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Self-Centering Precast Unit as Energy Dissipation Members in Precast Segmental Bridge Columns

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Abstract: This research aims to present a new generation of seismic-resisting systems designed for precast reinforced concrete (RC) bridge piers in modern sustainable cities to withstand moderate to high seismic activity. The proposed system consists of two self-centering (SC) systems operating in parallel to bring together all features of the required resiliency during a seismic action. The first/main system is a hollow core precast segmental bridge column, and the second is composed of an SC precast unit and energy dissipation (ED) steel reinforcements positioned in the main pier segment's hollow core. To study the performance of the proposed system, a finite element model was first developed to capture the behavior of experimentally tested precast bridge columns. After validation, the created model was systematically studied to investigate the performance of the entire proposed system under cyclic loading. The effects of three parameters related to the ED system were investigated, including the reinforcement ratio, the unbonded length of ED bars, and the SC post-tensioned force ratio. Furthermore, the impact of FRP wrapping on the lower part of the core column of the ED system was also investigated. An analytical model predicting the characteristic points of the lateral response of the proposed system based on the superposition concept is also proposed. The FE results showed that the entire proposed system is a new design-based resilient system with the ability to dissipate energy without compromising the SC capacity of the main resisting system. Compared to the typical precast hollow core segmental column, a 6% reinforcement ratio of the ED unit can cause a 60% increase in lateral resistance and a 220% increase in the ED capacity. The analytical model can successfully be applied in the design of the proposed system to provide customized ED capabilities and controlled lateral resistance.

Keywords: bridge pier; precast column; energy dissipation; self-centering; resilience



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1. Introduction

The construction of reinforced concrete (RC) bridge components using the in situ construction method is time-consuming, as all the construction processes are completed on-site. Accelerated Bridge Construction (ABC) [1] aims to reduce on-site construction time by prefabricating most structural elements in a factory. These prefabricated parts are then transported to the construction site ready for use. This technique can reduce the number of workers, control the quality of the structure, and improve the environment [2]. One promising system to promote the application of ABC in bridges is a self-centering (SC) precast segmental bridge pier system. This system was first introduced in 1999 based on the rocking beam–column connection presented by Priestley and Tao [3]. The general components of the SC precast column system consist of a series of precast segments pinned

together and connected to the cap beam and foundation by unbonded tendons. Previous experimental and numerical studies on the SC precast column system for bridges have shown that the joints between the segments provide an opening and closing mechanism (a rocking mechanism) under lateral loading [2]. This rocking mechanism can significantly reduce the level of damage caused by earthquakes. The force in the tendons ensures that the column is pulled back into the vertical position, minimizing the residual displacement of the system [4]. On the other hand, such a system suffers from low energy dissipation (ED) capacity compared to a conventional cast-in-place concrete column.

Several studies have been carried out in the research community to improve the ED capacity by equipping the system with additional energy dissipaters. In this case, the SC is obtained from the force in the unbonded tendons, while the additional energy dissipaters dissipate the energy. These dissipaters can generally be divided into two categories, namely internal ED reinforcing bars and external ED devices. For the first category, various internal ED bars have been investigated in the literature, including bars made of mild steel [5], high-strength steel [6], and shape memory alloys [7]. Among these strategies, using bonded mild steel bars running continuously through the column segments is one of the most common strategies to overcome ED deficiencies. As the system combines the tendon and the ED bars, it is generally called a precast hybrid system. Stone et al. [4] first presented the precast hybrid system, in which bonded mild steel bars were installed in SC precast beam-to-column connections in a precast moment-resisting frame. Due to the promising performance of the precast hybrid system and the achievement of the required design objectives [8], it has been applied to various SC structures, including precast shear walls [9–11], and precast moment-resisting frames [12–14]. Among them, Restrepo and Rahman [10] carried out an experimental study on SC precast shear walls. The wall units were designed to include mild steel bars placed longitudinally across the joint between the walls and the foundation to enhance the ED capacity and maintain the SC ability. Song et al. [13] investigated the lateral behavior of a one-bay, one-story SC moment-resisting RC frame. The beam–column connections incorporated two steel bolts to offer an adequate ED capacity. Guo et al. [14] tested a one-by-two bay, two-story SC moment-resisting RC frame. The adaptable ED capacity is achieved through the employment of web friction devices at the beam ends. More recently, the concept of a hybrid precast concrete system has been extended to concrete bridge piers [15–19].

Numerous research papers have proposed to improve the hysteretic ED ability of precast bridge piers by using mild steel bars over the segment joints [20–24]. Under seismic loading, the precast hybrid system obtains SC by rocking the precast elements over the unbonded tendons and dissipating energy via mild steel bars. Among these studies, Wang et al. [17] carried out an experimental investigation on four SC precast segmental long bridge columns under cyclic loading. The system has used various design configurations to increase the lateral strength and hysteretic ED capability of the columns. The addition of bonded mild steel bars across the segment joints is one of these design configurations. The performance of four short large-scale precast SC segmental bridge columns under lateral cyclic loading was investigated by Ou et al. [18]. The experimental program consisted of one typical precast column (without ED mild steel bars), and three precast columns had different ED mild steel bars ratios running longitudinally across the segment joint and post-tensioning forces. Bu et al. [21] conducted a study to evaluate the structural performance of various types and arrangements of reinforcement in bridge columns. The study involved testing a monolithic bridge column as a reference specimen, as well as four precast segmental bridge columns with different reinforcement configurations. In one of the specimens, bonded mild steel ED bars were incorporated to enhance the ED capacity of the precast bridge column. According to these studies, compared to a typical precast pier, the hybrid system can ensure a substantial increase in the ED capacity of the structure. In addition, the use of ED bars can increase the column's load-carrying capacity. However, severe damage characterized by cracking and crushing of the concrete was detected in the bottom segments with ED bars [17,18]. The hysteresis curve demonstrated a significant degradation

in both the strength and stiffness as a result of the severe spalling at the column toes due to the substantial gap opening between the segments' joint [21]. In addition, due to the large displacement of the column and an increase in the opening between the segments, a number of the ED bars fractured [17,18]. The test results also showed that the test specimens exhibited a large residual drift after unloading, which exceeded 1% due to the inelastic deformation of the steel bars. According to Japanese code and specifications, it is challenging to rehabilitate bridges that have a residual drift of more than 1% [25]. Therefore, using precast SC bridge piers with mild steel bars may improve the ED capacity but extreme post-earthquake damage and residual drift may still occur. Therefore, using the hybrid precast system as the main system may not achieve the desired rapid recover ability after severe earthquakes. However, it can be used as an ED source for the main SC system.

This research presents a novel resilient system for precast concrete bridge columns that can overcome the above limitations of precast SC bridge columns. The proposed system consists of a typical hollow RC precast column and a newly introduced hybrid RC precast column serving as an ED source. The ED core column (ED-CC) combines prestressed tendons and bonded mild steel bars. The precast segmental column and the ED-CC contact each other at the interface between the head faces of the ED-CC and the corresponding inner faces of the concrete segments of the precast column, as shown in Figure 1a. Under seismic loading, the ED-CC dissipates energy via mild steel bars. Since the main system (precast segmental column) and the damping source (ED-CC) exhibit SC ability, the entire system ensures controlled residual displacement of the proposed system. To investigate the lateral behavior of the proposed system, a finite element model was first developed for a typical precast segmental column that was experimentally tested. Then, the verified model was systematically studied with the proposed ED-CC. The investigated parameters are related to the design details of the ED-CC, including: reinforcement ratio, prestressing force level, and unbonded length of the reinforcing bars. To improve damage tolerance due to large compressive stresses at the toes of the ED-CC, the FRP sheathing on the lower part of the column was also investigated.

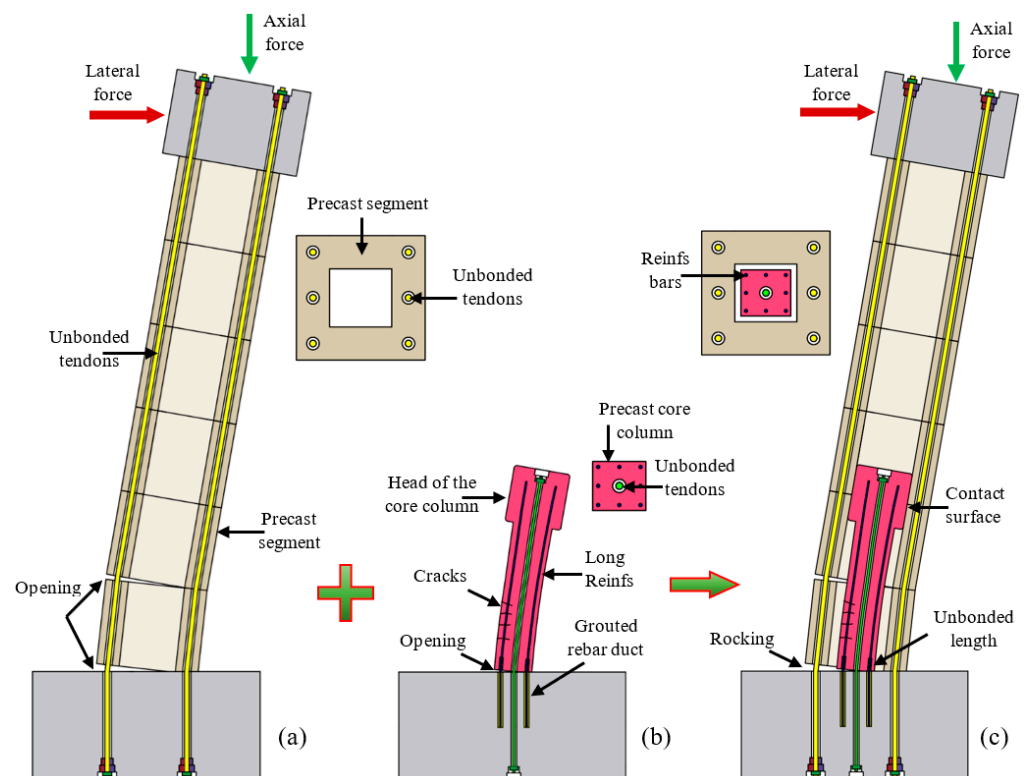


Figure 1. (a) Precast segmental hollow column, (b) precast hybrid core column, and (c) segmental column and hybrid core column.

2. Components

The components of the proposed system, as shown in Figure 1, consist of multiple prefabricated segments stacked on top of each other and connected to the cap beam and foundation by an unbonded tendon. Each prefabricated segment is reinforced with mild steel bars running in longitudinal and transverse directions that act as concrete confinement. However, these reinforcements do not extend over the joints between the segments. The only reinforcement that runs over the entire length of the column is the unbonded tendon. The connection between the column segments mainly depends on the clamping force exerted by the post-tensioned tendons. The force in the tendon provides the required frictional resistance to connect the segments and ensure the integrity of the entire system. The ED unit is a scaled precast column prestressed with unbonded tendons and containing mild steel bars. As shown in Figure 1, the ED-CC is positioned within the hollow space of the main precast column segments. The ED-CC is installed on the foundation with a grouted duct connection [26,27]. Contact between the ED-CC and the main precast column occurs at the head surfaces of the ED-CC and the corresponding internal surfaces of the segments in the main precast column.

3. Resisting Mechanism

The intended seismic resisting system ensures that the bridge's main substructure system and the ED units work in parallel. The main column resists lateral loading through a rocking mechanism, in which the prestressed force is applied via tendons to integrate all components of the bridge pier (the cap beam, the pier segments, and the foundation), as shown in Figure 1a. Post-tensioning is also effective in reducing the permanent deformations of the bridge after a seismic action: the rocking system is an SC system. The ED source is a non-emulative unit, consisting of a prefabricated RC column connected to the foundation and contributing to the lateral resistance system of the whole structure. The entire system will receive the necessary damping energy under lateral load from the mild steel bars yielding across the ED-CC and the foundation connection. The ED-CC will also contribute to resisting lateral loads and positively distribute the opening throughout the column segments, thus decreasing localized stress. Additionally, the prestress force in the tendons of the ED-CC can work in parallel with the main post-tension system of the entire structure to limit the residual drift that remains as a result of the mild steel bars' inelastic nature (controlling the residual drift). The core ED column is located within the main rocking system, and it may not affect its elastic stiffness. Even so, the yielding of the steel bars is responsible for the required ED. The ED-CC column and the main rocking system integrate only in the case of lateral displacement. Under lateral load, the contact between the head surfaces of the ED-CC and the corresponding internal surfaces of the main column segments will activate the lateral resistance of the ED-CC. In this case, the lateral resistance of the system is the summation of the lateral resistance of the main system and the lateral resistance of the added ED-CC. That is, the introduced system is a new resilient system capable of dissipating energy without compromising the seismic demands or the required reparability.

The moment capacity of the proposed system, M_y , is calculated using Equation (1) as a function of the moment capacity of the main SC segmental column, M_{SC} , and the moment capacity of the ED-CC, M_{ED} . As shown in Figure 2a, the main SC system behaves linearly with the initial stiffness K_1 until the rocking joints open at point Y_1 . From point Y_1 , the tendons elongate elastically and form a post-yielding stiffness K_2 . With the introduction of ED-CC, the system shows a flag-shaped hysteretic response described by the yield moment of the system M_{ED} , the flag height M_F , and two stiffnesses (K_3 and K_4), as shown in Figure 2b. Both the moment capacity of M_{SC} and M_{ED} can be designed based on the available recommendations and guidelines of the precast SC system [2,18]. For the main SC system, the moment capacity of the main segmental column can be calculated based on the simplified analytical model developed by Hewes and Priestley [2] for typical SC precast segmental bridge columns. Hewes' work was based on the concept of the "monolithic

beam analogy". For the ED–CC system, the moment capacity can be calculated using the "moment–interaction diagram" developed by Ou et al. [18]. The moment–interaction diagram can be constructed using curvature analyzes of a proposed cross-section and reinforcement details under consideration with varying axial force from the deformation of the unbonded tendon. When the main SC system is combined with the ED–CC system, the hysteretic moment–drift behavior is shown in Figure 2c. The moment–drift response of the proposed system is the superposition of the main SC system response and the ED–CC system response, as will be proved through the FE results. When the post-yielding stiffness of the SC system, K_2 , is relatively small compared to the initial stiffness, K_1 , and the initial stiffness of the ED–CC unit, K_3 , Equation (1) can be simplified as shown in Equation (2), neglecting the effect of the additional elongation of the tendon between the opening of the rocking interfaces and the yielding of the reinforcement in the ED–CC unit. For the unbonded tendon, neglecting this additional elongation is a simplifying assumption that allows for a more manageable analysis of the overall structural response due to the relatively small magnitude of this elongation compared to other deformations and the complexity it introduces to the analysis.

$$M_y = M_{SC} \left(1 - \frac{K_2}{K_1} \right) + M_{ED} \left(1 - \frac{K_2}{K_3} \right) \quad (1)$$

$$M_y \cong M_{SC} + M_{ED} \quad (2)$$

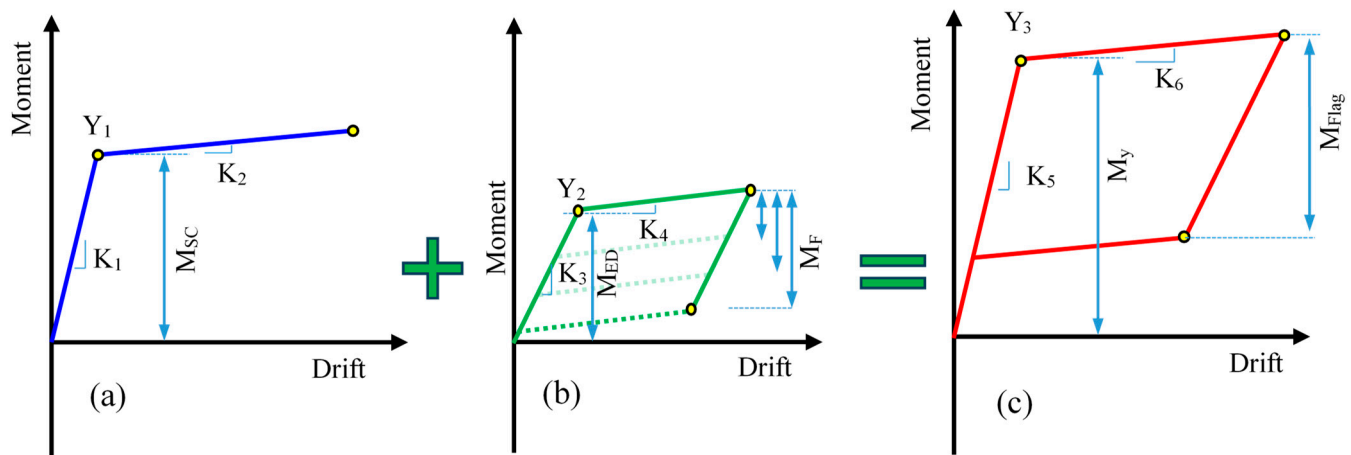


Figure 2. Schematic figure of the lateral response of (a) precast SC system, (b) precast ED–CC column, and (c) proposed system.

4. Finite Element Modeling

To investigate the lateral behavior of the new system, a precise and comprehensive 3D nonlinear finite element (FE) model of prefabricated bridge piers was generated by the ABAQUS program [28]. First, a comparison was made between the outcomes of the 3D FE and the existing test findings. Subsequently, the ED–CC was simulated and integrated into the previously validated model. The following section provides a brief overview of the precast test specimens provided in previous research [17], identifies the specifics of the models, and explains the stages of the modeling procedure.

4.1. Description of the Test Specimens

According to the available test specimens of precast concrete bridge columns conducted by Wang et al. [17], a detailed 3D FE model was developed. The model included three large-scale columns (P1, P2, and P3). The columns consisted of hollow precast concrete segments with cross-section dimensions of 1800×1200 mm and wall thickness of

300 mm, as shown in Figure 3. Each column had an RC cap beam and an RC foundation. The axial and prestressing forces on the three columns amounted to 4000 kN.

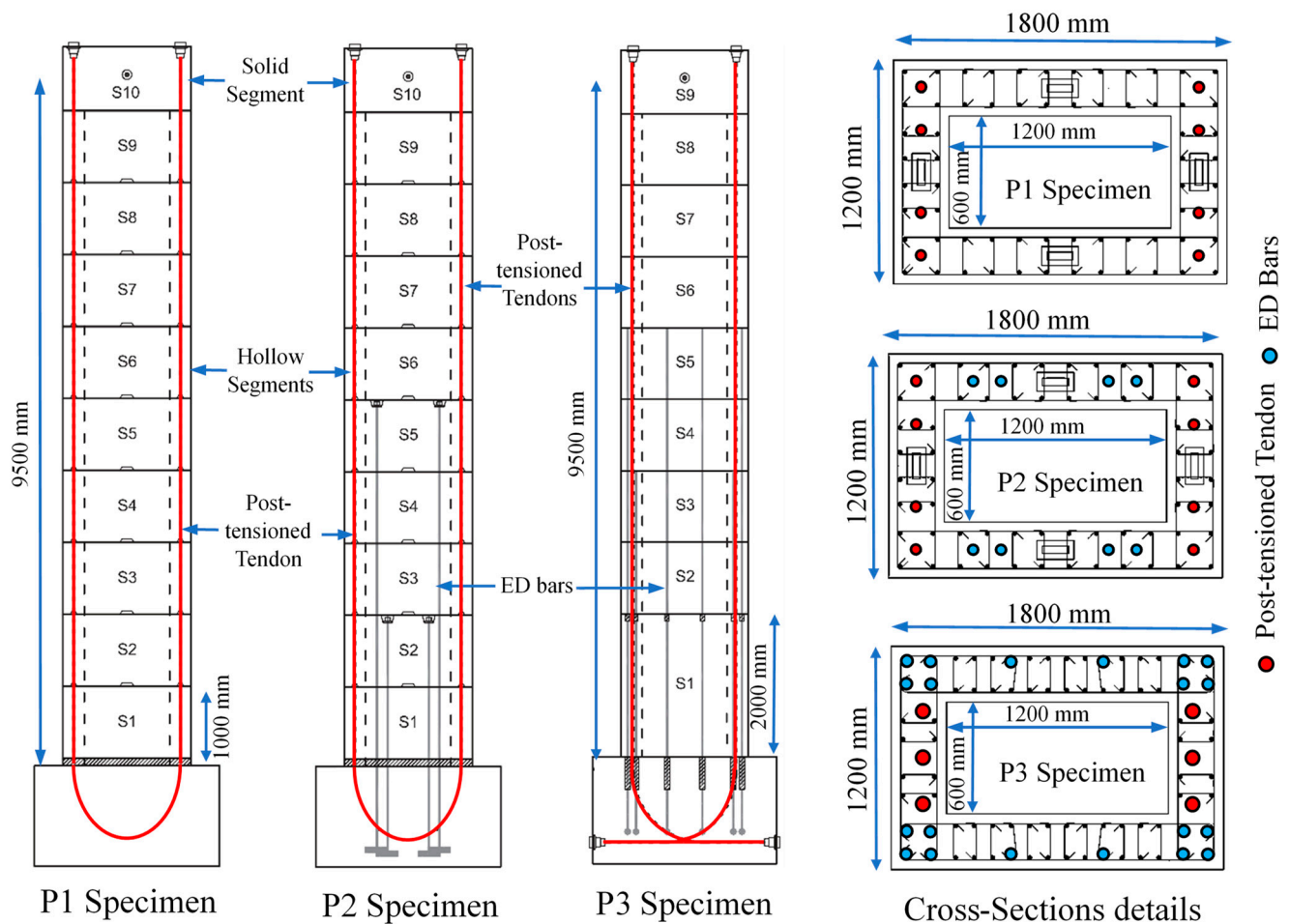


Figure 3. Design details of test specimens P1, P2, and P3.

For the test specimen P1, nine precast RC segments with a hollow cross-section were assembled, each one meter high. These segments were connected with eight unbonded tendons. The transverse reinforcement consisted of D13 mm mild steel bars, while the longitudinal reinforcement had D22 mm mild steel bars. The reinforcement of each segment did not continue from one segment to another. Consequently, the tendons acted as the only continuous longitudinal reinforcement, supporting the entire height of the column. The P1 test specimen was specifically intended for a regular SC precast concrete bridge pier.

P2 was designed with eight high-strength ED bars with a diameter of 36 mm at the foundation-S1 critical connection. To ensure a secure connection, T-headed threaded couplers were utilized in the foundation to fasten these bars. Additionally, four of these bars extended through grouted steel corrugated ducts up to S2, where they were firmly fastened with steel bolts and plates. The remaining four bars were also securely fastened and extended to the top of segment S5. Moreover, the prestressing tendons of specimen P2 were typically pressure-grouted during stress exposure to prevent corrosion and enhance the lateral strength of the column.

A total of eight RC precast segments were managed for the P3 test specimen. Furthermore, the height of the S1 segment was increased to 2 m. Six bonded tendons were positioned in the P3 specimen to ensure a more central distribution. In contrast to the P2 specimen, the whole circumference of the cross-section was equipped with twenty ED bars, which had a lower yield strength. The twenty ED bars extended from the RC

foundation to the first segment. Additionally, twelve bars of the twenty ED bars extended to the third segment, while eight ED bars of the twenty bars extended to the fifth segment. Additional steel bars were used to improve the connection between the first segment and the concrete foundation.

4.2. Modeling Procedure

All models created used an eight-node 3D brick element (C3D8R) to construct all concrete parts, such as the cap-beam block, precast RC columns, RC foundation, and ED–CC. The concrete damage plasticity model, also known as the CDP model, is commonly used in the ABAQUS program to define the materials of the concrete parts. This study utilized CDP input data, using a soft CDP model from reference [29]. Table 1 summarizes the maximum concrete compressive capacity for each specimen. In defining the parameters of the CDP model, the values of 30, 0.1, 1.16, 0.667, and 0.001 were assumed. These parameters correspond to the dilation angle, the flow potential eccentricity, the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress, the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian, and the viscosity parameter, respectively. These input data are consistent with the suggestions of the software documents [28] and available existing studies by Dawood et al. [30] and Li et al. [22].

Table 1. Properties of the materials [17].

	Items	P1	P2	P3
Concrete	Max Strength (MPa)	37	40	44
	Elastic modulus (GPa)	28.771	29.915	31.375
	Poisson's ratio		0.2	
ED bars	ED bars Arrangement	NA	S1–S2: 8 S3–S5: 4	S1: 20 S2–S3: 12 S3–S5: 8
	Elastic modulus (GPa)	NA	200	
	Yield strength (MPa)	NA	1080	490
	Tensile stress (MPa)	NA	1230	686
	Poisson's ratio	NA	0.3	
	Dimension		D13	
Transverse bars	Elastic modulus (GPa)		200	
	Yield strength (MPa)	372	372	511
	Tensile stress (MPa)	559	559	681
	Poisson's ratio		0.3	
	Dimension		D22	
Longitudinal bars	Elastic modulus (GPa)		200	
	Yield strength (MPa)	480	480	472
	Tensile stress (MPa)	637	637	663
	Poisson's ratio		0.3	
Tendons	Elastic modulus (GPa)		200	
	Yield strength (MPa)		1792	
	Tensile stress (MPa)		1888	
	Poisson's ratio		0.3	

The post-tensioned tendons, the cage reinforcement of the segments, and the longitudinal ED bars of the ED–CC were simulated with the truss element (T3D2). In order to anchor the unbonded tendons, a specified distance of the tendons was embedded in the RC cap beam and the RC foundation. On the other hand, the bond tendons, ED steel bars, and cage reinforcement bars of the segments were embedded in the solid concrete parts. The behavior of all truss elements was described with a bilinear elastic–plastic relationship based on the properties listed in Table 1. Surface contact elements were used to simulate the rocking manners between the joints of the column segments and the contact between the ED–CC head's outer surfaces and the column segments' inner surface. Friction with a coefficient of 0.5 governed the behavior of the tangential interaction between the surfaces of the solid parts [22]. To prevent interaction between the components through connection, the normal contact behavior of the interfaces was defined using hard contact. The BFRP

confinement for the lower part of the ED–CC is simulated with the shell element (S4R) available in the ABAQUS program version 6.2. A thickness of 2mm, 4mm, and 6mm was adopted for the BFRP shell element. The lower part of the foundation was fully restrained. In addition, the extended ED bars of the ED–CC were embedded in the foundation.

In order to assess the impact of different mesh dimensions on the outcomes of the model and enhance the efficiency of the program analysis, a sensitivity analysis was performed. The analysis focused on examining how the size of the mesh affected the results of the finite element model. Three specific mesh sizes were implemented for the P1 specimen: 50 × 50 mm, 100 × 100 mm, and 150 × 150 mm. Afterward, the numerical results were compared to the experimental results obtained from load–displacement curves. The findings indicated that the numerical curves were nearly indistinguishable, particularly for columns with mesh sizes of 50 mm and 100 mm. Consequently, three distinct mesh volumes were chosen to uniformly divide the column elements, as depicted in Figure 4. These mesh dimensions consisted of 100 mm, 150 mm, and 200 mm for the upper, middle, and lower segments of the column, respectively.

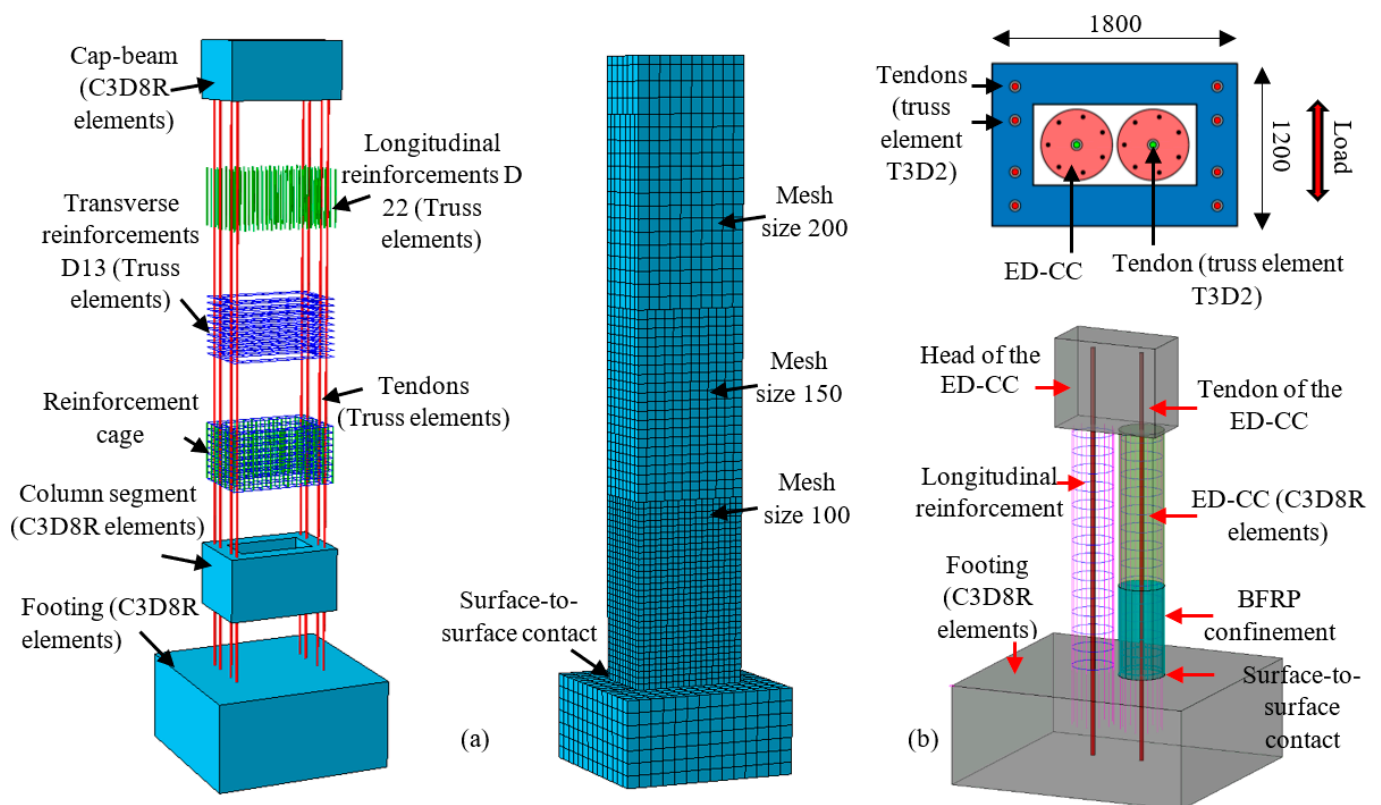


Figure 4. Finite element model of the ED–CC: (a) proposed column and (b) ED–CC.

The loading process of the developed models was carried out in three stages, corresponding to the loading order of the experimental test. The tendons were post-tensioned in the first step. The initial stress available in ABAQUS software version 6.2 was used to apply the post-tensioned force to the tendons in the first step. A gravity load of 4000 kN was applied to the top surface of the upper solid block of the column segments. Finally, the lateral load was applied using displacement control to drift ratio of 0.25, 0.375, 0.5, 0.75, 1.0, 1.5, 2.0, 3.0, and 4.0%. Additionally, the P3 test specimen experienced an additional 5% drift level.

4.3. Model Validation

In this section, a comparison is made between the FE results and test outputs of precast bridge pier columns subjected to cyclic loads. The comparison is divided into two stages, focusing on the damage mode and the hysteretic response.

4.3.1. Damage Pattern

Figure 5 illustrates the extent of damage observed in the FE model for specimen P1 and specimen P2 at a displacement ratio of 4%, and the damage pattern for specimen P3 at a displacement ratio of 5%. The figure also presents the damage pattern from experimental tests. The damage pattern in the FE outputs was assessed by selecting the strain limit. The dark elements of the model indicate that the pressure in these elements has exceeded the maximum compressive strength. This color was observed on the loading side of the first segment of sample P1, indicating that the stress in this part exceeded the maximum compressive strength. This observation is consistent with the experimental test findings of sample P1, where spalling was observed on the cover of the concrete in the lower part of segment S1, as shown in Figure 5a. Similarly, in sample P2, which utilized high-strength reinforcement bars, tension cracks were distributed along the lower part of the column. The same observation was noted in the test results, in which various cracks were observed in the bottom segments of the column, as shown in Figure 5b. Figure 5c demonstrates that the FE model results accurately captured the damage pattern of sample P3 compared with the test sample. There is a significant increase in the tensile stress at the bottom of the model, and blackening can be observed between the first and second segments of the model.

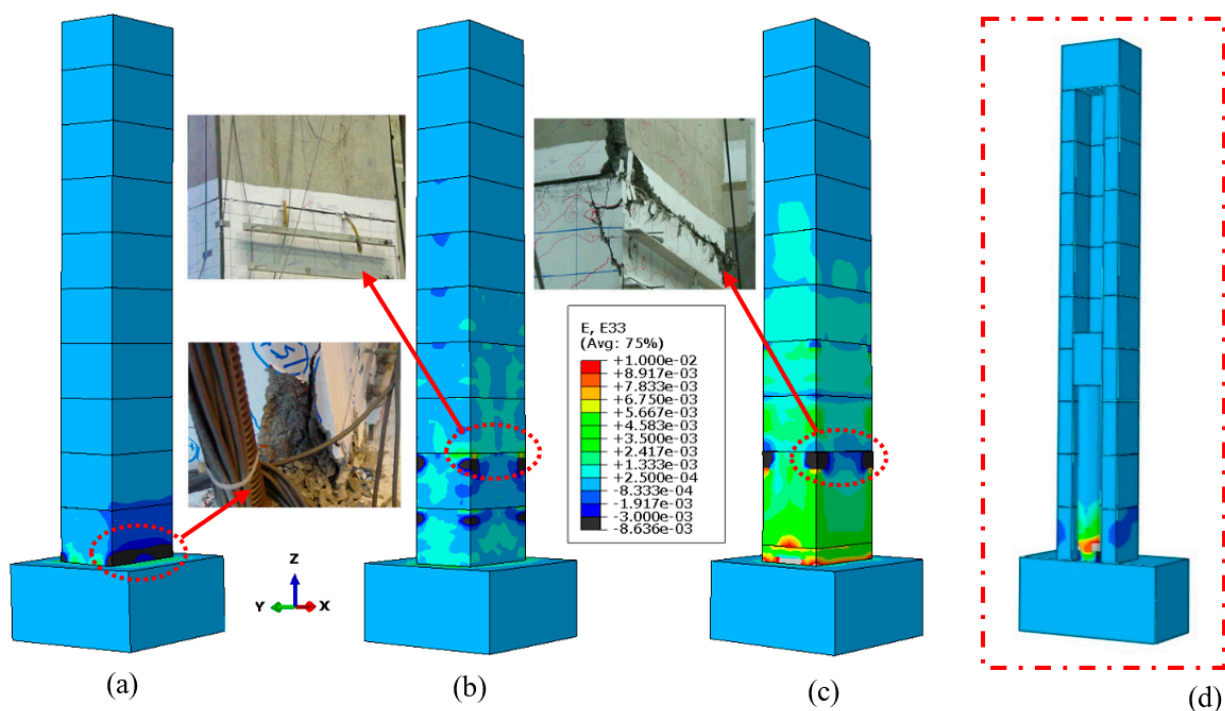


Figure 5. Damage pattern of the numerical models: (a) P1, (b) P2, (c) P3, and (d) proposed system (SC-R4%-PT10%).

4.3.2. Load–Displacement Hysteretic Curves

According to the findings of the experiments and the FE numerical calculations, the hysteric curves of the three prefabricated columns are shown in Figure 6. In the figure, the red dots represent the experimental test curves. In contrast, the continuous black lines represent the numerical model output. Regarding the initial stiffness, the stiffness after yielding, and the ultimate strength, the numerical findings shown in Figure 6 are consistent with the experimental results. Moreover, the stiffness degradation observed

in the tests is well reflected by the hysteresis curves of the FE models. The numerical FE model's hysteretic curves faithfully reproduce the residual displacement measured from the experimental test. Table 2 provides a quantitative analysis of the FE results and associated errors compared to the experimental test data for the lateral force at each drift level. It can be noted that the maximum average error between the predicted and experimental results for columns P1, P2, and P3 is below 8%, which is considered satisfactory. In general, the numerical models show considerable accuracy in predicting the lateral response of the precast segmental bridge columns.

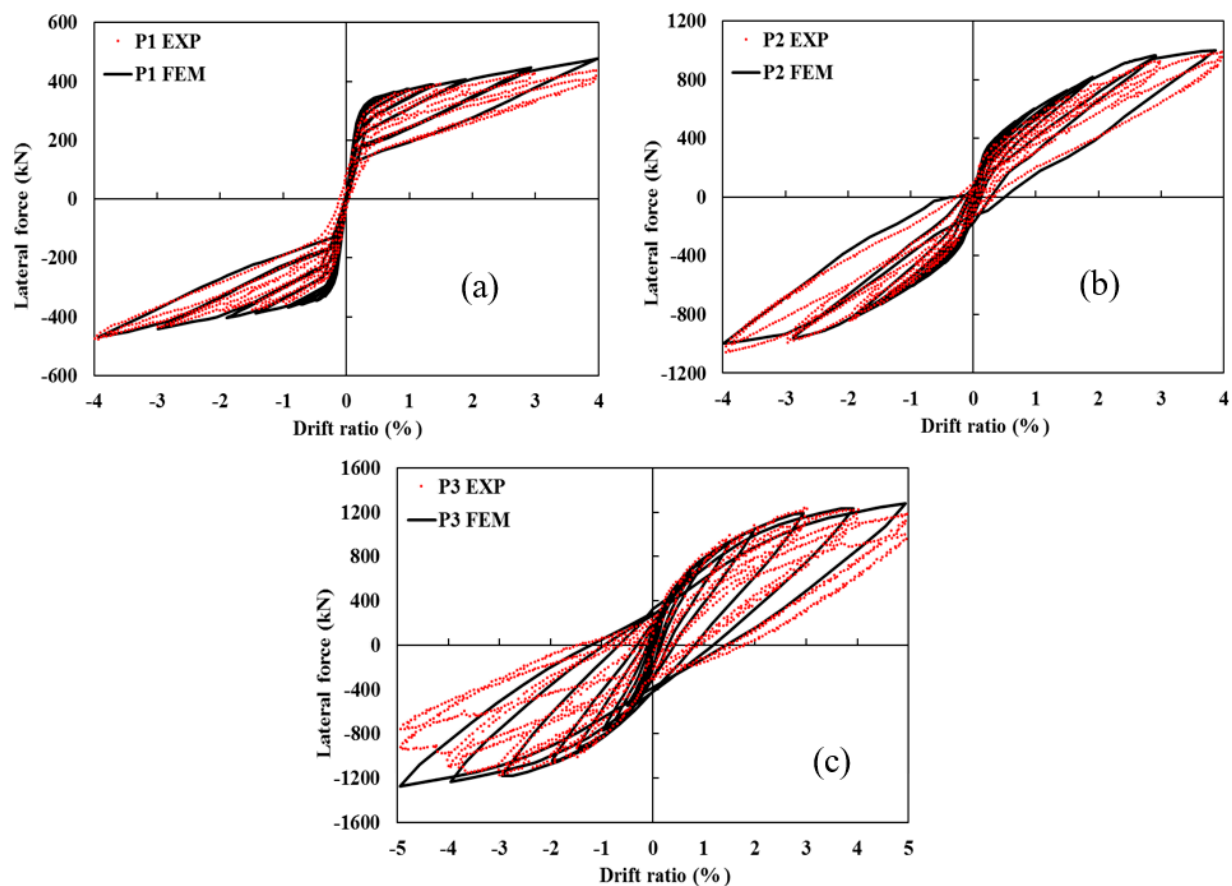


Figure 6. Comparison of hysteretic responses of experimental test specimens and FE models: (a) P1, (b) P2, and (c) P3.

Table 2. FE results and associated errors of lateral forces at each drift level.

Drift Ratio	P1			P2			P3		
	Exp (kN)	FEM (kN)	Error (%)	Exp (kN)	FEM (kN)	Error (%)	Exp (kN)	FEM (kN)	Error (%)
1	369	374.6	1.5	513	602.1	15.6	824.6	806	2.3
−1	−354.3	−368	3.9	−573.2	−602.4	5.0	−836	−824.2	1.4
1.5	392	389.5	0.6	652.4	730.8	11.9	991.2	899.3	9.3
−1.5	−376.3	−388.4	3.2	−722.9	−729.5	0.8	−1005.4	−928.3	7.7
2	396	407.4	2.9	750.2	816.9	8.8	1079	1026.4	4.9
−2	−379.8	−403.9	6.3	−817.1	−837.5	2.5	−1069.9	−1044.5	2.4
3	426	446.5	4.8	916.1	964.1	5.4	1231.7	1192.3	3.2
−3	−435.2	−442	1.6	−990.6	−956.5	2.7	−1169.5	−1176.7	0.6
4	435.5	376.3	13.6	980.2	999.8	2.0	1221.7	1229.7	0.7
−4	−474.6	−468.5	1.3	−1042.4	−997.4	4.3	−1098.4	−1231.8	12.1
5							1186.5	1281.5	8.0
−5							−933.2	−1273.8	36.5
Avg Error			4.0%			6.2%			7.4%

5. Parametric Study

Table 3 summarizes the four design parameters that were considered for the ED–CC. These parameters are the reinforcement ratio, the ratio of prestressing forces, the FRP wrap thickness, and the ED bars' unbonded length. Fifteen cases were included in the parametric analysis. Specimen P1 was selected to investigate the lateral behavior of the precast segmental column with ED–CC. The ED–CC consists of two prefabricated elements with a circular cross-section (see Figure 4), and its vertical length ratio is 40% of the main column length. This ratio was recommended by [31] to ensure stable lateral behavior. The circular cross-section was chosen to ensure priority in the confinement. Each fuse element is designed with a diameter of 600 mm. As shown in Figure 4, a solid block (the head of the ED–CC) connects the two fuse elements and acts as a cap beam for the ED–CC. The steel reinforcement of the ED–CC has a yield strength capacity of 235 MPa. The concrete of the ED–CC has a compressive strength capacity of 37 MPa, similar to that of the P1 specimen.

Table 3. Specimens with different design parameters.

No	Reinforcement Ratio %	Post-Tensioned Force Ratio %	Unbonded ED Bar Length (mm)	BFRP Thickness (mm)
SC-R2%-PT5%	2	5	200	NON
SC-R2%-PT10%	2	10	200	NON
SC-R2%-PT15%	2	15	200	NON
SC-R4%-PT5%	4	5	200	NON
SC-R4%-PT10%	4	10	200	NON
SC-R4%-PT15%	4	15	200	NON
SC-R6%-PT5%	6	5	200	NON
SC-R6%-PT10%	6	10	200	NON
SC-R6%-PT15%	6	15	200	NON
SC-R4%-U0	4	10	NON	NON
SC-R4%-U1	4	10	100	NON
SC-R4%-U2	4	10	200	NON
SC-R4%-U3	4	10	300	NON
SC-R4%-B0	4	10	200	NON
SC-R4%-2B	4	10	200	2
SC-R4%-4B	4	10	200	4
SC-R4%-6B	4	10	200	6

Note: specimens SC-R4%-PT10%, SC-R4%-U2, and SC-R4%-B0 have the same design parameters.

To increase the friction between the ED–CC and the foundation, it is necessary to increase the force in the tendons during the design. This ensures the maintenance of shear strength in the precast column and resistance to shear forces resulting from seismic actions [2]. However, the loads in the tendons can lead to high stress on the precast column during the rocking mechanism, decreasing the ED–CC's ductility [20]. Three prestressing force levels were carefully selected to investigate the effects of force ratio in ED–CC tendons on the response of the proposed system. These force ratios correspond to 5%, 10%, and 15% of the axial compressive strength capacity of the ED–CC ($A_g \cdot f'_c$), where (A_g) represents the gross section area of the ED–CC and (f'_c) represents the concrete compressive strength.

The prestressing force in the tendons of the ED–CC can be denoted as PT followed by the respective percentage; e.g., PT10% means that the prestressing force in the tendons is 10% of the axial compressive strength capacity of the column. The study investigates different reinforcement levels of 2%, 4%, and 6% for each prestressing force level. The parameter indicating the reinforcement ratio of the ED–CC is symbolized as R. For clarification, R4% means that the ratio of the reinforcement of the ED–CC is 4%. It is important to note that the different reinforcement ratios were achieved by modification of the cross-section area of the bar. At the same time, the number and arrangement of rebars remained unchanged.

To prevent early fatigue failure at low cycles, the unbonded length of the ED bars in the ED–CC at the rocking connection was adopted. The unbonded lengths of zero, 100, 200, and 300 mm were chosen to investigate how the ED bars in the ED–CC's unbonded length affected the proposed column's performance (as listed in Table 3). Among the listed cases

in Table 3, a length of 200 mm was left unbonded. The letter U expresses this parameter; for instance, U2 indicates that the ED bars in the ED-CC have an unbonded length of 200 mm.

Under lateral loads, the ED-CC allows a rocking mechanism. During the rocking of the column and at large displacements, the lower part of the ED-CC may suffer great damage due to the large compressive stress at the rocking interfaces. Therefore, to improve the maximum lateral strength capacity and control the damage of the ED-CC, the effects of three BFRP sheets' volumetric confinement ratios of 1.33%, 2.7%, and 4% were studied. These volumetric confinement ratios correspond to a thickness of 2 mm, 4 mm, and 6 mm of BFRP sheets. The maximum tensile strength and elastic modulus of the BFRP sheet were 2100 MPa and 91 GPa, respectively [32]. The height of the externally confined zone was 1000 mm from the top surface of the foundation, as shown in Figure 4b.

6. Results and Discussion

The results of the FE modeling for the proposed system compared to the reference specimen P1 are presented in the following section. The main outcomes analyzed and discussed include the failure mode, the hysteretic response, the capacity for energy dissipation, and the residual displacement.

6.1. Failure Mode

Compared to the damage pattern of the reference P1 column mentioned in Figure 5a, Figure 5d illustrates the proposed columns' damage mode at a drift ratio of 4%. The overall damage behavior of the proposed system is represented by the findings of the SC-R4%-PT10% columns. The strain levels of the proposed column RC segments are significantly lower than those of the P1 specimen. This is due to the redistribution of the opening between the column joints, which reduces the column segment strain. To the author's knowledge, all available ED devices increase the level of damage in the column segments. Therefore, adding ED-CC to the main column provides a damage control system for the precast bridge pier system. Figure 5d also shows that there are some tensile cracks (tensile strain) at the bottom of the ED-CC and concrete crushing (dark color) at the column toes of the ED-CC. These stresses result from the embedded ED bars, which transfer stresses to the surrounding concrete elements. The dark color indicates that the concrete cover was crushed at the toes of the ED-CC due to the rocking mechanism.

Figure 7 shows the ultimate tensile strain in the ED-CC at a drift ratio of 4% with different design details concerning the ED-CC. As shown in Figure 7 on the left, increasing the prestressing force level in the tendons results in a decrease in the maximum tensile strain on the ED-CC due to the increase in axial compressive stresses. This reduction is much clearer when the ratio of the reinforcement is 2%. If the prestressing force ratio is 10%, the maximum tensile strain value is almost the same when the reinforcement ratio increases from 2% to 6%. On the other hand, a prestressing force level of less than 10% (5%) leads to an increase in the maximum tensile stress with an increase in the reinforcement ratio. A prestressing force level of more than 10% (15%) reduces the maximum tensile strain by increasing the reinforcement ratio from 2% to 6%. The ED-CC with FRP confinements exhibited the lowest maximum tensile strain values of all the cases studied. As shown in Figure 7, increasing the FRP sheets' thickness caused a reduction in the maximum tensile strain values.

Figure 8 illustrates the ultimate compressive strain of the ED-CC at a drift ratio of 4% with different design details, including the reinforcement ratio, the amount of prestressing force, and the confinement thicknesses. As seen in the figure, the higher the prestressing force in the tendons, the higher the maximum compressive strain of the ED-CC. The increase in compressive strain is more obvious when the reinforcement ratio is 2%. This increase is due to the increased axial force, which exerts a large axial stress on the toes of the ED-CC during the rocking mechanism. Compared to the tensile strains shown in Figure 7, the effect of the prestressed force is more pronounced at the maximum compressive strain. Compared to SC-R4%-PT10%, the use of FRP sheets can lead to a considerable decrease in

the compressive strain. This decrease is directly related to the increase in the volumetric confinement ratio of the BFRP sheets from 1.33% to 4%.

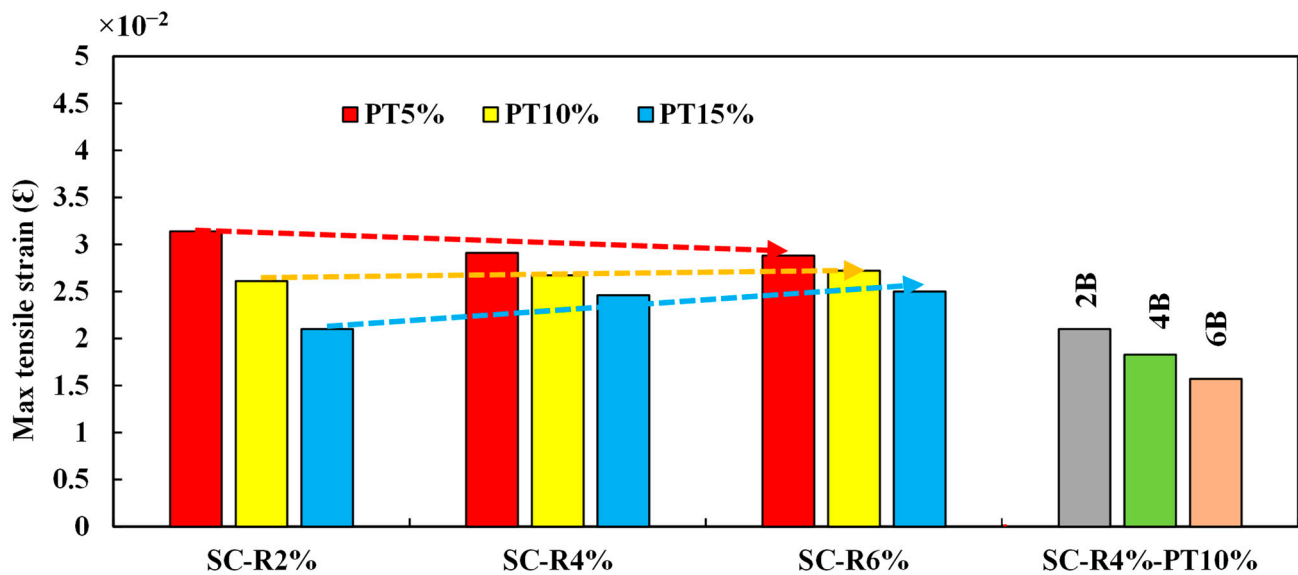


Figure 7. Maximum tensile strain of the concrete material of the ED-CC.

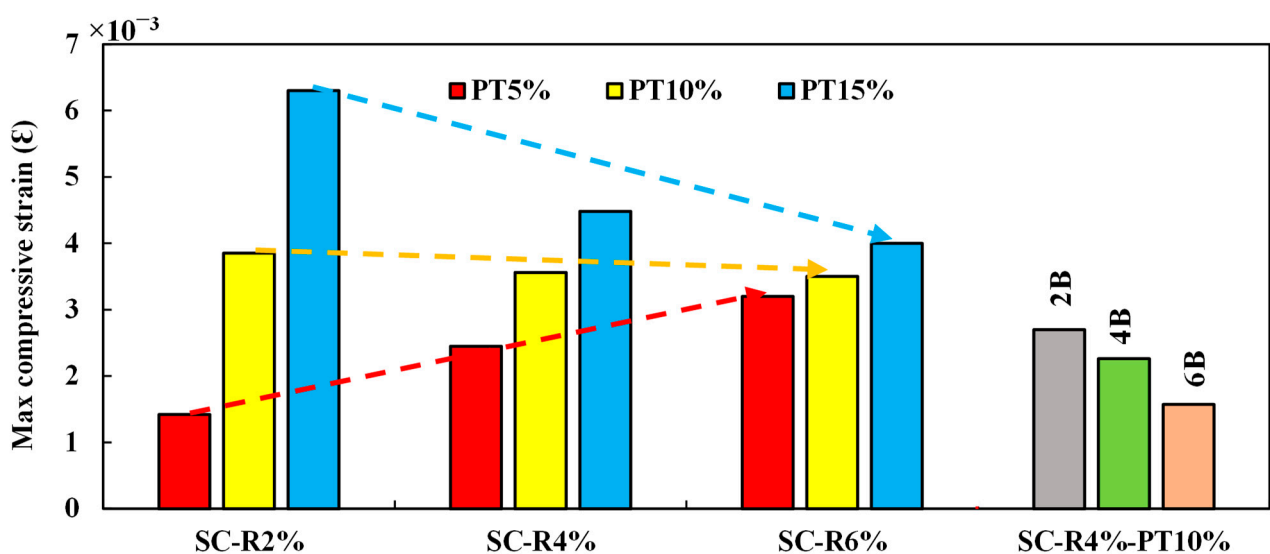


Figure 8. Ultimate compression strain of the concrete material of the ED-CC.

The plastic strain results available in ABAQUS outputs were used to measure the damage mode under the influence of the investigated parameters. Figure 9 presents the damage pattern of the ED-CC. From the figure, the damage of all models is based on the reinforcement ratio and the applied prestressing level. Considering the same reinforcement ratio, generally, an increase in the prestressing force level leads to a decrease in tensile stresses and an increase in the concrete compression stresses, especially at the toes of the ED-CC, as presented in Figure 9a. Considering a constant prestressing force ratio, increasing the reinforcement ratio from 2% to 6% results in an increase in the contribution of the ED-CC unit to the system lateral resistance and, thus, an increase in the tensile stresses. On the other hand, an increase in the reinforcement ratio from 2% to 6% leads to a decrease in the crushing of the concrete at the toes of the ED-CC. Regarding the impact of the unbonded length of the ED bars at a 4% reinforcement ratio and a prestressing force level of 10%, Figure 9b on the left shows the damage behavior of the ED-CC at

different unbonded lengths. As shown in the figure, the tensile cracks shift upwards as the unbonded length of the ED bars increases from zero to 300 mm. The reason for this is that increasing the unbonded length can lead to a redistribution of strains in the ED bars, which prevents localized stresses at the rocking connection, as seen in Figure 10. Consequently, by increasing the unbonded length, the tensile cracks are shifted to a higher part of the ED–CC. The reduction of strain in the ED bars allows the ED–CC to experience large lateral displacements without causing tensile fracture of the bars. For the models with FRP confinements, Figure 9c shows the damage mode of the ED–CC when the steel reinforcement ratio is 4%, the prestressing force is 10%, the unbonded length is 200 mm, and the thicknesses of the FRP sheet are zero, 2 mm, 4 mm, and 6 mm. From the figure, FRP confinement can significantly mitigate the damage level of the ED–CC with the main precast segmental column. Increasing the FRP thickness from 2 mm to 6 mm can successfully improve the compression resistance at the toes of the ED–CC, with no noticeable evidence of localized stresses. Ultimately, it can be concluded from the observed damage behavior in all studied cases that the prestressing force ratio, the unbonded length of the ED bars to the surrounding concrete, and the confinement details of the ED unit are the key parameters controlling the damage level of the ED system. Since the simulated cases represent a full-scale structure, a prestressing level of 10%, an unbonded length of 200 mm to 300 mm, and external confinement of the lower part of the ED core column with FRP sheets are recommended to control the damage level of the ED system.

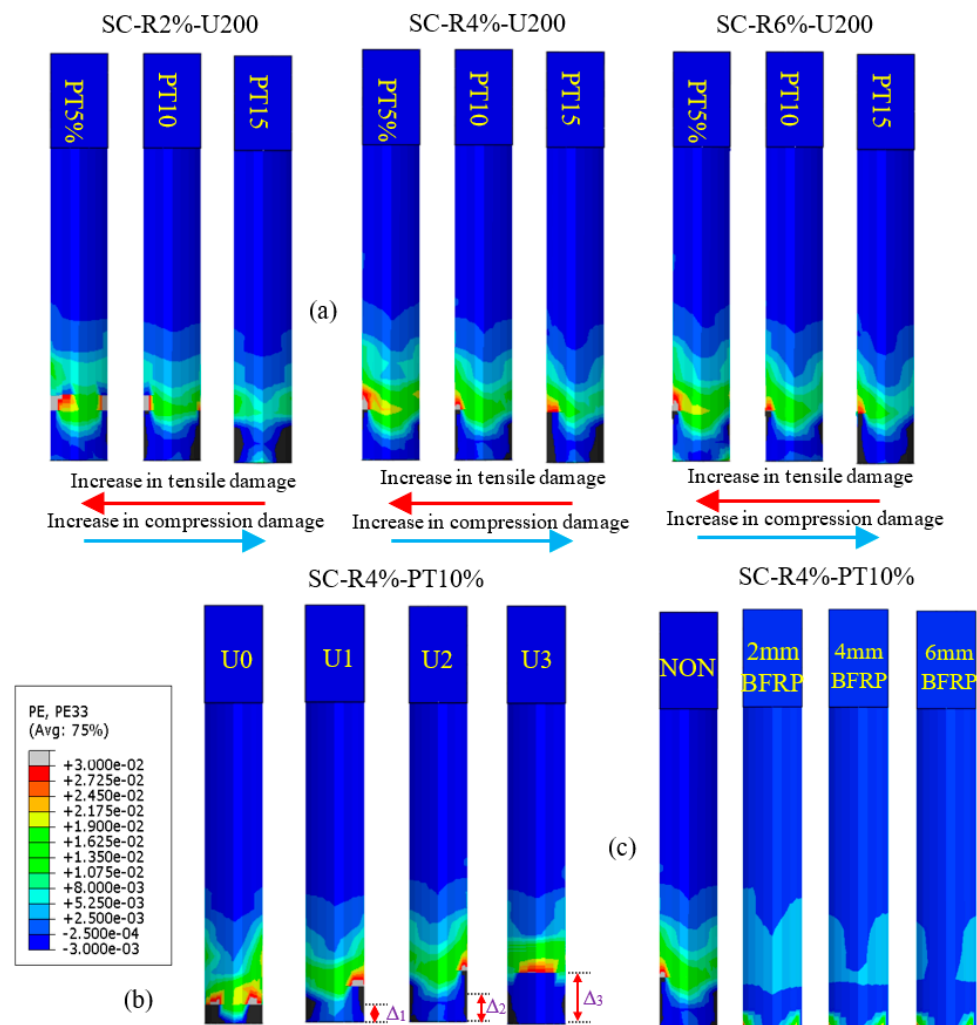


Figure 9. Plastic strain outputs for the ED–CC with various parameters, including (a) post-tensioned force level, reinforcement ratios, (b) unbonded length, and (c) confinement thicknesses.

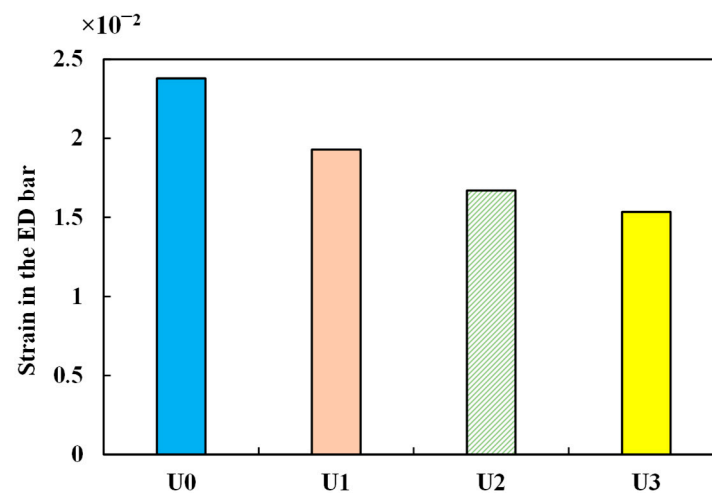


Figure 10. Strain in the ED bars of the ED–CC with different unbonded lengths of zero, 100 mm, 200 mm, and 300 mm.

6.2. Hysteretic Response

The lateral load–displacement hysteretic curve serves as a significant indicator for assessing the seismic performance of the proposed system. Figures 11–13 display the hysteretic curves for all simulated cases. The hysteretic curves of all samples are flag-shaped and exhibit high SC capability. This flag shape is the result of the yielding of the mild steel bars at the interface between the ED–CC and the foundation. As the reinforcement ratio of the ED–CC increases, the flag-shaped area also enlarges, as depicted in Figure 11. Each hysteretic curve can be divided into two phases. The first phase is linear up to the point where the curve yields, which physically corresponds to the opening between the column segments. The second phase occurs after the yield point and is characterized by non-linearity with the development of post-yield stiffness. This hardening behavior is a result of the increase in prestressing force with the applied lateral displacement. It is evident that, in comparison to the reference sample P1, the hysteretic curves of all samples display an upward shift in the yield point and an increase in the initial stiffness and post-yield stiffness, as illustrated in Figure 11. This improvement is attributed to that the ED–CC works in parallel with the main system in resisting the applied lateral loads. At a constant prestress of 10%, and with a reinforcement ratio of the ED–CC set at 2%, 4%, and 6%, the initial stiffness of the load–displacement hysteretic curves for the proposed column is respectively 23%, 31%, and 36% higher than that of reference specimen P1. Furthermore, compared to sample P1, the load–displacement yield point of the SC-R2%-PT10%, SC-R4%-PT10%, and SC-R6%-PT10% samples is increased by 30%, 51%, and 62%, respectively. Additionally, the use of the ED–CC increases the maximum lateral load of the proposed column by 31%, 48%, and 61% in comparison to specimen P1 for reinforcement ratios of 2%, 4%, and 6%, respectively. This increase is a result of the hardening of the reinforcement and tendons in the ED–CC. The calculated results of the proposed analytical model with the FEM results are also shown in Figure 11. As shown in the figure, the proposed analytical model can successfully predict the main characteristic points of the lateral response of the proposed system with good accuracy compared with the numerical FE results.

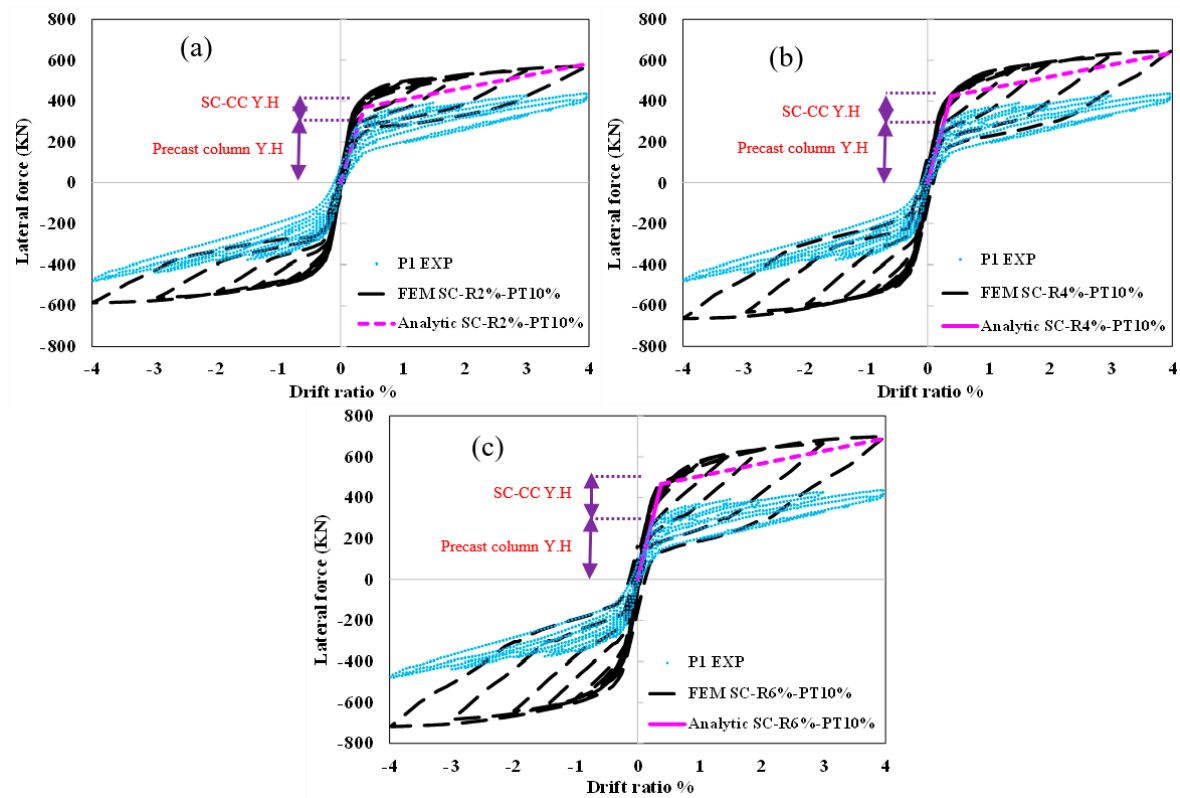


Figure 11. Comparison between the load–displacement hysteresis curves of the P1 sample and the proposed column, considering three different reinforcements for the ED–CC: (a) R2%, (b) R4%, and (c) R6%.

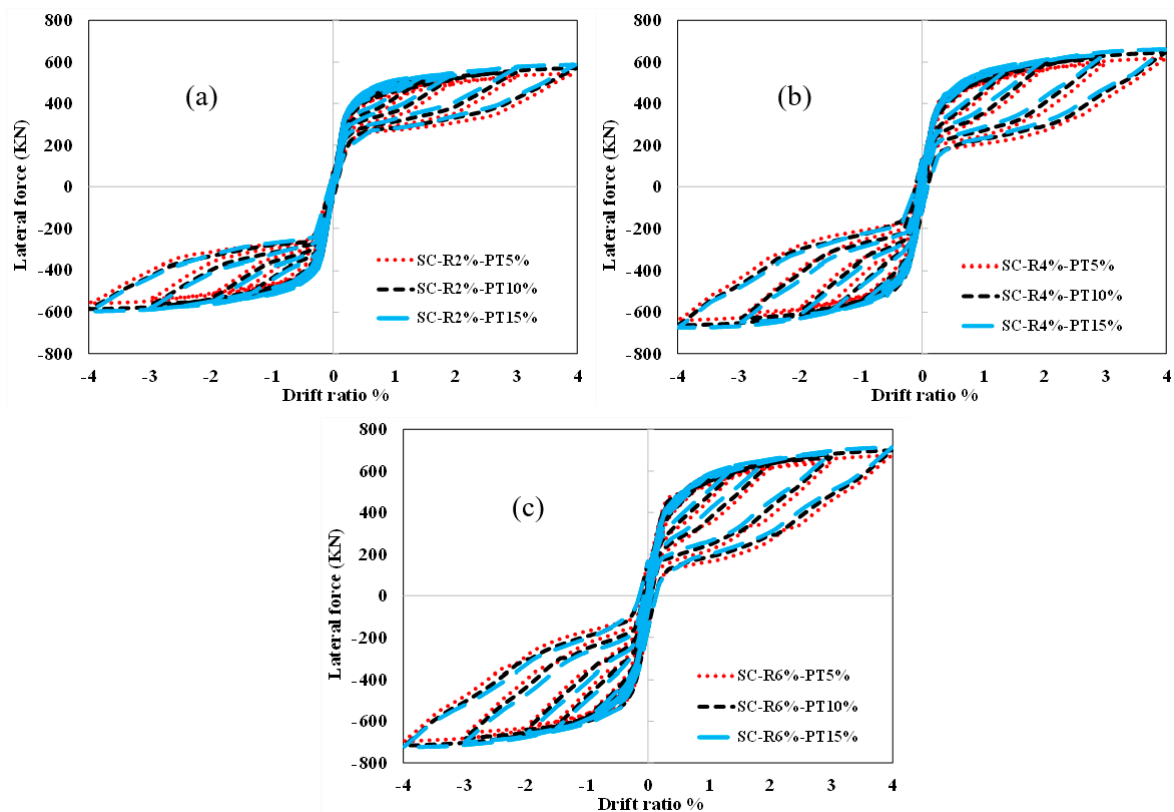


Figure 12. Hysteresis load–displacement curves of the presented column with post-tensioned force levels ranging from 5% to 15% and reinforcement percentages of (a) 2%, (b) 4%, and (c) 6%.

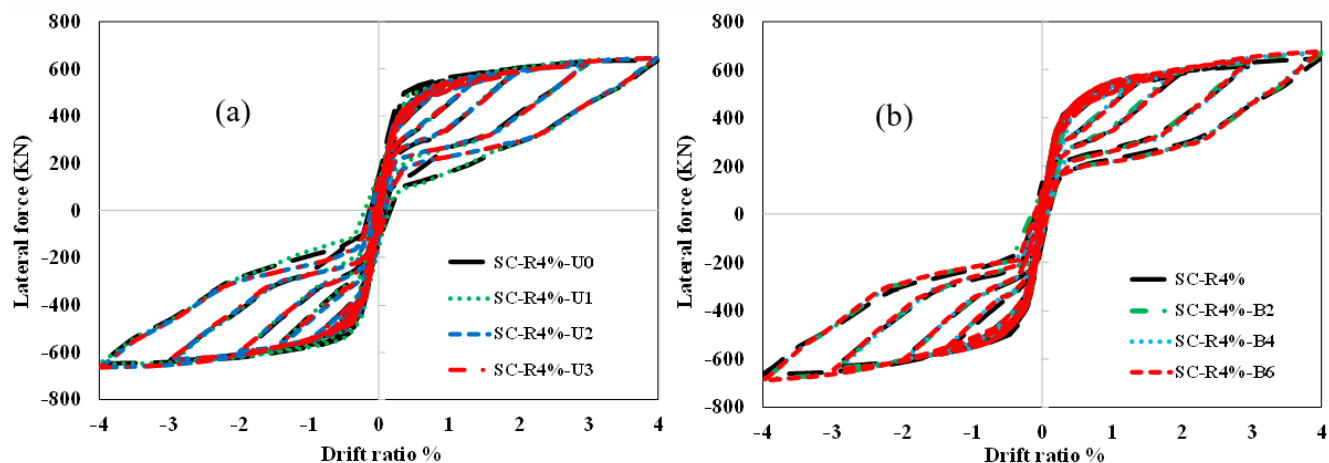


Figure 13. Hysteresis load–displacement curves of the proposed column under the effect of (a) unbonded length of the ED bars and (b) thickness of the FRP confinement of the SC hidden column.

Figure 12 illustrates the impact of varying reinforcement percentages of 2%, 4%, and 6%, as well as different prestressing force levels in the tendons (5%, 10%, and 15%), on the hysteresis performance of the proposed column, employing an ED–CC with an unbonded length of 200 mm. As shown in the figure, increasing the prestressing force level of the tendons leads to a slight increase in the corresponding initial stiffness, yield strength, post-yield stiffness, and maximum lateral resistance. Figure 12a illustrates that, at a reinforcement ratio of 2%, the entire system can withstand maximum lateral loads of 544 MPa, 570 MPa, and 587 MPa. These loads are associated with prestressing force levels of 5%, 10%, and 15%, respectively. The proposed column exhibits a maximum lateral load of 618 MPa, 645 MPa, and 660 MPa, which correspond to prestressing force levels of 5%, 10%, and 15%, respectively, when the reinforcement ratio is 4%. At the same prestressing force level, the maximum lateral load of the column is 675 MPa, 700 MPa, and 714 MPa when the reinforcement ratio of the ED–CC is 6%. This improvement is due to the increase in the lateral strength of the ED–CC.

For a constant prestressing force of 10% and a reinforcement ratio of 4% of the ED–CC, the hysteretic response of the proposed column is illustrated in Figure 13a, demonstrating the variations caused by adopting ED bars with different unbonded lengths, namely zero, 100 mm, 200 mm, and 300 mm. As shown in the figure, all hysteretic curves show identical initial stiffness, yield stiffness, and maximum lateral strength behavior. On the other hand, increasing the unbonded length reduces the yield strength of the hysteretic curves. Using an unbonded length of 200 mm or more leads to a stable behavior of the hysteretic curve with the stability of the curves' yield strength and the flag shape's area.

The hysteretic load–displacement curves of the proposed column with BFRP wrapping of varying thicknesses (zero, 2 mm, 4 mm, and 6 mm) can be seen in Figure 13b. As shown in Figure 13b, confining the ED–CC with different layers of FRP sheets has little effect on the load–displacement curves of the proposed column: the hysteretic curves exhibit a marked resemblance in terms of the initial stiffness, post-yielding stiffness, and the area enclosed by cycle loops.

6.3. Energy Dissipation

The equivalent viscous damping (EVD) ratio of all models with various design factors associated with the design details of the ED–CC is illustrated in Figures 14 and 15. Compared with the typical prefabricated P1 specimen, all specimens have a higher EVD ratio (from 8% to 13.5%). The improvement in the EVD ratio is due to the flag shape in the hysteric response of the proposed column. Figure 14 shows the EVD ratio of the introduced column with prestressing forces between 5% and 15% and a reinforcement ratio between 2% and 6%. As seen in the figure, the EVD ratio increases significantly with

increasing reinforcement ratio. The maximum EVD ratio of the columns SC-R2%-PT10%, SC-R4%-PT10%, and SC-R6%-PT10% with a reinforcement ratio of 2%, 4%, and 6% is 8.5%, 11%, and 13%, respectively. Furthermore, any decrease in the prestressing force of the tendons is associated with an increase in the EVD ratio of the proposed column, which is more pronounced when the reinforcement ratio is 6%.

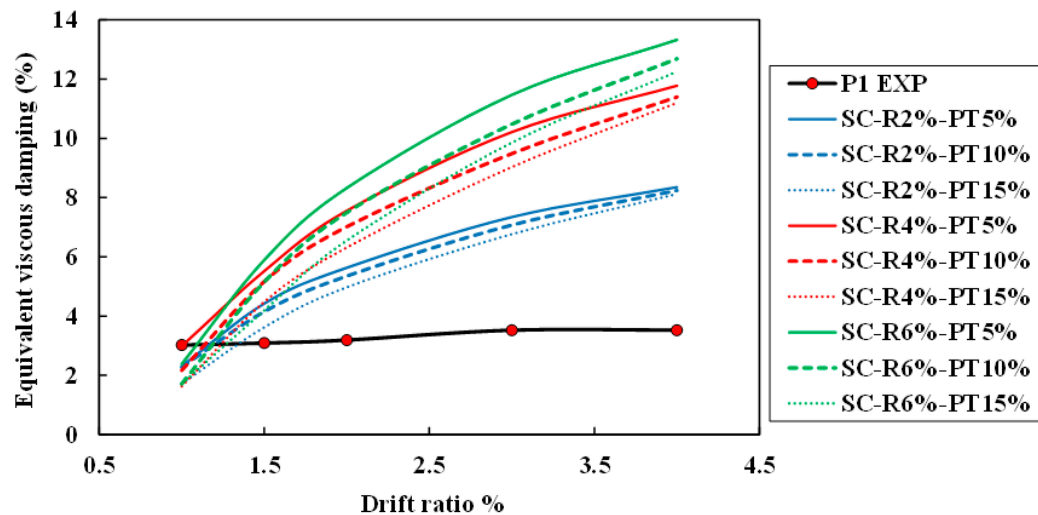


Figure 14. EVD ratio of the proposed column with post-tensioned force levels ranging from 5% to 15% and reinforcement ratio ranging from 2% to 6%.

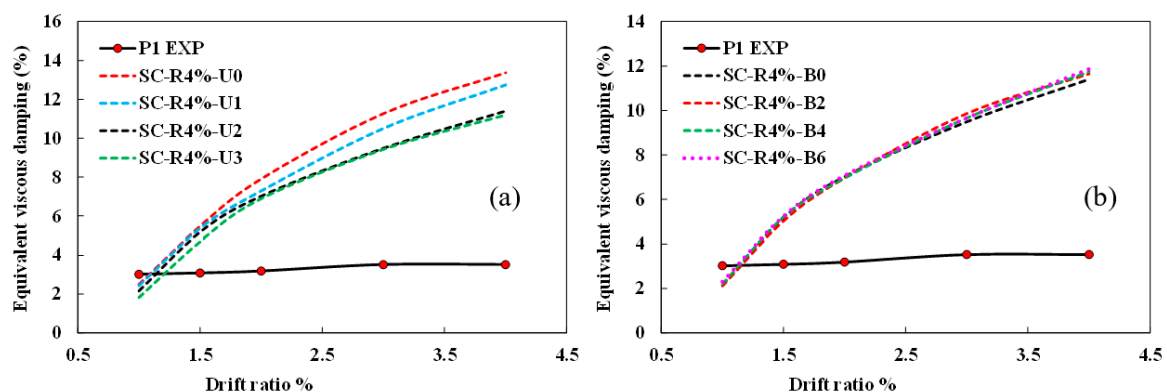


Figure 15. EVD ratio of the proposed precast bridge column under the influence of (a) unbonded length of the ED bars and (b) thickness of the FRP confinement of the SC hidden column.

For a constant prestressing force of 10% and a reinforcement ratio of 4% of the ED-CC, Figure 15a shows the EVD ratio of the proposed precast bridge column due to the use of ED bars with unbonded lengths of zero, 100 mm, 200 mm, and 300 mm. Compared to the fully bonded reinforcement, there is no clear effect of the unbonded length on the EVD ratio at column drift equal to or less than 2%. With further loading, using an unbonded length of >100 mm leads to comparable EVD ratios, which are lower than the fully bonded case. Figure 15b shows the EVD ratio of the proposed column due to the confinement of the ED-CC with BFRP sheets of zero, 2 mm, 4 mm, and 6 mm thickness. From the figure, the EVD ratio of the ED-CC columns is independent on the confinement ratio, and this behavior can be attributed to the comparable hysteretic responses of the studied columns with such confinement details.

6.4. Residual Deformation

Figure 16 depicts the correlation between the residual displacement of the introduced precast bridge column and the reinforcement ratio of the ED–CC. Considering the fact that the main segmental column and the ED–CC are self-centered, the residual displacement values of all columns are highly controlled, as shown in Figure 16. Although the reinforcement ratio is increased from 2% to 6%, the residual drift values of these specimens are still far from the resilience limit (i.e., less than 1%). Similarly, reducing the prestressing force in the tendons of the ED–CC unit from 15% to 5% led to a very limited increase in the residual displacement. The residual deformation of all columns at an ultimate displacement ratio of 4% is less than 0.16%.

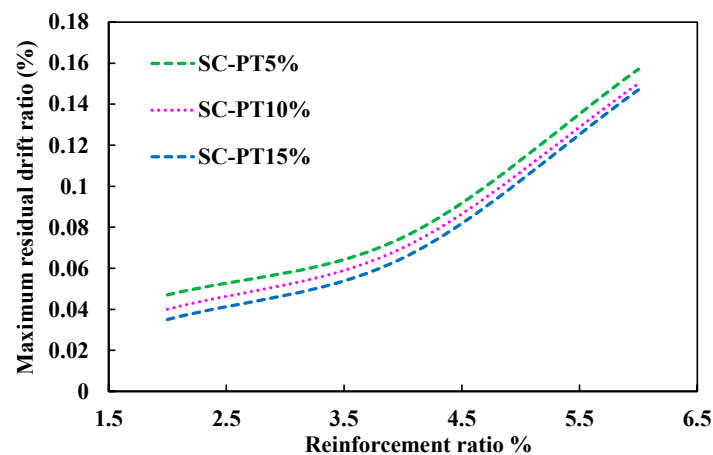


Figure 16. Residual deformation versus reinforcement ratio.

7. Conclusions

This study aims to present a new generation of seismic-resisting systems applicable to the construction of substructure systems of modern bridges that include SC precast hollow-core segmental columns. The hollow core of such columns is proposed to be provided with an SC RC precast unit, integrally combined with mild steel bars as a source of ED. An extensive FE numerical study was conducted on almost full-scale hollow-core segmental bridge columns to evaluate the behavior of the proposed system. The subsequent conclusions can be made:

- (1) Adding the proposed ED system to the main seismic-resisting system can ensure the required energy dissipation level and improve the system's lateral resistance. Compared to the typical precast hollow-core segmental column, a 6% reinforcement ratio of the ED unit can cause a 60% increase in lateral resistance.
- (2) The entire proposed system is a new resilient system with the ability to dissipate energy without compromising the SC capacity of the main resisting system. Up to a 4% drift ratio, all examined cases showed a residual displacement of less than 0.16%.
- (3) Unlike traditional/available ED systems, the proposed ED system protects the lower segments of the main resisting system from localized damage during the rocking mechanism.
- (4) The prestressing force ratio, the unbonded length of the ED bars to the surrounding concrete, and the confinement details of the ED unit are the key parameters controlling the damage level of the ED system. Depending on the scale of the simulated columns, design recommendations can be as follows:
 - (a) The height of the ED–CC can be postulated to be 40% of the complete height of the main SC column. This coefficient has proven to be efficient in the current study, guaranteeing structural harmony and stability.
 - (b) The ED–CC reinforcement ratio, ranging from 2% to 6%, assumes a paramount role in governing both the supplementary capacity of the ED and the lateral strength of the system.

- (c) When the ED bars ratios are 2% to 6%, a prestressing level of 10% can ensure a balance between the tensile and compressive stresses of the concrete material of the ED–CC.
 - (d) An unbonded length of 200 mm to 300 mm can lead to a redistribution of the stresses so that the maximum tensile stress (breaking point) of the ED bars can be delayed to withstand larger lateral drift (4%).
 - (e) The external confinement of the lower part of the ED core column with FRP sheets is a key design parameter to increase concrete compressive strength at high drift ratios.
- (5) The proposed analytical model successfully predicts the characteristic points of the lateral response of the proposed system based on the superposition concept. The most common influential design parameters, such as reinforcement ratios and details, concrete dimensions, and material characteristics, can be adopted to define the contribution of the ED system to the main seismic system (energy dissipation capacity and lateral strength).

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