Seismic performance of pre-fabricated segmental bridges with an innovative layered-UHPC connection

Ruiling WANG¹,² · Biao MA¹ · Xu CHEN³

Received: 14 November 2021 / Accepted: 22 June 2022 / Published online: 1 August 2022
© The Author(s), under exclusive licence to Springer Nature B.V. 2022

Abstract
Ultra-high-performance concrete (UHPC) has been regarded as promising alternative to provide reliable connections between different segments (e.g., columns and pier footing/cap) during accelerated bridge construction (ABC) procedures. This paper proposes an innovative layered-UHPC connection for the pre-fabricated segmental (PFS) piers, whose seismic performance was validated through quasi-static experiments. Based on the test results, design procedure is presented for PFS pier with this connection, to overcome the observed drawbacks and achieve high-level performance. The layered-UHPC connection ensures the emulative performance of pre-fabricated bridges as cast-in-place (CIP) ones, as well as provides greater economic efficiency than traditional UHPC connections. Based on experimental results, key issues concerning this connection, including the tensile behavior of UHPC, height of connection region, thickness of UHPC layer and steel bars in grouting bed, are presented and discussed. Then a seismic design procedure is proposed utilizing the capacity protection philosophy. The layered-UHPC connection is expected as capacity-protected component without damage, since it provides anchorage for longitudinal steel bars. While the pre-fabricated region is designed as ductile component undergoing nonlinearity during strong earthquakes. Following the detailed elaborations about the design philosophy, requirement and implementation steps, this procedure is further presented through illustration examples using PFS piers with various heights. The results show that PFS piers designed according to this procedure could meet the requirement under both frequent and rare earthquakes. Note that the PFS piers with this layered-UHPC connections could be designed similar to and emulative as CIP ones, which is believed friendly to designers in engineering practice.

Keywords Pre-fabricated segmental bridges · layered-UHPC connection · Quasi-static experiment · Capacity protection · Seismic performance

1 Shanghai Municipal Engineering Design Institute (Group) Co., LTD, 200092 Shanghai, China
2 Shanghai Municipal Engineering Design Institute, 200438 Shanghai, China
3 International Research Institute of Disaster Science, Tohoku University, 980-8576 Sendai,
1 Introduction

Accelerated Bridge Construction (ABC) techniques are being popular in recent years (Ou et al. 2010; Tazarv and Saiidi 2015; Wang et al. 2018), which improves construction quality and safety (Ameli et al. 2016), while minimize the environmental impacts (Billington and Breen 2000), traffic disruption (Perotti et al. 2013), onsite construction time (Tekie and Ellingwood 2003), as well as life-cycle cost (Caltrans 2019). In current engineering practice, the pier columns are usually pre-fabricated in factories, and then connected to the cast-in-place (CIP) foundations in engineering sites (Li and Liu 2003). Therefore, the connections between pre-fabricated columns and foundations are especially significant for those bridges located in regions with high seismic hazard level, which should be reliable enough during earthquake events. These connections are commonly categorized as emulative and non-emulative ones, depending on their effects on structural seismic performance (White and Palermo 2016; Kurama et al. 2018; Wang et al. 2020). The emulative type enables that the pre-fabricated segmental (PFS) bridges perform similar with their CIP counterparts during earthquakes, as described by Haber et al. (2017) and Marshall et al. (2020). These connections integrated the separated elements as a whole with grout or late cast concrete (Ameli et al. 2015), including pocket and socket connections, mechanical coupler connections, grouted duct connections and so on (Steuck et al. 2009; Haraldsson et al. 2013; Haber et al. 2014). The non-emulative connections usually employ un-bonded post-tensioned tendons or high-strength steel bars to integrate different elements, which could enhance the rocking performance of columns under earthquake excitations (Ou et al. 2010; Moustafa and ElGawady 2018; Wang et al. 2018, Markogiannaki et al. 2017; Chen and Li 2020). Generally, emulative connections are more popular in current engineering practice, since they possess better performance in structural reliability and construction tolerance (Wang et al. 2020).

Ultra-high-performance concrete (UHPC) is a new generation of cement-based materials and has gained increasing attention in the field of civil engineering (Feng et al. 2019; Hung et al. 2019; Siwowski and Rajchel 2019). Compared with conventional concrete, UHPC possesses superior performance in terms of both compressive and tensile properties, as well as durability and bond behavior (Huang et al. 2019; Meng et al. 2020). Therefore, this material provides as an alternative solution for the connection between pre-fabricated pier columns and foundations of bridges constructed with ABC techniques. Ichikawa et al. (2016) employed UHPC segments at the plastic hinge region of piers to minimize the damage during earthquakes. The results obtained from cyclic and hybrid dynamic tests for three specimens showed that these UHPC segments could effectively improve the seismic performance of columns, which experienced neither significant spalling nor crushing. Tazarv and Saiidi (2015) conducted cyclic tests for a precast column connected to foundation with UHPC filled duct and pointed out that this type of connection was emulative of the CIP counterpart.

Wang et al. (2019) proposed an innovative pocket connection utilizing noncontact lap-spliced bars and UHPC grout for segmental pier columns and conducted quasi-static cyclic tests to investigate the seismic performance of this connection. By comparing experimental results with numerical results of a CIP column, the authors pointed out that the proposed connection served as an emulative one, which made the precast columns yield comparable seismic performance with its CIP counterpart. Wang et al. (2020) then developed refined numerical models of the specimen, which was verified through comparing both hysteretic and local responses obtained from the experiment. With this model, the deformation of the
A segmental pier was further decomposed into various components of column deformation, including flexure, shear and bar-slip. However, this type of connection had an expanded cage at the base of precast pier column and consumed a large amount of UHPC material, which is not that economic efficient. Additionally, all these aforementioned investigations mainly concentrated on the seismic performance of segmental bridge piers using UHPC connections, through either numerical or experimental studies (e.g., Mohebbi et al. 2017). While scarce literature has been provided concerning the seismic design procedures of this type of bridges to date.

This paper proposes an innovative connection (termed as layered-UHPC hereafter), as well as the corresponding seismic design procedure for bridges with pre-fabricated pier columns using this connection. This innovative connection consumes less UHPC material than previous ones and thus leads to higher economic efficiency, while the seismic performance can be emulative as CIP and previous UHPC columns following the proposed design procedure. The configuration and quasi-static experiments of PFS piers with this innovative UHPC-layered connection is first introduced. Then the key issues concerning the connection are discussed, including the tensile behavior of UHPC, height of connection region, thickness of UHPC layer and steel bars in grouting bed. Based on these results, the seismic design procedure is proposed utilizing the capacity protection philosophy. In this procedure, the layered-UHPC connection regions are regarded as capacity-protected components, while the pre-fabricated standard regions are designed as ductile ones experiencing nonlinear responses and damage during earthquake events. The main design philosophy and requirements are presented, followed by the detailed description of the implementation steps. Finally, the effectiveness of this proposed procedure is validated by illustration examples, in which PFS piers with various heights are designed, and their seismic performance subjected to earthquakes with different hazard levels is assessed accordingly.

2 The layered-UHPC connection and quasi-static tests

This section first describes the configuration and main advantages of the proposed layered-UHPC connection. Then the quasi-static tests are introduced, including the design details, test protocols and measured responses. Note that the main objective of the current study is to provide an overall understanding of seismic performance and design of PFS piers with the layered-UHPC connection. Therefore, the experimental results are presented briefly, and additional work is on the way concerning further and more detailed analysis of these tests.

2.1 Configuration and advantages

In the previously proposed UHPC-type connection for PFS piers, an expanded cage filled with UHPC material was designed at pier base, to assemble pre-cast columns and foundations (Xu et al. 2021). While offering reliable connection between different segments, this design consumes large amount of UHPC of high cost, which is not that economic efficient. Therefore, the current study proposes an innovative layered-UHPC connection as an alternative. As shown in Fig. 1, the pier with this connection is composed of pre-cast concrete column (named as ’standard region’) and layered-UHPC connection region, while a grouting bed of several centimeters is employed between the pier base and the foundation.
Besides the higher economic efficiency due to less consumption of UHPC material, this connection could facilitate the construction process as well. Since the UHPC layer provides anchorage for overlapped longitudinal steels extended from pier column and pile cap, no lashing or welding is required for these steel bars. Additionally, the state of UHPC could be observed conveniently as it is casted at outer surface of the proposed connection, which is favored for inspection of the construction quality and post-earthquake damage. While UHPC layer encloses the conventional concrete, it also provides further confinement and thus improves the corresponding performance.

2.2 Quasi-static experiments

2.2.1 Specimen details

With the scale factor of 1/2.5, the clear column height of the specimen is 5.6 m as presented in Fig. 2, representing the 14.0 m prototype. The heights of pier cap and foundation are 0.9 m and 0.6 m, respectively, leading to a total specimen height of 7.1 m. The dimensions of cross sections are 0.8 m × 0.8 m, 0.9 m × 0.9, and 2.4 m × 2.4 m for the pier column, pier cap, and foundation, respectively. Correspondingly, the masses of the column, pier cap, and foundation are 9.3 t, 6.3 t, and 9.0 t, respectively. The foundation is rigidly fixed to the ground utilizing anchor bolts, and the column is integrally casted with the pier cap.

While the general dimensions of specimen (e.g., height and section dimensions) can be determined according to the scale factor, the reinforcement should be designed achieving equivalent sectional flexural stiffness (Chen et al. 2018) as shown in Fig. 2. 8 pairs of Φ32 HRB400 twin-bars are used as longitudinal reinforcement for the section of standard region (A-A section), while another 8 pairs of Φ36 HRB400 twin-bars are extended from the foundation into the UHPC connection region (B-B section). The corresponding steel ratios in A-A and B-B section are thus about 2.0% and 4.5%, respectively. Φ16 HRB400 steels are used providing transverse reinforcement, with spacings of 0.1 and 0.2 m inside and outside the intensified region, leading to volume transverse steel ratios of 1.0% and 0.5%, respectively. According to the configuration and diameters of the deployed steel bars, the height of layered-UHPC connection region at pier base was determined as 0.5 m based on previous pulling out tests for steel bars and UHPC materials, which would be introduced.
in the following parts. The thickness of UHPC varied from 0.14 m at the top of connection region to 0.18 m at the column base.

The measured material properties utilized in the specimen, including steel, conventional concrete and UHPC, are presented in Tables 1 and 2. By comparing data in these tables, UHPC is observed possessing significantly greater strength than conventional concrete, in terms of both compression and tension. Note that more information concerning measuring the tension properties of UHPC material will be presented in the following section.

### 2.2.2 Test setup, instrumentation and loading protocols

Figure 3 plots the quasi-static test setup of the specimen, where the lateral cyclic loads are exerted by a servo-hydraulic actuator. A hydraulic jack is used to prevent potential lateral slip of the foundation during loading. Two symmetrically deployed vertical actuators, each

---

**Table 1** Properties of steel bars

| Type          | Yield stress (MPa) | Ultimate stress (MPa) |
|---------------|--------------------|-----------------------|
| Longitudinal  | 425                | 613                   |
| Transverse    | 410                | 602                   |

**Table 2** Properties of UHPC and concrete (C40)

| Material                  | Compressive strength (MPa) | Tensile strength (MPa) | Young’s modulus (GPa) |
|---------------------------|----------------------------|------------------------|-----------------------|
| UHPC                      | 135.8                      | 13.5                   | 45.7                  |
| Concrete (C40)            | 45.1                       | --                     | 38.9                  |
applying 410 kN force, are employed realizing axial compression ratio of 8.3% at the pier base. Note that the vertical forces are determined to ensure the axial compression ratio at pier base identical to that of the prototype, which is generally less than 10% in highway bridges (Chen et al. 2020; Fan, et al. 2022; Chen and Xiong 2022).

During the testing procedure, the lateral displacement at the centroid of pier cap and mid-height of column is measured by displacement transducers. The strain gauges are used to record the strain levels of longitudinal steels at critical section. Note that to avoid adverse effects on the anchorage performance between UHPC and steels, no gauges are implemented in the layered-UHPC connection region.

The loading protocol is plotted in Fig. 4, which shows a combined force-controlled and displacement-controlled manner during the cyclic loading. During initial elastic response stage, force-controlled procedure is employed, while the displacement-controlled manner is more effective and thus utilized when the specimen starts to yield (Xu et al. 2021). In current tests, a total of twelve loading levels are contained in Fig. 4, in which two force-controlled levels are first conducted up to lateral force of 250 kN when yielding of longitudinal steels is expected. Then ten displacement-controlled loadings are carried out, initiating with lateral displacement of 40 mm, and terminating at lateral displacement of 360 mm leading to final collapse, which is defined as the lateral force lower than 80% of its peak value. During each loading level, three cycles are conducted to investigate the strength and stiffness degradation of the specimen.

2.2.3 Experimental results

The observed damage for some typical loading levels is presented in Fig. 5, while the description for the whole procedure of each level is listed in Table 3. In the figure and table, both the lateral displacement and the drift ratio normalized by clear column height (5.6 m) are provided. From Table 3, the cracks are first observed occurring and developing in the standard regions composed of conventional concrete, followed by slighter cracks in
layered-UHPC connection region, as well as cracks in grouting bed between pier base and foundation.

After the implementation of the first displacement-controlled loading level, the specimen starts to yield with lateral displacement increasing to 80 mm, and the crack initiates in grouting bed at this level as well (Fig. 5 (a) and (d)). While the lateral force reaches peak value (432 kN) at lateral displacement of 200 mm, it starts to decrease at the Level 8 loading shown in Fig. 5 (b) and (e), in which the crack width of grouting bed increases to 10 mm. At Level 11 with displacement of 340 mm, the lateral force decreases to 84% of the peak value, indicating approaching final collapse. Obvious local concrete spalling is presented in Fig. 5 (c) in the standard region, and the crack in grouting bed reaches about 15 mm. Note
that these cracks in grouting bed could drastically increase the strain of longitudinal steel bars and might lead to pre-mature collapse of the whole structure.

Generally, more cracks could be observed in standard region than in the UHPC connection region as presented in Fig. 5 (a~c), which implies that the damage is mainly shifted into the standard region, thus protecting the layered-UHPC connection at pier base during earthquakes. This feature is favored in engineering practice, since less damage in UHPC connection region will help to provide more reliable anchorage of longitudinal steel bars, providing better performance of PFS piers with this type of connection. To further improve the reliability of anchorage, cracks are required to be fully avoided in the UHPC connection region in the design procedure that will be elaborated in the following section.

Table 3 Observations for each loading level

| Loading level | Lateral force (kN) or displacement/ drift (mm/%) | Observations |
|---------------|-----------------------------------------------|--------------|
| Force-controlled | 1 150 | flexural cracks in standard region distributed within 0.8~1.3 m above the pier base; no damage in UHPC connection region and grouting bed |
| 2 250 | more cracks in standard region; minor cracks in UHPC connection region and grouting bed |
| Displacement-controlled | 3 40 (0.71%) | development of cracks in standard region; longitudinal steels nearing yielding |
| 4 80 (1.43%) | more of cracks in standard region; occurrence of cracks in foundation; initiation of crack in grouting bed |
| 5 120 (2.14%) | development of cracks, with maximum width about 2 mm; |
| 6 160 (2.86%) | development of cracks, with maximum width about 3 mm; slight spalling of cover concrete in standard region; grouting bed with cracks width about 5 mm |
| 7 200 (2.57%) | standard region with maximum crack width about 4 mm; peak lateral force (432 kN); grouting bed with crack width about 7 mm |
| 8 240 (4.28%) | development of cracks; decrease of lateral strength; grouting bed with crack width about 10 mm |
| 9 280 (5.00%) | more obvious concrete spalling at 1.0 m above pier base; grouting bed with crack width about 13 mm |
| 10 320 (5.71%) | substantial development of multiple cracks in standard region; significant concrete spalling |
| 11 340 (6.07%) | obvious cracks in foundation; grouting bed with crack width about 15 mm; lateral force reducing to 84% of peak value, nearing to final collapse |
| 12 360 (6.42%) | Extremely significant concrete spalling, with exposure of transverse steels; |
The hysteretic responses and skeleton curve between lateral force and displacement for the specimen are presented in Fig. 6. Consistent with the statement in Table 3; Fig. 6 (a) shows that the peak lateral strength of the specimen reaches 432 kN corresponding to the displacement of 200 mm, followed by a descending trend with decreased stiffness. Finally, the lateral force reduces lower than 80% of peak strength, implying the structural collapse. Additionally, the hysteretic loops of the specimen with layered-UHPC connection are stable and comparatively full without obvious pinching effect, implying reliable energy dissipation performance.

The backbone curve of lateral force and displacement in positive direction is shown in Fig. 6 (b) for illustration, which is obtained by connecting peak force during each loading level. The corresponding bi-linear idealization is computed using equal energy principle and presented as well for comparison. From this figure, the displacement and force for the equivalent yielding state are observed as 91 mm and 402 kN, respectively. The corresponding displacement ductility ratio at ultimate (collapse) state is about 4.0, which is comparable to that of CIP piers (approximately 4–5). Similarly, the initial equivalent stiffness can be computed as 4.42 kN/mm from this figure, which finally decreases to a scant stiffness of 1.12 kN/mm at the final collapse state (360 mm).

Based on the aforementioned results, the plastic behavior of the specimen is shifted into the RC standard region above UHPC connection region. This feature ensures the reliable anchorage between UHPC material and steel bars during cyclic tests, avoiding structural collapse due to bond failure. While the proposed layered-UHPC connection could provide similar displacement ductility to conventional CIP piers as well, indicating emulative performance with its CIP counterparts.

3 Essential issues of the layered-UHPC connection

Based on the previous experimental observations and results, several essential issues of the layer-UHPC connection are concluded and discussed in this section. The tensile behavior of UHPC material, height of layered-UHPC connection, thickness of UHPC layer, and steel bars in grouting bed are introduced sequentially.

3.1 Tensile behavior of UHPC material

One of the most significant advantages of UHPC over conventional concrete is its superior tensile behavior. According to the material tests conducted in the quasi-static experiment, as well as previous studies (Wang et al. 2021a, b), the maximum tensile strength of UHPC
could reach above 10 MPa, with ultimate tensile strain up to 3000 $\mu$e. Therefore, although the tensile capacity of conventional concrete is generally neglected in earthquake engineering, it should be incorporated for PFS piers using UHPC material, due to the considerable resistance provided for bending and tension.

Note that these tensile properties of UHPC were measured through uniaxial tensile experiments (shown in Fig. 7), rather than splitting tensile test that is usually conducted for conventional concrete. As suggested by SIA2052 (MCS-EPFL 2016), four parameters could be employed to represent the tensile performance of UHPC, including ultimate stress ($f_{Ute}$) and strain ($\varepsilon_{Ute}$) for linear-elastic stage, and maximum stress ($f_{Utu}$) and corresponding strain ($\varepsilon_{Utu}$), as illustrated in Fig. 7 (b).

3.2 Height of layered-UHPC connection region

As mentioned previously, the layered-UHPC connection region is expected to provide anchorage for longitudinal steels, requiring sufficient height to avoid failure caused by pulling out of steels. Pullout experiments were conducted to investigate the anchorage length required between UHPC and steel bars, in which the anchorage length was regarded as sufficient if fracture of longitudinal steels happened in prior to pulling out failure.

Based on the test results, the anchorage length was found related to the diameter ($d$), deployment, and stress type of longitudinal steel bars: (1) when $d$ is less than 25 mm, anchorage length greater than 4.5$d$, 3.5$d$, 7.5$d$ and 6.0$d$ are sufficient for tensile, compressive, overlapped tensile and overlapped compressive steel bars, respectively; (2) when $d$ increases over 25 mm, these values are suggested as 5.0$d$, 4.0$d$, 8.5$d$ and 7.0$d$, respectively. Note that for the twin steels used as in the quasi-static experiments, the $d$ values should be considered as the equivalent diameters leading to the same cross section area.

Note that in engineering practice, greater values of anchorage length might be adopted for conservative purpose to ensure reliability during earthquake events. Additionally, certain spaces would be reserved at each end of UHPC connection region as well, for the convenience of construction. Therefore, the length of UHPC connection ($L_{UH}$) could be 10$d$ or greater in real existing structures.
3.3 Thickness of UHPC layer in connection region

The thickness of UHPC layer used in the proposed novel connection is of great significance, which is required to achieve balance between structural safety and financial efficiency. Here the term ‘thickness’ indicates the width of UPHC layer enclosing the conventional concrete; for example, in the static experiment, the thickness are 140 mm and 180 mm at the top and base section of the UHPC connection region, respectively. A minimum thickness ensuring the seismic safety is desired to ensure the layered-UHPC section able to accommodate the seismic demands exerted by standard RC region, especially with respect to tensile behavior.

Note that the parameters obtained from axial tensile experiment shown in Fig. 7 could not be directly used for design in engineering practice, since the uncertainty of materials should be considered to provide results with sufficient reliability. According to SIA2052 (MCS-EPFL 2016), both the elastic and ultimate strength obtained from material tests ($f_{Ute}$ and $f_{Utu}$) should be modified by several partial factors before utilized for design, as shown in Eqs. (1) and (2). Upon the design strength ($f_{Uted}$ and $f_{Utud}$) are computed with these equations, they could be adopted to determine the required thickness of UHPC layer, which will be further illustrated in Sect. 4 with illustrative numerical examples.

$$f_{Utud} = \frac{\eta h U \cdot \eta k \cdot f_{Ut}}{\gamma_U} \quad (1)$$

$$f_{Uted} = \frac{\eta h U \cdot \eta k \cdot f_{Ute}}{\gamma_U} \quad (2)$$

where $\eta_k$ is the partial factor equal to 1.0 and 0.85 for global and local analysis, respectively; $\gamma_U$ equals 1.3 and 1.4 for reinforced and unreinforced structures, respectively; $\eta h U$ indicates the influence of component thickness on the strength of UHPC, and could be determined as in Fig. 8 according to SIA2052 (MCS-EPFL 2016):

3.4 Steel bars in grouting bed

Generally, the thickness of grouting bed is quite small in engineering practice, causing the fact that cracks and damage usually occur in this region prior to that in columns, as observed in previous quasi-static tests. While excessive cracks are developed in grouting beds in extreme scenarios, extensive tensile strains could be introduced into longitudinal steels...
which may lead to structural failure. Consequently, the performance of grouting bed should be guaranteed to ensure the proposed innovative system emulative as those cast-in-place piers, which is the objective for most PFS piers constructed with ABC techniques (Wang et al. 2021a, b).

In the current study, the strain of longitudinal steels in grouting bed are preliminary required remaining elastic, to avoid excessive damage in this region. This objective aims to achieve balance between fully utilizing the strength of steels and minimizing crack width in grouting bed, which could be modified during practice according to the design requirements. That is, the restriction on the steel strains could be relaxed at the expense of increasing the damage at grouting beds. While further investigations should be conducted in the future, in terms of both theoretical and experimental analysis, to provide more comprehensive insights and suggestions concerning this issue.

### 4 Seismic design procedure for PFS piers with layered-UHPC connection

This section proposes a detailed design procedure for the PFS piers with layered-UHPC connection. The general philosophy, including the capacity protection concept, classification of ductile and capacity-protected members, as well as general design targets is introduced first. Then, detailed design approaches for each component are presented, followed by a step-by-step flowchart.

#### 4.1 Design philosophy

To make the design procedure proposed in this paper more friendly to engineers, it is developed based on the capacity protection philosophy that is widely accepted and adopted in current seismic design specifications. With this philosophy, designers could determine where the damage concentrates according to the requirement in engineering practice and protect the most crucial components at the expense of increasing damage in secondary ones. More details of this significant design philosophy could be found in amounts of literatures (Priestley et al. 1996, China 2011, Franchin and Pinto 2014; Xia et al. 2022).

For the connection investigated in current study (Fig. 1), the occurrence of cracks in UHPC material would lead to potential failure of anchorage of steels in pier columns and pile caps. Therefore, the layered-UHPC connection regions are expected to perform without any cracks and remain undamaged during earthquake events; that is, they should be designed as capacity-protected components in the bridges with PFS piers. While on the other hand, the standard regions composed of conventional RC sections are designed as ductile components, in which the potential nonlinear performance and damage are supposed to occur.

To achieve these design purposes and avoid cracks in UHPC connection regions, the tensile strain demands of outmost UHPC fibers should be less than the corresponding capacity, even when the ultimate state of standard regions is reached. Additionally, the tensile strain of steel in grouting bed is limited as well, to prevent the occurrence of cracks with excessive width in this region. The detailed approaches and frameworks realizing this design philosophy will be described in the following sub-sections. Note that these design requirements are
proposed to overcome the drawbacks of the specimen during the tests, to achieve high-level
performance. Therefore, some of them may not be fully satisfied during the tests (e.g., the
occurrence of cracks in UHPC connection region).

4.2 Seismic design approach for different components

Note that current Chinese codes, e.g., Code for Seismic Design of Urban Bridges (referred to as Chinese Code hereafter) (CJJ 166, 2011), generally focus on conventional concrete bridges. While for unconventional ones, such as those with long spans (cable-stayed or suspension bridges), tall piers (over 40 m), and new material (UHPC herein), special investigations should be conducted based on the codes. Therefore, this section introduces the design approach for each of the components of the PFS pier with layered-UHPC in detail.

4.2.1 Standard region

The standard RC regions in the PFS piers are generally designed and required to perform similar to their CIP counterparts, to ensure the reliability during earthquake events (Ameli et al. 2015; Xu et al. 2019; Wang et al. 2021a, b). Thus, the design procedures presented in current codes for conventional CIP bridges could be utilized herein. When E1 level earthquakes (i.e., frequent earthquakes) in Chinese Code are considered, the standard regions should perform elastically and attain functionality without any repair; that is, the seismic demands of the regions are required less than the capacity of the initial yielding of RC sections.

While for the E2 level earthquakes (i.e., rare earthquakes), yielding is generally allowed in RC sections, since requiring elastic behavior under these strong motions is economic inefficient (Pang and Wang 2021; Zhong et al. 2022). Consequently, the displacement-based design procedure utilized in current codes is employed for standard regions, in which the displacement demand at pier top is required less than the corresponding capacity. As specified in Chinese Code, the ultimate displacement at the top of single column pier, Δ_u, could be computed by:

\[ \Delta_u = \frac{1}{3}H^2\varphi_y + (H - \frac{L_p}{2})\theta_u \]  

(3)

in which \( H \) means the pier height; \( \varphi_y \) is the effective yielding curvature of RC section; \( L_p \) and \( \theta_u \) denote the length of equivalent plastic hinge and ultimate rotation of plastic hinge, respectively, and could be expressed as:

\[ L_p = 0.08H + 0.022f_yd_{bl} \geq 0.044f_yd_{bl} \]  

(4)

\[ \theta_u = L_p(\varphi_u - \varphi_y)/K \]  

(5)

where \( f_y \) and \( d_{bl} \) are the yield strength and diameter of longitudinal steel bars, respectively; \( \varphi_u \) denotes the ultimate curvature of RC sections; \( K \) means the safety factor, which is suggested as 2.0 according to Chinese Code (CJJ 166, 2011).
Note that these equations are utilized to calculate the displacement capacity of standard regions in PFS pier columns herein; thus, the pier height, \( H \), should be replaced by the height of standard regions, \( L_{st} \), during seismic design procedures.

### 4.2.2 UHPC connection region

To ensure the UHPC connection regions performing as capacity-protected components without cracks, the tensile strain capacity of UHPC material should be greater than the corresponding demand caused by the bending moment of standard regions under both E1 and E2 level earthquakes. When subjected to E1 level earthquakes, the tensile strain demands of UHPC could be obtained by applying the moment demands \( (M_{UH,E1}) \) computed through elastic method (e.g., response spectral analysis) on fiber section models.

When yielding occurs in standard regions under E2 level earthquakes, the calculation of moment demands \( (M_{UH,E2}) \) of UHPC connection regions become more complex, which could be estimated using Eq. (6). Then the corresponding tensile strain demand could be obtained through exerting \( M_{UH,E2} \) on the fiber section model, which is required less than the tensile capacity.

\[
M_{UH,E2} = \varphi_1 \varphi_2 M_y
\]

In this Eq. (6), \( M_y \) denotes the effective yielding moment of standard regions, which can be obtained through M-\( \varphi \) analysis. Note that since the base of standard region is at the top of UHPC connection (see in Fig. 2), \( M_{UH,E2} \), i.e., the response at the base of UHPC connection, should be greater than \( M_y \). Considering the triangular distribution of moment from pier top to base, a factor of \( \varphi_1 = H/L_{st} \) should be incorporated, where \( H \) and \( L_{st} \) are the pier height and the length of standard region, respectively. Additionally, \( \varphi_2 \) is the over-strength factor that accounts for the over-strength of materials, which is 1.2 in current study as suggested by Chinese Code (CJJ 166.2011).

### 4.2.3 Grouting bed

As mentioned previously, cracks with excessive width should be prevented in grouting bed, to avoid introduction of large tensile strain in steel bars and great displacement at pier top. This aim is realized in this study by requiring steel bars in this region perform elastically even during rare earthquakes; i.e., the tensile strain demand \( \varepsilon_D \) of longitudinal steel remain less than the elastic limit \( \varepsilon_{se} (= f_y/E_s) \), where \( f_y \) and \( E_s \) are the yielding stress and elastic modulus, respectively. The elasticity of longitudinal steel bars could be achieved by extending the steels in both column and foundation across through the grouting bed, which significantly increases the steel ratio in this region; alternatively, this aim might be achieved by using more steel bars or bars with greater diameter across grouting bed. While \( \varepsilon_D \) could be obtained through applying \( M_{UH,E2} \) on the fiber section model as well, similar to computing tensile strain of UHPC.
4.2.4 Seismic design details

Besides the computation of seismic demand and requirement of capacity, another significant aspect of bridge seismic design is the construction details, which should be carefully conducted to ensure that the structures are able to perform as designed (Priestley et al. 1996). For example, sufficient transverse steels should be deployed in pier columns to guarantee the pre-determined ductile performance, as widely recognized in capacity protection design philosophy and utilized in current seismic design specifications. Since the standard region performs as CIP elements, the construction requirement of transverse steels in these PFS piers is suggested to be determined according to current codes. For instance, as specified in Chinese Code, the length of intensified region should exceed the greater value of (1) the section dimension in the input direction; and (2) the region where moment response overwhelms 80% of the maximum demands during earthquakes. More details could be found in (JTG/T 2231-01-2020).

4.3 Seismic design procedure

Based on the above statement, the flowchart of the seismic design procedure for PFS piers using the innovative layered-UHPC connection is presented in Fig. 9. In this procedure, the layered-UHPC connection region is designed as capacity-protected member, whose minimum height is required to provide reliable anchorage capacity. While the maximum tensile strain of steels in grouting bed is limited to avoid excessive cracks and structural displacement. Then the standard regions could be designed and checked in identical manner as cast-in-place columns.

One of the most significant advantages of this procedure is that it remains quite similar to that for CIP pier columns, and thus it is believed friendly to designers and convenient for promotion. Note that the limitation of longitudinal steel strain (e.g., demand $\varepsilon_{D}$ less than elastic limit $\varepsilon_{se}$ in current study as mentioned in Sect. 4.2.3) in grouting bed could be modified according to design requirements in practice. However, once this modification is conducted, the design parameters of UHPC connection region should be modified accordingly. For example, if the steels are allowed to perform nonlinearly, the corresponding anchorage length, as well as the height of layered-UHPC connection region $L_{UH}$, need to be improved.

The design steps of this procedure are summarized as following.

(1) According to the static and service loadings, determine the main properties of PFS piers, e.g., the height of pier column, dimensions of cross sections and deployment of reinforcement;

(2) Based on the capacity protection philosophy, define the design targets for different components, e.g., avoid cracks in UHPC and set limitation for steel strains in grouting beds;

(3) Specify the design parameters: (a) determine anchorage length and total height of connection region ($L_{UH}$) based on the diameter of steels; (b) determine the thickness of UHPC based on bending moment exerted by standard region; (c) determine the amount of steels in grouting bed based on the pre-set strain limitation;

(4) According to current seismic design codes, check the performance of standard region with the above parameters of layered-UHPC connection region.

Note that the parameters of UHPC connection regions and grouting beds mentioned in Step (3) are generally dominated by rare earthquakes, rather than the frequent ones. Addi-
tionally, when frequent motions are considered, initial yielding of RC sections should be determined using design strength of materials (concrete, steel and UHPC); while under rare excitations, the effective yielding moment \( M_y \) used in Eq. (6) should be computed based on the standard values of material strength.

5 Illustrative examples

This section first introduces an existing PFS bridge pier designed with the proposed procedure, where various pier heights are considered to provide a more comprehensive insight. Then the design results are presented and checked according to requirements and procedures presented in Sect. 4, validating the efficacy.
Response spectral analysis (RSA) method is usually recommended in current seismic design codes to accomplish capacity protection design, due to its simplicity in concept and high efficiency for computation. In this method, the lateral forces obtained by multiplying the mass and the corresponding spectral acceleration are applied on the structures; then the seismic demands can be estimated through this simplified static manner. More details about this significant method can found in various codes and previous studies.

While in current study, the prototype PFS piers are designed as ‘regular’ ones specified in Chinese Code, for which the RSA is expected to be suitable. Therefore, this simple procedure is employed to estimate the seismic demands for the following illustrative examples.

5.1 Prototype of the PFS pier with layered-UHPC connection

Figure 10 shows the existing prototype PFS pier considered in this study, where the column is integrated with the pile cap and beam cap utilizing the layered-UHPC connection regions mentioned previously. The superstructure, which is not presented in this figure, weighs 5036 kN and causes axial compression ratios around 9% at pier base, which is similar to that considered in the specimen (8.3%) for quasi-static tests. Three GJZ $350 \times 350 \times 84$ rubber bearings are employed between the top of pier cap and the base of girders. As plotted in Fig. 10, the standard region, with the length denoted as $L_{st}$, contains rectangular cross section composed of conventional concrete, which is 2.2 m and 1.5 m in transverse and longitudinal direction, respectively. The RC section in standard region is reinforced by totally 66 HRB400 longitudinal steel bars with diameter ($d$) equals 32 mm (Fig. 10 (b)), corresponding to a longitudinal steel ratio of 1.60%, which is a commonly adopted value in practice and comparable to that in the test specimen (2.0%).

Due to the overlap of longitudinal steel bars extended from pier column and pile cap, the anchorage length is suggested no less than 8.5$d$, since the diameter of these steels exceeds 25 mm. Because twin configuration is utilized for the longitudinal steels, the diameter $d$ considered herein is the equivalent value leading to identical area of two bars, which is about 45 mm. For conservative purpose, this anchorage length value is usually selected...
as 10$d$ in engineering practice. While considering 25 mm at each end of UHPC region for construction, $L_{UH}$ is finally determined as 500 mm ($=10\times45 \text{ mm}+2\times25 \text{ mm}$) in this existing prototype to ensure reliable anchorage of steels; and the thickness of UHPC layer in the cross section is preliminary determined as 350 mm.

The detailed information of materials used in this prototype, including steels, conventional concrete and UHPC, are presented in Tables 4 and 5.

To understand the efficiency of the design procedure more comprehensively, the pier height, i.e., $H$ in Fig. 10 (a), is considered ranging from 6 to 15 m in the following sections for parametric analysis. While the height of UHPC ($L_{UH}$) remains 500 mm, since it is determined to provide sufficient anchorage length.

### 5.2 Seismic design following the procedure

Since this considered prototype performs as a single cantilever column during earthquakes, its seismic behavior is supposed to be similar in two perpendicular directions, and thus the design process is only conducted in transverse direction to avoid data redundancy. Additionally, only the UHPC region connecting pier base and pile cap is focused on in this section, as the demands of the connection region between pier column and cap beam are generally insignificant for cantilever columns.

### 5.2.1 Design acceleration response spectra

Since the prototype bridge is constructed in western China (Urumqi, Xinjiang) with high seismic hazard level (Zhong et al. 2021; Chen et al. 2022), the target acceleration response spectra with 5% damping are developed as specified in Chinese Code. As presented in Fig. 11, the engineering site is with peak ground acceleration (PGA) values, achieving 0.124 and 0.405 g, respectively, for the frequent and rare earthquakes. While the fundamental
vibration periods of the investigated piers with various heights ($H = 6 \sim 15$ m) are plotted in this figure as well for illustration.

### 5.2.2 Moment-curvature analysis for standard and UHPC connection regions

To conduct the moment-curvature analysis for various sections, the constitutive relations of steel, conventional concrete and UHPC should be defined in prior. Figure 12 shows the strain-stress relationships for all the materials, corresponding to both design and standard strengths, which are utilized in seismic design for frequent and rare earthquakes, respectively.

Based on these constitutive relations, the results of moment-curvature analysis for standard regions in 10 m pier are presented in Fig. 13 for two earthquake levels as an illustration; in this figure, both actual and idealized M-$\phi$ curves are presented, and the critical values are denoted as well. The analysis is conducted using Xtract software (Chadwell and Imbsen 2004), during which the 2.2 m $\times$ 1.5 m cross section is meshed into 540 fibers representing different materials. All the results in this figure correspond to the axial force caused by static loadings, which is about 6000 kN in this case. Note that for seismic design of frequent and rare earthquake events, the moment values of initial and effective yield will be utilized as section capacities, respectively, as mentioned in Sect. 4.3.

### 5.2.3 Seismic response and capacity for frequent earthquakes

Figure 14 (a) shows the maximum bending moment of the standard regions for piers with various heights, as well as the corresponding initial yield moment (capacity). Note that the
moment capacity slightly increases with pier heights, due to the greater axial compression force caused by the increase of pier column. The results in this figure demonstrate that for all the pier models considered, the standard regions remain elastic with bending moment demands less than the initial yielding value, which meets the requirement proposed in previous section.

While for the layered-UHPC connection regions, the tensile strain demands of the outermost UHPC fibers for each pier column are presented in Fig. 14 (b), which are obtained by exerting the bending moment calculated from RSA on the fiber section models. This figure shows that the maximum value among all responses \((H=8 \text{ m})\) is less than 800 \(\mu\varepsilon\), accounting for about 40% of the crack value (2000 \(\mu\varepsilon\) as denoted in Table 5), satisfying the requirement proposed previously.

### 5.2.4 Seismic response and capacity for rare earthquake

Figure 15 (a) presents the bending moment demands for various pier models under rare earthquake situation, in which the effective yielding moment values are presented as well. From this figure, yielding is observed occurring for all the scenarios considered. According to Sect. 4, the displacement demands at the top of piers are calculated and plotted in Fig. 15 (b), together with the corresponding capacities computed using Eqs. (3) – (5). The results in Fig. 15 (b) show that the displacement capacities are always greater than the demands, indicating that the standard regions of designed PFS bridge piers could remain reliable under rare earthquakes.

While on the other hand, the seismic demands in this scenario \(M_{\text{UH E2}}\) exerted on the UHPC connection regions should be obtained using Eq. (6) for all the pier, due to the yielding mechanism of standard regions. When exerting these bending moment demands on fiber section models, the maximum tensile strains of UHPC are obtained and plotted in Fig. 16 (a). The results show that the tensile strain demands of UHPC fibers remain lower than the
capacity ($\varepsilon_t = 2000 \mu e$), except for the pier with height of 6 m. This phenomenon indicates that the UHPC region in this column is not fully capacity-protected, with potential occurrence of cracks. Although cracks were observed within UHPC region during the previous static tests, they are required to be fully avoided in current design, to improve the anchorage reliability and the overall seismic performance of the FPS piers.

Therefore, the thickness of UHPC layer in the 6 m pier should be modified accordingly. For example, when the thickness ($t_{UH}$) increases from 350 to 400 mm, this 6 m pier could yield tensile strain demands lower than 2000 $\mu e$, as presented in Fig. 16 (b). That is, UHPC connection region in this case performs as capacity-protected, and thus remain elastic and undamaged. This fact also demonstrates that the seismic performance of this type of piers is dominated by rare earthquakes rather than frequent ones.

Additionally, the statement of grouting beds between standard and UHPC connection regions is considered when subjected to rare earthquakes as well. In current study, the tensile strains of longitudinal steel bars in this region are required less than yielding value ($2000 \mu e$). From the tensile strain results presented in Fig. 17, the steels are observed remain elastic for piers with heights between 8 and 15 m, indicating that the damage in grouting bed is limited and would not lead to excessive demands to the whole structures in these scenarios. While for 6 and 7 m pier, the demands exceed yielding value slightly. Depending on the decision of owners and designers, two alternatives could be adopted for this situation: increasing the number of longitudinal steels and relaxing the limitation of steels. Once the latter one is selected, the parameters as $L_{UH}$ should be modified accordingly to ensure reliable seismic performance.
6 Conclusions

This paper proposes an innovative layered-UHPC connection for pre-fabricated segmental (PFS) bridges constructed with ABC techniques, which possesses higher economic efficiency than conventional ones by using less UHPC material. Based on the quasi-static experiments, seismic design procedure for piers with this connection is proposed to overcome the drawbacks of specimen observed during tests and achieve high-level seismic performance. According to the results, the following conclusions could be obtained:

1. Compared with previous connections with UHPC filling an expanded cage, the one proposed in current study is more economic efficient, which only employs UHPC layer at pier base. Additionally, the construction quality and damage state of UHPC material could be easily inspected in this connection since it is casted at the outer surface. The seismic performance of conventional concrete could be improved as well due to confinement provided by UHPC layer.

2. Quasi-static experiments for PFS pier column with this connection show that cracks and damage occur in standard RC concrete region prior to layered-UHPC connection region, indicating the shifting of nonlinearity. While greater lateral loadings are exerted, relative opening are observed at the interface between the layered-UHPC connection region and foundation (i.e., grouting bed). Note that excessive width in this region can lead to extensive tensile strain in longitudinal bars and large displacement at pier top, which is thus suggested to be controlled in the proposed design procedure.

3. The test specimen is observed with comparative displacement ductility as cast-in-place ones and full hysteretic loops without obvious pitching effect. These phenomena indicate that the PFS pier with the innovative layered-UHPC connection possesses emulative performance with cast-in-place counterparts and is potential alternative for application in seismic regions.

4. Based on the experiments, the seismic design procedure for PFS bridges using this type of connection is presented using the capacity protection philosophy widely adopted in

Fig. 17 Demands of tensile strain of longitudinal steel in grouting bed under E2 earthquake
current design codes. UHPC connection regions are designed as capacity protected components to provide reliable anchorage between longitudinal steel bars in the pre-fabricated component and the foundation. While the standard regions are utilized as ductile ones and expected experiencing potential nonlinearity and damage in strong earthquakes. The width of potential cracks developed in grouting bed is constrained through setting limitations for the tensile strain demands of longitudinal steels, to avoid structural failure caused by excessive damage of this region.

5 The targets in the proposed design procedure are specified to overcome the drawbacks of the test specimens and further improve seismic performance of the FPS piers. For instance, the cracks observed within UHPC connection region in the experiments might deteriorate the anchorage reliability of longitudinal steel bars, and thus are required to be fully avoided during design.

6 From numerical illustrative examples considering PFS bridges with various pier heights, the proposed procedure is shown able to effectively guide the seismic design of PFS piers with the layered-UHPC connection. Note that in addition to ensure the emulative performance of PFS piers using the layered-UHPC connection as their CIP counterparts, this proposed seismic design procedure could be implemented similar to that specified in current seismic design codes. These characters are believed significantly friendly to designers for application in engineering practice, which would help the promotions of this innovative connection.

The proposed design procedure for layered-UHPC connection is only preliminary validated through parametric numerical analysis. Current work is on the way preparing further experiments, where the specimens will utilize the bridges in the illustrative examples as prototypes and follow the design procedure proposed in this study. With the help of the results of the future tests, the efficiency of this design procedure will be further validated.

Acknowledgements The authors gratefully acknowledge the support by the National Natural Science Foundation of China (No. 51908348) and Shanghai Rising-Star Program (No. 21QB1405000).

Author contribution Xu CHEN: Conceptualization; Data curation; Formal analysis; Funding acquisition; Investigation; Methodology; Writing. Biao MA: Resources; Writing-review & editing. Ruilong WANG: Conceptualization; Data curation; Funding acquisition; Writing-review & editing.

Funding National Natural Science Foundation of China (No. 51908348) & Shanghai Rising-Star Program (No. 21QB1405000).

Data Availability All data, models, or code generated or used during the study are available from the corresponding author by request.

Declarations

Conflict of interest The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

References

Ameli MJ, Brown DN, Parks JE, Pantelides CP (2016) “Seismic Column-to-Footing Connections Using Grouted Splice Sleeves”. Aci Struct J 113(5):1021–1030
Ameli MJ, Parks JE, Brown DN, Pantelides CP (2015) “Seismic evaluation of grouted splice sleeve connections for reinforced precast concrete column-to-cap beam joints in accelerated bridge construction.” Pci Journal: 80–103

Billington SL, Breen JE (2000) Improving Standard Bridges with Attention to Cast-in-Place Substructure. J Bridge Engineering 5(4):344–351

Caltrans (2019) Caltrans seismic design criteria version 2.0. California Dept. of Transportation, Sacramento, CA

Chadwell CB, Imbsen RA (2004) XTRACT: A tool for axial force-ultimate curvature interactions Structures: Building on the past, securing the future. 2004:1–9

Chen X, Li C. (2020) Seismic assessment of earthquake-resilient tall pier bridges using rocking foundation retrofitted with various energy dissipation devices. Structural Control and Health Monitoring. 27(11):e2625.

Chen X, Guan Z, Li J, Spencer BF Jr (2018) Shake table tests of tall-pier bridges to evaluate seismic performance. J Bridge Engineering 23(9):04018058

Chen X, Xiang N, Li J et al (2020) Influence of near-fault pulse-like motion characteristics on seismic performance of tall pier bridges with fragility analysis. J Earthquake Eng 26(4):2001–2022

Chen X, Xiang N, Guan Z, Li J (2022) Seismic vulnerability assessment of tall pier bridges under mainshock-aftershock-like earthquake sequences using vector-valued intensity measure. Eng Struct 253:113732

Chen X, Xiong J (2022) Seismic resilient design with base isolation device using friction pendulum bearing and viscous damper. Soil Dyn Earthq Eng 153:107073

CJJ 166–2011 (2011) Code for seismic design of urban bridges. Ministry of Housing and Urban-Rural Development of the People’s Republic of China, Beijing

Fan W, Zhong Z, Huang X, Sun W, Mao W. (2022) Multi-platform simulation of reinforced concrete structures under impact loading. Engineering Structures. 266:114523.

Feng J, Gao X, Li J, Dong H, Yao W, Wang X, Sun W (2019) Influence of fiber mixture on impact response of ultra-high-performance hybrid fiber reinforced cementitious composite. Compos Part B-Engineering 163:487–496

Franchin P, Pinto PE (2014) Performance-based seismic design of integral abutment bridges. Bull Earthq Eng 12(2):939–960

Haber ZB, Mackie KR, Al-Jelawy HM (2017) Testing and Analysis of Precast Columns with Grouted Sleeve Connections and Shifted Plastic Hinging. J Bridge Engineering 22(10):14

Haber ZB, Saidi MS, Sanders DH (2014) “Seismic Performance of Precast Columns with Mechanically Spliced Column-Footing Connections”. Act Struct J 111(3):639–650

Haraldsson OS, Janes TM, Eberhard MO, Stanton JF (2013) Seismic Resistance of Socket Connection between Footing and Precast Column. J Bridge Engineering 18(9):910–919

Huang H, Wang R, Gao X (2019) “Improvement effect of fiber alignment on resistance to elevated temperature of ultra-high performance concrete.” Composites Part B-Engineering 177

Hung C-C, Lee H-S, Chan SN (2019) Tension-stiffening effect in steel-reinforced UHPC composites: Constitutive model and effects of steel fibers, loading patterns, and rebar sizes. Compos Part B-Engineering 158:269–278

Ichikawa S, Matsuzaki H, Moustafa A, ElGawady MA, Kawashima K (2016) “Seismic-Resistant Bridge Columns with Ultrahigh-Performance Concrete Segments.” Journal of Bridge Engineering 21(9)

JTG/T 2231-01-2020 (2020) Guidelines for seismic design of highway bridges. Ministry of Transport of the People’s Republic of China, Beijing

Kurama YC, Sritharan S, Fleischman RB, Restrepo JI, Henry RS, Cleland NM, Ghosh SK, Bonelli P (2018) “Seismic-Resistant Precast Concrete Structures: State of the Art.” Journal of Structural Engineering 144(4)

Li C, Liu Y (2003) Optimum multiple tuned mass dampers for structures under the ground acceleration based on the uniform distribution of system parameters. Earthquake Eng Struct Dynam 32(5):671–690

Markogiannaki O, Orologopoulos N, Tegos I (2017) Experimental and analytical study on hollow precast piers with unbonded conventional reinforcement to control seismic and in-service response of bridges. In: 6th ECCOMAS thematic conference on computational methods in structural dynamics and earthquake engineering, pp 282–296

Marshall C, Cantrell J, Marsh M, Ebrahimpour A (2020) A Precast Pier System for ABC in Seismic Regions, Structures Congress 2020, 183–192

MCS-EPFL (2016)

Meng Q, Wu C, Li J, Liu Z, Wu P, Yang Y, Wang Z (2020) “Steel/basalt rebar reinforced Ultra-High Performance Concrete components against methane-air explosion loads.” Composites Part B-Engineering 198
Mohebbi A, Saiidi MS, Itani AM (2017) Seismic design of precast piers with pocket connections, CFRP tendons and ECC/UHPC columns. Int J Bridge Eng Spec Issue 2017:99–123
Moustafa A, ElGawady MA (2018) “Shaking Table Testing of Segmental Hollow-Core FRP-Concrete-Steel Bridge Columns.” Journal of Bridge Engineering 23(5)
Ou YC, Wang PH, Tsai MS, Chang KC, Lee GC (2010) Large-Scale Experimental Study of Precast Segmental Unbonded Posttensioned Concrete Bridge Columns for Seismic Regions. J Struct Engineering-Asce 136(3):255–264
Pang Y, Wang X (2021) Cloud-IDA-MSA Conversion of Fragility Curves for Efficient and High-Fidelity Resilience Assessment. J Struct Eng 147(5):04021049
Perotti F, Domaneschi M, De Grandis S (2013) The numerical computation of seismic fragility of base-isolated Nuclear Power Plants buildings. Nucl Eng Des 262:189–200
Priestley MN, Seible F, Calvi GM, Calvi GM (1996) Seismic design and retrofit of bridges. John Wiley & Sons
Siwowski T, Rajchel M (2019) “Structural performance of a hybrid FRP composite - lightweight concrete bridge girder.” Composites Part B-Engineering 174
Steuck KP, Eberhard MO, Stanton JF (2009) “Anchorage of Large-Diameter Reinforcing Bars in Ducts”. Aci Struct J 106(4):506–513
Tazarv M, Saiidi MS (2015) UHPC-filled duct connections for accelerated bridge construction of RC columns in high seismic zones. Eng Struct 99:413–422
Tekie PB, Ellingwood BR (2003) Seismic fragility assessment of concrete gravity dams. Earthquake Eng Struct Dynam 32(14):2221–2240
Wang J, Liu J, Wang Z, Liu T, Liu J, Zhang J (2021a) Cost-Effective UHPC for accelerated bridge construction: material properties, structural elements, and structural applications. J Bridge Engineering 26(2):04020117
Wang JQ, Wang Z, Tang YC, Liu TX, Zhang J (2018) Cyclic loading test of self-centering precast segmental unbonded posttensioned UHPFRC bridge columns. Bull Earthq Eng 16(11):5227–5255
Wang R, Ma B, Chen X (2021b) Seismic performance of pre-fabricated segmental bridge piers with grouted splice sleeve connections. Engineering Structures 229
Wang Z, Wang J, Li J, Han F, Zhang J (2019) Large-scale quasi-static testing of precast bridge column with pocket connections using noncontact lap-spliced bars and UHPC grout. Bull Earthq Eng 17(9):5021–5044
Wang Z, Wang J, Zhao G, Zhang J (2020) Numerical study on seismic behavior of precast bridge columns with large-diameter bars and UHPC grout considering the bar-slip effect. Bull Earthq Eng 18(10):4963–4984
White S, Palermo A (2016) Quasi-Static Testing of Posttensioned Nonemulative Column-Footing Connections for Bridge Piers. Journal of Bridge Engineering 21(6)
Xia X, Wu S, Sun S, Du Q, Long F (2022) Lateral hysteretic behavior of a novel metal rubber bridge bearing. Eng Struct 256:114051
Xu W, Ma B, Duan X, Li J (2021) Experimental investigation of seismic behavior of UHPC connection between precast columns and footings in bridges. Eng Struct 239:112344
Xu W, Ma B, Huang H, Su J, Li J, Wang R (2019) The Seismic Performance of Precast Bridge Piers with Grouted Sleeves. Engineering Mechanics. (in Chinese)
Zhong J, Yang T, Pang Y, Yuan W. (2021) A Novel Structure-Pulse Coupled Model for Quantifying the Column Ductility Demand under Pulse-Like GMs. Journal of Earthquake Engineering: 1–19. https://doi.org/10.1080/13632469.2021.1989348
Zhong J, Ni M, Hu H et al (2022) Uncoupled multivariate power models for estimating performance-based seismic damage states of column curvature ductility. Structures 36:752–764

Publisher’s Note Springer Nature remains neutral with regard to jurisdictional claims in published maps and institutional affiliations.