ratio.

\[
c_{L,D} = \frac{c_{c,D}}{d_{L,D}} \tag{1}
\]

The design details of the columns tested in this study are presented in Fig. 1(a). To prevent the yielding of the steel strand before a drift ratio of 5% was achieved, duct tape was used to wrap 2300 mm of the steel strand upward from the column bottom to avoid bonding between the steel strand and the concrete, as shown in Fig. 1(b). The aforementioned process reduced the stress on the steel strand and ensured that the steel strand did not yield prematurely. Figure 1(c) displays the wedge applied to the anchor. Because the steel strand was not prestressed, it could experience repeated tension and

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**Table 1** Material properties

| Column | \( \varepsilon_{yl} \) (10^-3) | \( \varepsilon_{ul} \) (10^-3) | \( f_{yl} \) (MPa) | \( f_{ul} \) (MPa) | \( \varepsilon_{py} \) (10^-3) | \( \varepsilon_{pu} \) (10^-3) | \( f_{py} \) (MPa) | \( f_{pu} \) (MPa) | \( f_{yh} \) (MPa) | \( f_{c} \) (MPa) |
|--------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| CCC    | 2.4             | 114.4           | 466.1           | 662.3           | -               | -               | -               | -               | 459.2           | 37.3            |
| CSC40  | 8.9             | 45.7            | 1667.0          | 1987.7          | 33.0            | 33.9            | 33.9            | 33.9            | 33.9            | 33.9            |
| CSC30  |                 |                 |                 |                 | 30              | 544.6           | 5.5             |                 |                 |                 |
| CSC20  |                 |                 |                 |                 | 20              | 554.6           | 3.6             |                 |                 |                 |

**Table 2** Design variables of the columns

| Column | \( \rho_{lb} \) (%) | \( \rho_{lp} \) (%) | Strand design | \( c_{c,D} \) (mm) | \( d_{L,D} \) (mm) | \( c_{L,D} \) (%) |
|--------|-------------------|--------------------|----------------|-------------------|-------------------|-----------------|
| CCC    | 1.69              | -                  | -              | 30                | 544.6             | 5.5             |
| CSC40  | 0.47              | 12-D15.2           | 40             | 534.6             | 7.5               |                 |
| CSC30  |                   |                    | 30             | 544.6             | 5.5               |                 |
| CSC20  |                   |                    | 20             | 554.6             | 3.6               |                 |
compression when the column was subjected to a cyclic loading test. Therefore, two single-strand tension anchors were welded back to back to the two sides of a steel plate, as shown in Fig. 1(d), and were used to provide tension and compression anchorage to each
strand. Figure 1(e) depicts inserting the wedge into the anchor. Figure 1(f) illustrates the construction of the specimens.

2.2 Test setup
The columns were tested using single-curvature cyclic loading in the reaction wall zone at the National Center for Research on Earthquake Engineering. The test setup is displayed in Fig. 2. The foundation of the column was fixed to the strong floor. Two hydraulic jacks were installed on the top of the column to apply an axial compressive load of $0.1 \, A_g f'_c$. This load remained constant throughout the testing. A horizontal actuator was used to apply displacement-controlled lateral cyclic loading to the column to nominal drift ratios of 0.25%, 0.375%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, 3.0%, 4.0%, 5.0%, 6.0%, 7.0%, and 8.0%. The loading to each drift ratio was repeated twice. The test was concluded after the strength of the column was considerably decreased.

3 Test results
3.1 Damage observations
The CCC (control column without steel strands) exhibited flexural cracks at a nominal drift ratio of 0.25%. The lateral force of the column was maximum at a nominal drift ratio of 2.0% in the positive and negative directions. According to the orientation of the column during testing, west to east was defined as the positive direction (compression on the east side), and east to west was defined as the negative direction (compression on the west side). At a nominal drift ratio of 3.0%, spalling of the concrete cover was observed near the base of the column. At a nominal drift ratio of 7.0%, the longitudinal steel deformed bars fractured.

CSC40 (with steel strands and concrete cover thickness of 40 mm) exhibited flexural cracks at a nominal drift ratio of 0.25%. The lateral force of the column reached a maximum value at a nominal drift ratio of 2% in the negative direction. At a nominal drift ratio of 4.0%, spalling of the concrete cover was observed near the base of the column. The lateral force of the column reached a maximum value at a nominal drift ratio of 5.0% in the positive direction. At a nominal drift ratio of 6.0%, the compressed steel strands began to bulge (Fig. 3(a)). At a nominal drift ratio of 7.0%, the longitudinal steel deformed bars fractured, as displayed in Fig. 3(b).
CSC30 (with steel strands and concrete cover thickness of 30 mm) exhibited flexural cracks at a nominal drift ratio of 0.25%. At a nominal drift ratio of 4.0%, spalling of the concrete cover was observed near the base of the column. The lateral force of the column reached a maximum value at a nominal drift ratio of 5.0% in the positive and negative directions. At a nominal drift ratio of 6.0%, the compressed steel strands began to bulge (Fig. 3(c)). At a nominal drift ratio of 7.0%, the longitudinal steel deformed bars fractured, as depicted in Fig. 3(d).

CSC20 (with steel strands and concrete cover thickness of 20 mm) exhibited flexural cracks at a nominal drift ratio of 0.25%. The lateral force of the column reached a maximum value at a nominal drift ratio of 5.0% in the positive and negative directions. At a nominal drift ratio of 6.0%, the compressed steel strands began to bulge, and the concrete cover was severely spalled (Fig. 3(e)). At a nominal drift ratio of 7.0%, the longitudinal steel deformed bars fractured, as illustrated in Fig. 3(f).

The test results revealed that the maximum lateral force of the columns with steel strands was higher than that of the column without steel strands. Moreover, the
maximum lateral force was observed at a higher drift ratio when using steel strands than when not using steel strands. These results indicate that using steel strands is beneficial for developing and maintaining the post-yield stiffness ratio. However, the drift ratio for the maximum force in the positive direction was 5.0% for CSC40, whereas that in the negative direction was substantially reduced to 2.0%. This result was likely attributed to the non-uniform distribution of actual concrete cover thicknesses of the column. This phenomenon is further discussed in subsequent sections.

3.2 Hysteretic response

The lateral force–displacement response of each column was measured and recorded. The envelope response was obtained from the peak value of the first cycle for each drift ratio in the lateral force–displacement response, and the envelope response of each column was idealized using a bilinear relationship in accordance with Federal Emergency Management Agency (FEMA) standard 356 (2000). The first line of the bilinear relationship passed through the envelope response at approximately 0.6 $P_y$, where $P_y$ is the force of the idealized yield point. The first line ended at the idealized yield point. The second line began from the idealized yield point and ended at the ultimate drift on the envelope response. The yield point was adjusted such that the area below the idealized bilinear relationship approximated that below the envelope response. The process of obtaining the idealized bilinear relationship is illustrated in Fig. 4. In Fig. 4, $D_y$ is the idealized yield drift; $P_y$ is the idealized yield force; $D_u$ is the ultimate drift; $P_u$ is the lateral force at $D_u$; $k_i$ is the initial stiffness; $k_2$ is the post-yield stiffness; and $\alpha$ is the post-yield stiffness ratio. The post-yield stiffness ratio $\alpha$ is defined as follows:

$$\alpha = \frac{k_2}{k_i}$$

![Idealized bilinear relationship](image-url)
The ultimate drift ($D_u$) of each column was defined in accordance with the damage condition described in Sect. 3.1. For CCC, the drift of the ultimate point was defined as the drift level immediately before the longitudinal steel bars fractured. For CSC40, CSC30, and CSC20, the ultimate drift was defined as the drift immediately before the steel strands bulged because the lateral force declined after this point. The positive post-yield stiffness could not be maintained after strand bulging. The ultimate force ($P_u$) was the force corresponding to the ultimate drift in the envelope response of each column.

The lateral force–displacement response, the envelope response, and the idealized response of each column are illustrated in Fig. 5. The lateral force presented in Fig. 5 was modified to remove the P-Delta effect of the axial load system (Ou et al. 2015). The drift ratios shown in Fig. 5 were measured using an optical system and differed from the nominal drift ratios.

![Graphs illustrating lateral force–displacement behavior of columns](image)
Table 3 reveals that compared with the conventional column (CCC), the three columns with steel strands (CSC40, CSC30, and CSC20) had higher ultimate strength and post-yield stiffness ratios; thus, these three columns have better self-centering ability after an earthquake. The test results revealed that CSC40, which had the largest specified concrete cover ratio of 7.5%, had the smallest average post-yield stiffness ratio (2.1%), followed by CSC30 and CSC20 (specified concrete cover ratios of 5.5 and 3.6%, respectively, and post-yield stiffness ratios of 3.9 and 6.4%, respectively). Because the concrete cover was not confined and could spall easily, a higher concrete cover ratio resulted in a higher loss of concrete in the compression zone. The loss of the compression zone resulted in the shortening of the distance between the resultant tension and compression forces acting on the critical column cross-section (column base) and decreased the post-yield stiffness. This explanation is further supported by other test observations presented in subsequent sections.

In Table 3, $D_p$ is the peak drift; and $P_p$ is the peak lateral force.

### 3.3 Energy dissipation

The energy dissipation of columns was evaluated using the equivalent viscous damping ratio calculated as follows:

$$
\beta_{eq} = \frac{1}{2\pi} \times \left( \frac{E_D}{K_{eff} D^2} \right)
$$

(3)

$$
K_{eff} = \frac{|F^+| + |F^-|}{|\Delta^+| + |\Delta^-|}
$$

(4)

where $\beta_{eq}$ is the equivalent viscous damping ratio; $E_D$ is the energy dissipation of a hysteretic loop; $D$ is the maximum displacement of a hysteretic loop; $K_{eff}$ is the effective stiffness; $\Delta^+$ and $\Delta^-$ are the maximum positive and negative displacement of a hysteretic hoop, respectively; and $F^+$ and $F^-$ are the forces corresponding to $\Delta^+$ and $\Delta^-$, respectively.

Figure 6 presents the equivalent damping ratio of each column (the average value of two cycles) at each nominal drift ratio. The lateral strength of the columns with steel strands was partly contributed by the steel strands. However, because the strands maintained predominantly elastic to enable self-centering, as evident from the strain

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### Table 3  Force–drift capacity of the columns

| Column | Loading direction | $D_y$ (%) | $P_y$ (tf) | $P_p$ (tf) | $P_o$ (tf) | $D_o$ (tf) | $P_u$ (tf) | $k_1$ (tf/m) | $k_2$ (tf/m) | $\alpha$ (%) |
|--------|-------------------|-----------|------------|------------|------------|-------------|------------|---------------|---------------|-------------|
| CCC    | +                 | 0.80      | 35.3       | 1.83       | 36.0       | 5.70        | 34.8       | 1444          | -3            | -0.2        | -0.3 (avg)  |
|        | -                 | -0.80     | -35.1      | -1.73      | -35.2      | -5.82       | -34.3      | 1436          | -6            | -0.4        |
| CSC40  | +                 | 0.89      | 38.0       | 4.96       | 42.6       | 4.96        | 42.6       | 1405          | 37            | 2.6         | 2.1 (avg.)  |
|        | -                 | -1.01     | -38.2      | -4.99      | -41.3      | -4.95       | -40.5      | 1240          | 19            | 1.5         |
| CSC30  | +                 | 1.16      | 39.1       | 4.94       | 41.2       | 4.94        | 41.2       | 1125          | 25            | 2.2         | 3.9 (avg.)  |
|        | -                 | -1.13     | -37.8      | -4.94      | -45.0      | -4.94       | -45.0      | 1100          | 62            | 5.7         |
| CSC20  | +                 | 0.99      | 36.8       | 4.66       | 43.4       | 4.66        | 43.4       | 1216          | 60            | 4.9         | 6.4 (avg.)  |
|        | -                 | -0.97     | -34.2      | -4.87      | -45.1      | -4.87       | -45.1      | 1156          | 91            | 7.9         |
data presented in the next section, the strands could not provide hysteretic energy
dissipation through yielding. Figure 6 reveals that at a drift ratio higher than 2.0%, the
equivalent damping ratio of the conventional column (CCC) was higher than those
of the columns with steel strands (CSC40, CSC30, and CSC20). The effect of different
concrete cover thickness ratios on the equivalent damping ratio was insignificant.

3.4 Tensile strain responses
Strain gauges were installed on the longitudinal reinforcement (deformed bars and
steel strands) to observe the strain responses under loading. Figure 7 presents the
strain distribution of the longitudinal deformed bars of the columns with strands
(CSC40, CSC30, and CSC20), and Fig. 8 shows the strain distribution of the steel
strands of these columns. The y axis of Figs. 7 and 8 represents the elevation of the
strain gauge from the bottom of the column.

Figures 7 and 8 reveal that the maximum strains of the longitudinal deformed bars
exceeded the yield strain at a drift ratio of 1.0%. At a drift ratio of 4.0%, the maximum
strain exceeded 0.02. The strain distribution of the steel strands was substantially dif-
ferent from that of the longitudinal deformed bars. The strains were similar across the
unbonded length (from y = 0 to 2300 mm). Wrapping the strands with tape effectively
prevented bonding between the steel strand and the concrete; thus, the strain values did
not vary. Because of the spreading of the strain across the strands, their maximum strain
was considerably smaller than that of the longitudinal deformed bars. At a drift ratio
of 4.0%, the maximum strain was approximately 0.004, which was considerably smaller
than the yield strain of the strands (i.e., 0.0089). The use of partially unbonded strands
effectively reduced the maximum strain and delayed yielding. Comparing the strain
responses between the three columns reveal that the differences in the peak value and

Fig. 6 Equivalent damping ratios of the columns
distribution of strains were not significant for the same drift ratio. This indicates that the effect of the concrete cover thickness ratio on the strain responses was not significant.

3.5 Analysis of the concrete cover thickness ratio

As described in Sects. 3.1 and 3.2, the post-yield stiffness ratios differed significantly between positive and negative loading directions for columns with steel strands (CSC40, CSC30, and CSC20). After completing a cyclic loading test, the crushed and spalled concrete of the column was removed, and the concrete cover thickness around the column was measured (Fig. 9(a)). Figure 9(b)–(e) presents the measurement results for the four columns tested. Moreover, Table 4 compares the measured concrete cover thickness and post-yield stiffness ratio of the columns. The offset of the core concrete centroid was calculated based on the measured concrete cover thickness.

According to the data in Table 4, CCC and CSC20 had similar concrete cover thickness on their east and west sides. The specified concrete cover thickness of CSC40 was 40 mm; however, the measurement results revealed that the thicknesses of its east and west sides were 33 and 50 mm, respectively. The core concrete centroid was offset to the east by approximately 8.5 mm. The specified concrete cover thickness of CSC30 was 30 mm; however, the measurement results revealed that the thickness of its east and
The core concrete centroid of this column was offset to the west by approximately 14.0 mm. The variation of the measured concrete cover thickness around the columns was due to the construction error. However, this variation also provided more test data between the cover thickness and the post-yield stiffness ratio.

In Table 4, $c_{c,A}$ is the measured concrete cover thickness.

For further exploring the influence of the loss of concrete cover on the post-yield stiffness ratio, the measured concrete cover thickness ($c_{c,A}$) was divided by the measured effective depth ($d_{e,A}$) to obtain the measured concrete cover ratio ($c_{L,A}$). $d_{e,A}$ is the
measured distance from the extreme compression fiber of the section to the centroid of the farthest layer of longitudinal tension reinforcement.

\[ c_{L,A} = \frac{c_{c,A}}{d_{L,A}} \]  

(5)

Table 5 and Fig. 10 show that the post-yield stiffness ratio tended to be inversely proportional to the measured concrete cover thickness ratio. This was because the distance of the resultant tension and compression forces (moment arm) acting on the critical column cross-section was positively related to the effective depth of the cross-section. As the concrete cover thickness ratio increased, the moment arm tended to decrease by a higher percentage after the loss of concrete cover. The reduction in the moment arm likely decreased the increase rate of the column lateral strength, leading to a smaller post-yield stiffness ratio. In the positive direction of CSC40 (CSC40+) and for CSC40 – and CSC30 +, the concrete cover thickness ratio on the compression side was large, which resulted in a considerable loss of the moment arm after spalling of the concrete cover. Thus, the post-yield stiffness ratios were small. Smaller concrete cover thickness ratios were observed on the compression side for

Table 5  Analysis of the concrete cover thickness ratio

| Column | Loading direction | \( c_{c,A} \) (mm) | \( d_{L,A} \) (mm) | \( c_{c,A} \)/\( d_{L,A} \) (%) | \( \alpha \)/\( d_{L,A} \) (%) |
|--------|-------------------|-------------------|-------------------|-----------------|-----------------|
| CSC40  | + (E)             | 33                | 524.6             | 6.3             | 2.6             |
|        | - (W)             | 50                | 541.6             | 9.2             | 1.5             |
| CSC30  | + (E)             | 48                | 554.6             | 8.7             | 1.2             |
|        | - (W)             | 20                | 526.6             | 3.8             | 5.7             |
| CSC20  | + (E)             | 25                | 550.6             | 4.5             | 4.9             |
|        | - (W)             | 24                | 549.6             | 4.4             | 7.9             |

Fig. 10  Relationship between the concrete cover thickness ratio and post-yield stiffness ratio
CSC30−, CSC20+, and CSC20− than for CSC40+, CSC40−, and CSC30+; thus, for CSC30−, CSC20+, and CSC20−, the reduction in the moment arm was smaller after the concrete cover spalled. The post-yield stiffness ratios for CSC30−, CSC20+, and CSC20− were higher than those for CSC40+, CSC40−, and CSC30+, which revealed the influence of the loss of concrete cover thickness ratio on the post-yield stiffness ratio.

A linear regression was used to derive a formula to relate the measured concrete cover ratio \( c_{L,A} \) with the post-yield stiffness ratio \( \alpha \). The derived linear regression formula is as follows:

\[
\alpha = -1.01c_{L,A} + 10.16
\]  

Note that the effective depth of the cross-section \( d_{L,A} \) was used to represent the effect of the moment arm. It is known that the moment arm depends on design parameters such as the concrete strength, the axial load, and the amount and distribution of longitudinal reinforcement. Since bridge columns usually have a low axial load and the moment arm is not sensitive to the concrete strength and the design of longitudinal reinforcement for typical bridge columns, the use of the effective depth provides a good approximation of the effect of the moment arm. Moreover, the use of the effective depth simplifies the calculation. Hence, Eq. (6) can be used in the preliminary design when most of the design details are still unknown to determine the feasibility of using the proposed column to reduce the residual displacement. Further research can be done to refine Eq. (6) to include the effects of design parameters more directly. Note that the post-yield stiffness ratio depends on the amount, distribution, and strength of the steel strands relative to those of the deformed bars. Thus, care should be taken if Eq. (6) is used for columns with the proportion of the steel strands relative to the deformed bars significantly different than that examined in this research.

Kawashima et al. (1998) reported that if the post-yield stiffness ratio is 5%, the residual displacement of a bridge column can be effectively controlled. In accordance with this result, Eq. (6) reveals that the concrete cover thickness ratio should not exceed 5.1%. However, the concrete cover is used to protect reinforcements from corrosion. Based on ACI 318–19 (ACI 2019), the concrete cover thickness for columns exposed to weather or in contact with the ground should not be less than 50 mm. This is a typical condition for bridge columns. For the case of a concrete cover thickness of 50 mm, the upper limit of 5.1% concrete cover thickness ratio suggests that the effective depth of the cross-section of the proposed self-centering column be not less than 980 mm to ensure a post-yield stiffness ratio of at least 5%.

4 Conclusion

A new self-centering bridge column has been proposed by the authors. Unstressed partially unbonded seven-wire steel strands were used as elastic elements to reduce the residual displacement of the proposed column. In this study, the influence of the concrete cover thickness ratio on the post-yield stiffness of the proposed column was examined. The behavior of four large-scale column specimens was studied through cyclic load testing. Important conclusions are summarized as follows.
1. The proposed columns with unstressed partially unbonded steel strands achieved post-yield stiffness ratios higher than the conventional column. The partial unbonding effectively reduced the strain of the strands and kept the strands in the elastic range after yielding of the longitudinal steel deformed bars.

2. The concrete cover thickness ratio did not have a significant effect on the hysteretic energy dissipation and the strain response of the longitudinal reinforcement of the proposed columns. However, the concrete cover thickness ratio significantly impacted the post-yield stiffness ratio. Test results showed that the post-yield stiffness ratio of the proposed column tended to be inversely proportional to the concrete cover thickness ratio. This was because the distance between the resultant tension and compression forces acting on the column cross-section (moment arm) tended to decrease with increasing concrete cover thickness ratio after the loss of the concrete cover. The decrease in moment arm reduced the increase rate of the lateral strength of the column in the post-yield range and hence decreased the post-yield stiffness ratio.

3. A relationship between the concrete cover thickness ratio and the post-yield stiffness ratio was proposed for the preliminary design of the proposed column. Based on the relationship, to achieve a post-yield stiffness ratio of 5%, as recommended in the literature to control the residual displacement of a column after a strong earthquake, the concrete cover thickness ratio should not exceed 5.1%. For a bridge column with a concrete cover thickness of 50 mm, this means the effective depth of the column cross-section should not be less than 980 mm.

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Authors’ contributions
Yu-Chen Ou conceived this study, participated in its design and coordination, and reviewed and revised the manuscript. Jhen-Wei Wu participated in the experiments, analysis of the test data, and drafting of the manuscript. Ade Yuniati Pratiwi participated in the experimental studies and analysis of the test data. All the authors read and approved the final manuscript.

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Availability of data and materials
The data and materials generated or used during the study are available from the corresponding author by request.

Declarations
Competing interests
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References
Abrahamson NA (1998) Seismological aspects of near-fault ground motions. The 5th Caltrans Seismic Research Workshop, Sacramento June 18, Session I
ACI Committee 318, Building Code Requirement for Structural Concrete (ACI 318-19) and Commentary (ACI318R-19) (2019), American Concrete Institute, Farmington Hills, p 503.
ASTM A416/A416M (2010) Standard specification for steel strand, uncoated seven-wire for prestressed concrete. American Society for Testing and Materials International, West Conshohocken.

Central Geological Survey, MOEA (2019). https://faultnew.moeacgs.gov.tw

Chou CC, Chen YC (2006) Cyclic tests of post-tensioned precast CFT segmental bridge columns with unbonded strands. Earthq Eng Struct Dyn 35:159–175

Fahmy MFM, Wu Z, Wu G, Sun Z (2010) Post-yield stiffnesses and residual deformations of RC bridge columns reinforced with ordinary rebars and steel fiber composite bars. Eng Struct 32:2969–2983

FEMA 356 (2000) Prestandard and commentary for the seismic rehabilitation of buildings. Federal Emergency Management Agency, Washington, DC

Hewes JT (2002) Seismic design and performance of precast concrete segmental bridge columns. Ph.D. dissertation. University of California, San Diego

Ibrahim AI, Wu G, Sun Z, Cui H (2017) Cyclic behavior of concrete columns reinforced with partially unbonded hybrid. Eng Struct 131:311–323

Iemura H, Takahashi Y, Sogabe N (2006) Two-level seismic design method using post-yield stiffness and its application to unbonded bar reinforced concrete piers. Structural Eng/earthquake Eng 23(1):109s–116s

Iemura H, Takahashi Y, Sogabe N (2004) Development of unbonded bar reinforced concrete structure. Proceedings of 13th World Conference on Earthquake Engineering. 13 WCEE Secretariat, Vancouver

Kalkan E, Kannath SK (2006) Effects of fling step and forward directivity on seismic response of buildings. Earthq Spectra 22(2):367–390

Kawashima K, MacRae GA, Hoshikuma J, Nagaya K (1998) Residual displacement response spectrum. J Struct Eng 124(5):523–530

Liu J, Zhang J, Li X, Cao W (2021) Cyclic behavior of damage-controllable steel fiber reinforced high-strength concrete reduced-scale frame structures. Eng Struct 252:111810

Ou YC (2007) Precast segmental post-tensioned concrete bridge columns for seismic regions. A dissertation submitted in partial fulfillment for the requirements of the degree of Doctor Philosophy in Faculty of the Graduate School of State University of New York at Buffalo

Ou YC, Tsai MS, Chang KC, Lee GC (2010a) Cyclic behavior of precast segmental concrete bridge columns with high performance or conventional steel reinforcing bars as energy dissipation bars. Earthq Eng Struct Dyn 39:1181–1198

Ou YC, Wang PH, Tsai MS, Chang KC, Lee GC (2010b) Large-scale experimental study of post-tensioned unbonded posttensioned concrete bridge columns for seismic regions. J Struct Eng 136(3):255–264

Ou YC, Ngo SH, Roh H, Yin SY, Wang JC, Wang PH (2015) Seismic performance of concrete columns with innovative seven- and eleven-spiral reinforcement. ACI Struct J 112(5):579–592

Ou YC, Wu JW, Pratiwi AY (2022) Cyclic behavior columns with partially unbonded seven-wire steel strands to increase post-yield stiffness. Eng Struct 258:114112

Phan V, Saidi MS, Anderson J, Ghasemi H (2006) An exploratory experimental study of near-fault ground motion effects on reinforced concrete bridge columns. Proceedings of the 100th Anniversary Earthquake Conference, Commemorating The 1906 San Francisco Earthquake, 8th US National Conference on Earthquake Engineering, San Francisco, CA, United States. Paper No. 377. Earthquake Engineering Research Institute

Somerville PG, Smith NF, Graves RW, Abrahamson NA (1997) Modification of empirical strong ground motion attenuation relations to include the amplitude and duration effects of rupture directivity. Seismol Res Lett 68(1):199–222

Yamanobe S, Sogabe N, Iemura H, Takahashi Y (2008) Development of high-seismic-performance reinforcement concrete bridge pier with high-performance plastic hinge. Proc JSCE 64(2):317–332

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