Comparison Study of Time-varying Seismic Fragility of Precast Segmental and Cast-in-place Bridge Columns in High-speed Railway Bridges

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Comparison study of time-varying seismic fragility of precast segmental and cast-in-place bridge columns in high-speed railway bridges

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Abstract: Anxiety about the long-term seismic performance of precast segmental bridge columns (PSBCs) limits their wide application in harsh environments and high intensities. However, railway route selection inevitably occurs in offshore areas, and it is urgent to study the seismic performance of the PSBC considering environmental erosion and seismic effects. In order to study the application of precast segmental bridge columns (PSBCs) in offshore high intensity areas, this paper comprehensively compares the time-varying seismic fragility of precast segmental and cast-in-place bridge columns (CPBCs) in high-speed railway bridges. Based on an offshore high-speed railway column, finite element models of representative PSBCs and CPBCs are established and verified by experiments. Then, the verified finite models are utilized to implement the time-varying fragility analysis by considering chloride ion erosion under four damage states. The main conclusions are as follows: the exceeding probabilities of PSBC and CPBC are close in intact bridges under different damage states and seismic intensities. With the prolongation of bridge service time, the exceeding probabilities of PSBC increase more rapidly than those of CPBC due to the discontinuity of PSBC segments, which leads to the acceleration of corrosion of steel bars. When the columns reach the design working life, the exceeding probability and PGA median of PSBC are higher than those of CPBC columns under four damage states. Taking the medium damage state as an example, the maximum exceeding probability of PSBC and CPBC was 97.2% and 89.1% at the medium damage state, and the PGA median of PSBC was 24% lower than that of CPBC at 100 years. This work provides a theoretical foundation for the better application of PSBC in offshore high intensity areas.

Keywords: seismic fragility, time-varying, precast segmental bridge column, damage state.

1 Introduction

In recent decades, researchers have shown an increased interest in modular construction of precast structures due to their lower environmental pollution, fast construction speed and reliable component quality. In the field of the bridge engineering, precast segmental construction has been widely used in superstructures and substructures (Kashani et al., 2019). However, the application of precast components in the substructure occurred later than that in the superstructure. PSBC was first applied to JFK Causeway in Texas in the 1970s, and the bridge community has realized the obvious advantages of prefabricated columns. Compared with PSBC, the use of precast bridge columns can improve the

engineering quality, and reduce the number of on-site personnel and the environmental impact(Sideris et al., 2014; Zhang Y, 2019). However, the anxieties of the long-term seismic performance of PSBC limit its application in high -intensity and offshore environmental areas(Hung et al., 2017). Therefore, the time-varying seismic vulnerability of the CPBC should be studied and compared with that of the PSBC in detail.

Seismic vulnerability analysis is widely used for the seismic performance of investigated bridges and is a part of the seismic risk analysis framework of the Pacific Earthquake Engineering Research Center (PEER)(Rao et al., 2016). It quantifies the exceeding probability of structural or component damage in earthquakes. In the offshore environment, corrosion deterioration (especially caused by chloride ions) is considered to be the key factor affecting the quantification of bridge vulnerability. The probability of bridge damage increases with the extension of service time(Liang et al., 2020, 2021). Meanwhile, many coastal areas are high -intensity zones. The seismic performance of cast-in-place bridges is expected to be significantly lower than their design performance under the combined action of earthquakes and chloride ion corrosion(Ou & Nguyen, 2016; Yuan et al., 2017). Numerous studies of the time-varying vulnerability of cast-in-place bridges have been reported considering the accelerated degradation of materials in coastal areas and earthquake impacts in fault zones. Biondini proposed a probabilistic approach to predict the lifetime seismic performance of concrete bridges exposed to aggressive environments (Biondini et al., 2014). Li established a framework for evaluating the time-varying seismic performance of uncertain degraded CPBC by considering material parameters, corrosion initiation time and uncertainty of ground motion (Li et al., 2018). Cheng proposed a seismic vulnerability analysis method for CPBC corrosion degradation based on a time-varying estimation of the displacement ratio as the bearing capacity index(Cheng et al., 2019).Based on the Duracrete model and previous experimental results, Li determined the probability distribution types and statistical characteristics of various environmental and corrosion parameters, and a framework for analyzing the seismic vulnerability of CPBC subjected to chloride ion erosion was proposed (Li et al., 2020).

PSBC has a different structural composition than CPBC, which includes joints, energy dissipation (ED) bars and unbonded prestressed tendon (PT) strands(Li et al., 2017). Its working performance is very different from CPBC. Due to the discontinuity between segments, the seismic performance of PSBC is recognized to be weaker than that of CPBC(Jia et al., 2020). Scholars at home and abroad have performed many studies on the working performance of PSBCs, such as cyclic load tests(Li et al., 2017), shaking table tests(Li et al., 2019), and impact tests(Do et al., 2019) and so on. The PSBC is more likely to show quasi -elastic behavior in medium and small earthquakes(Tong et al., 2021). Through the experimental study (Zhanghua et al., 2021), an appropriate slenderness ratio and longitudinal bars can significantly improve the deformation capacity of PSBC under earthquakes. These studies enable PSBC to be widely used in low intensity areas. Some scholars have studied the seismic vulnerability of PSBCs in high -intensity areas. Lee and Billington

analyzed the seismic vulnerability of a bridge system supported by PSBC in the PBEE framework and evaluated its economic loss(Lee & Billington, 2011). Ehsan Ahmadi used incremental dynamic analysis (IDA) to generate a vulnerability curve for studying the vulnerability of PSBC, and the results indicated that low columns are more prone to earthquake collapse than slender columns(Ahmadi & Kashani, 2021). However, many high-intensity areas are in coastal areas, and when bridges are built in these areas, the structures are vulnerable to chloride ion corrosion in seawater. Therefore, the coupling effect of material performance degradation and earthquakes should be considered. Scholars at home and abroad rarely study the time-varying seismic vulnerability of PSBCs. From the above review, this paper studies the time-varying seismic vulnerability of PSBC on the basis of predecessors and compares it with that of CPBC.

This paper makes a comparative study on the long-term seismic vulnerability of PSBCs and CPBCs. First, the bridge model is established according to an example bridge in an offshore high-intensity area in Section 2, and then the established model is tested and verified to verify the effectiveness of the modeling method in Section 3. In Section 4, the displacement ductility ratio is selected as the time-varying seismic fragility index, the ground motion meeting the local site conditions is selected from PEER for probabilistic seismic demand analysis, the fragility curve is drawn, and the median PGA is obtained. The main conclusions are drawn in Section 5.

2 Establishment of Finite Element Model

2.1 Description of the finite element model of PSBC and CPBC

Fig.1 Finite element model of PSBC and CPBC

Without loss of generality, a typical five-span continuous rigid frame high-speed railway bridge is adopted in the present study as the reference model. The site conditions are class 2, the seismic fortification intensity is 8 degrees, and the designed service life is 100 years. The concrete grade is C45, and the protective layer thickness is 45 mm. The stirrups in PSBC and CPBC are 10mm HPB300, and the longitudinal bars are 32mm HRB500, with reinforcement ratios of 0.56%

and 0.70%, respectively. In CPBC, 28mm HRB500 steel bar is used for ED bars and 1860 steel strand with a diameter of 15.2mm is used for unbonded PT strands, ED bars and unbonded PT strands run through all sections.

Taking the column of the high-speed railway bridge as an example, the finite element model of PSBC and CPBC is established as shown in Fig.1. The models are simulated using fiber-based nonlinear elements in OpenSees which can effectively simulate the hysterical behavior of columns and dynamic response under earthquake excitation(Pan et al., 2017). Each segment of PSBC and the body of CPBC are all simulated by Displacement-Based Beam-Column element, the bottom node is consolidated, and the top node is released. Different from CPBC, in the joint section of PSBC, the ED bar fiber is decorated. The joint sections between the column segments are simulated by ZeroLengthSection element in OpenSees. The existing PSBC test results showed that the joints will open and close under the action of horizontal displacement, and there is basically no shear dislocation between the segments (Chou & Chen, 2006; Ou et al., 2007, 2010, 2010) . Corotational Truss element is used to simulate unbonded PT strands, the division of Corotational Truss element corresponds to Displacement-based Beam-Column element of the column body.

2.2 Time-varying material and mechanical properties

Concrete02 is selected to model the constitutive behavior of the concrete, which can better estimate the residual deformation of the concrete(Kent & Park, 1990). The reinforcing bars is modelled using the steel02 in OpenSees, which is capable of considering isotropic strain hardening and the Bauschinger effect(Spacone & Filippou, 1991). The parameter values in the steel model for longitudinal bars are: $f_y=500\text{MPa}$, $f_u=630\text{MPa}$, $E_s=1.93\times 10^5\text{MPa}$, $E_{sh}=6.0\times 10^3\text{MPa}$, $\varepsilon_{sh}=0.01$, $\varepsilon_{su}=0.15$, the corresponding values for stirrups are $f_y=300\text{MPa}$, $f_u=420\text{MPa}$, $E_s=1.93\times 10^5\text{MPa}$, $E_{sh}=6.0\times 10^3\text{MPa}$, $\varepsilon_{sh}=0.01$, $\varepsilon_{su}=0.15$.

In offshore environments, bridge columns are susceptible to chloride ion corrosion in seawater. The diffusion mechanism of chloride ions in RC structures with different exposure conditions is different. The initial corrosion time of reinforcement considering chloride ion corrosion is:

$$t_i = \frac{c}{4D_c} \left[\text{erf}^{-1} \left(1 - \frac{M_{cr}}{M_s} \right) \right]^2 \times 10^{-6} + 0.2t_1 \quad (1)$$

where t_i is the beginning corrosion time of the reinforcement (year); c is the thickness of the concrete cover (mm); D_c is the diffusivity coefficient of chloride ions (mm^2/year); $\text{erf}(\cdot)$ represents the Gauss error function; M_{cr} is the critical chloride concentration of reinforcement corrosion (kg/m^3); M_s is the chloride concentration at the concrete surface (kg/m^3); t_1 is the accumulation time for chloride ions to reach a stable value on the concrete surface.

The rust expansion cracking time of the concrete protective layer includes the initial corrosion time of the reinforcement and the migration and diffusion time process of the corrosion products. It can be expressed as follows(Liang

et al., 2019):

$$t_{cr} = t_i + t_c \quad (2)$$

where t_{cr} is the rust expansion cracking time of the concrete cover, t_c is the time from steel bar corrosion to concrete cracking.

After the concrete cover cracks, the corrosion rate of reinforcement further accelerates with the deepening of chloride ion erosion, the concrete cover cracks and the corrosion rate of reinforcement accelerates, the time-variant diameter model of corroded reinforcement considering the cracking of concrete cover is as follows(Stewart & Rosowsky, 1998):

$$d(t) = \begin{cases} d_0 & t \leq t_i \\ d_0 - 2\lambda_1(t - t_i) & t_i < t \leq t_{cr} \\ d_0 - 2\lambda_1(t_{cr} - t_i) - 2\lambda_2(t - t_{cr}) & t > t_{cr} \end{cases} \quad (3)$$

where, $d(t)$ is the time-variant diameter; d_0 is the initial diameter of the reinforcement (mm); λ_1 is the corrosion rate before the cracking of the concrete cover (mm/year); λ_2 is the corrosion rate after the cracking of the concrete cover (mm/year).

The yield strength and ultimate strength of the reinforcement degenerated with the corrosion, and the degradation formula can be expressed as (Du et al., 2005):

$$f_{y,corr} = (1 - \beta_y Q_{corr}) f_y \quad (4)$$

$$f_{u,corr} = (1 - \beta_u Q_{corr}) f_u \quad (5)$$

$$Q_{corr} = \left(\frac{D(t)}{d_0} \right)^2 \quad (6)$$

where $f_{y,corr}$ and $f_{u,corr}$ are the yield strength (MPa) and the ultimate strength (MPa) of the corroded reinforcement; β_y and β_u are the reduction coefficients of the yield strength and the ultimate strength, which are taken as 0.0049 and 0.0065 respectively considering the field site of the investigated bridge; f_y and f_u are the yield strength (MPa) and ultimate strength (MPa) of the reinforcement before corrosion; Q_{corr} is the corrosion rate of the reinforcement.

Due to the deep location of the column PT strands in the engineering example, the equivalent concrete protective layer thickness is 181.72mm, and the corrosion time of the unbonded PT strands is 207.66 years, so the chloride ion corrosion effect of PT strands can be ignored.

Table1 Parameters related to reinforcement corrosion

Parameter	Stirrup	The first row of reinforcement	The second row of reinforcement	The first row ED bars	The second row ED bars
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	CPBC	PSBC	CPBC	PSBC	CPBC	PSBC	CPBC	PSBC	CPBC	PSBC
$c(\text{mm})$	45	45	55	55	87	67	-	26.79	-	40.77
$D_c(\text{mm}^2/\text{year})$	224.06	224.06	224.6	224.06	224.06	224.06	-	224.06	-	224.06
$M_{cr}(\text{kg}/\text{m}^3)$	2.1	2.1	2.1	2.1	2.1	2.1	-	2.1	-	2.1
$M_s(\text{kg}/\text{m}^3)$	3.618	3.618	3.618	3.618	3.618	3.618	-	3.618	-	3.618
$t_i(\text{year})$	15.74	15.74	21.93	21.93	50.07	30.99	-	8.37	-	14.22
$t_{cr}(\text{year})$	24.13	24.13	30.33	30.33	58.46	39.39	-	16.76	-	22.61
$\lambda_1(\text{mm}/\text{year})$	0.0121	0.0121	0.0099	0.0099	0.0063	0.0082	-	0.0204	-	0.0134
$\lambda_2(\text{mm}/\text{year})$	0.0304	0.0304	0.0248	0.0248	0.0157	0.0204	-	0.0509	-	0.0335

The relevant parameters of reinforcement corrosion under chloride ion corrosion are shown in Table 1, according to the site conditions of the investigated bridge. Table 1 indicates that the initial corrosion time of bars in segment in Row 1 of CPBC and PSBC is later than that of stirrup and earlier than that of bars in segment in Row 2, the corrosion rate is slower than that of stirrup and faster than that of bars in segment in Row 2. The thickness of the concrete cover after equivalent cement mortar between PSBC joints is much less than that of the segment. The corrosion time of reinforcement in PSBC is earlier and faster than that in CPBC, and the corrosion of bars in the segment and ED bars in the through joint shall be considered. The joints of PSBC commonly adapt epoxy adhesive joints whose durability factor is identical to that of direct wet joints, so the chloride ion erosion effect at the joints should be considered in the form of direct wet joints in this paper. The farther the steel bar is from the concrete surface, the later the corrosion time, and the smaller the corrosion rate before and after cracking.

Fig.2 Analysis of time-varying steel character

In the whole life cycle, the diameter, yield strength and ultimate strength of the reinforcement decrease with the service time extension. The stirrup is the outermost reinforcement, and the stirrup damage is stronger than that of the longitudinal bars and ED bars. Due to the same concrete cover of PSBC and CPBC, the stirrup damage degrees of PSBC and CPBC are the same. Compared with the second row of longitudinal bars of CPBC, the first row of longitudinal bars of CPBC is easily eroded by chloride ions in seawater, and the mechanical performance of the material decreases the most.

PSBC has joints, and the ED bars pass through the joints. The ED bars in PSBC contact sea water first and are corroded by chloride ions in sea water, especially the first row of ED bars. The mechanical properties of the material decline the fastest, as shown in Fig. 2. After 100 years of service, the reinforcement diameter, yield strength and ultimate strength of CPBC are reduced by 11%, 10%, and 14%, respectively; the reinforcement diameter, yield strength and ultimate strength of PSBC are reduced by 32%, 26%, and 35%, respectively. The necessity of considering chloride ion corrosion is verified.

3 The Verification of Finite Element Model

Fig.3 The verification of PSBC model

The previous experimental results are used to verify the accuracy of the current modeling method in Section 2. Bu (Bu et al., 2015) compared and analyzed four reduced scale PSBC specimens with different reinforcement types and one CPBC specimen through experience tests. This paper selects the experimental results of the PSBC specimen for model verification. The experiment PSBC selected is shown in Fig.3 (a). The piles, column body and beam are cast with C40 concrete. The perforated unbonded PT strand ratio was 0.31%, the initial tension was 252 kN, and the ED bar ratio was 0.71%. The stirrup spacing of the S1 segment is 50 mm, the stirrup ratio is 0.96%, and the stirrup spacing and ratio of the others are 80mm and 0.6%, respectively. The FE model of the selected column was established by using the identical modeling method, and the corresponding model is shown in Fig. 3(b). In the experiment, the displacement-controlled method was used for the loading test process, and the controlled draft levels included 0.1%, 0.2%, 0.3%, 0.5%, 0.75%, 1%, 1.5%, 2.0%, 2.5%, 3.0%, 3.5%, 4.0%, 4.5%, 5.0%, 6.0% and 7.0% with two cycles for each drift level.

Fig.4 Comparison between the FE model and experimental results

To verify the accuracy of the established model, comparisons of the load-displacement hysteretic curve, skeleton curve, vertical displacement of the column bottom joint and stress variation of the PT strands from the simulated and experimental results are shown in Fig.4. The results from the FE model are in good agreement with the experimental results in the reverse loading process. Some trivial discrepancies exist in the forward loading process due to the influence of inevitable construction and loading errors, which cause an asymmetrical shape in the load-displacement curve. Comparing Fig. 3 (a) and (b), the maximum bearing capacity from the FE model and test in the reverse process is 68 kN and 63 kN, respectively, with an error of approximately 8%. Fig.3(c) shows that the maximum opening displacements of the column bottom are 15mm and 13mm, and the maximum displacements when the joint is closed are both 5 5mmmm. Fig. 3(d) shows that the maximum stress values of the prestressed reinforcement under the positive and negative horizontal displacement angles of the finite element model and the test model are 1495MPa, 1484MPa, 1412MPa and 1550MPa, respectively. These comparison values indicate that the modeling method used is reasonable and that the models established in this paper can be used in the following seismic analysis with reliable calculation.

4 Time-varying seismic fragility analysis results

Seismic fragility is usually defined as a probability function that structure or component exceeds its a certain ultimate condition under given seismic intensity:

$$P_f[D \geq C | I_M] = \Phi\left[\frac{\ln(\mu_D / \mu_C)}{\sqrt{\beta_D^2 + \beta_C^2}}\right] \quad (7)$$

where, D is the seismic demand; C is the bearing capacity; I_M is the seismic intensity parameter; μ_C and μ_D are the mean values of the bearing capacity and structure seismic demand respectively, β_C and β_D are the logarithmic standard deviations of the bearing capacity and structure seismic demand respectively. The PGA is selected as the seismic intensity parameter, denoted as I_M , $\sqrt{(\beta_D^2 + \beta_C^2)}$ is 0.5; $\Phi(\cdot)$ is the standard normal accumulation distribution function

A linear correlation between seismic demand (D) and seismic intensity parameter (I_M) on logarithmic scale is usually used, and then seismic demand can be expressed as a function of seismic intensity parameter:

$$\ln(\mu_D) = \ln(a) + b \cdot \ln(I_M) \quad (8)$$

where a and b are the coefficients obtained from the structural response simulations under earthquake lading.

Therefore, the quantitative formula of conditional failure probability (exceeding probability) can be obtained as:

$$P_f[D \geq C | I_M] = \Phi\left[\frac{\ln(a \cdot (I_M)^b / \mu_C)}{0.5}\right] \quad (9)$$

4.1 Ground motion selection

Table 2 The selected ground motion

Station	Earthquake events	Time	Site	PGA (g)	Magnitude
RSN-6	Imperial	1940	El Centro Array #9	0.28	6.95
RSN-126	Gazli	1976	Karakyr	0.70	6.8
RSN-737	Loma	1989	Agnews State Hospital	0.17	6.93
RSN1100	Kobe	1995	Abeno	0.22	6.9
RSN1147	Kocaeli	1999	Ambarli	0.25	7.51
RSN1183	Chi-Chi	1999	CHY008	0.12	7.62
RSN4855	Chuetsu-oki	2007	Sanjo	0.11	6.8
RSN5672	Iwate	2008	MYG013	0.32	6.9
RSN5823	El Mayor-Cucapah	2010	Chihuahua	0.25	7.2
RSN6912	Darfield	2010	Hulverstone Drive Pumping Station	0.14	7

Ten earthquakes that meet the local site conditions are selected from the strong earthquake database of the PEER center, as shown in Table 2. Based on these actual ground motion records and Equation (10), 10 groups of 150 ground motions are randomly generated in the range of 0.01g-1.0g.

$$a_g^{(i)}(t) = k_i a(t) \quad (10)$$

where $a_g^{(i)}(t)$ is the ground motion record after the first amplitude modulation; $a(t)$ is the original actual ground motion record, and k_i is the amplitude modulation coefficient.

4.2 Damage index

Seismic damage can be divided into four types: minor damage, medium damage, serious damage and complete damage. The corresponding damage indexes of the four damage stages are the displacement ductility ratio of the steel bar for the first yield time (μ_{cyl}), the displacement-ductility ratio of the section equivalent yield (μ_{cy}), the displacement ductility ratio when the protecting layer strain of the bridge column is 0.004, and the maximum displacement ductility ratio (μ_{c4}), and the maximum displacement ductility ratio (μ_{cmax}). The definition of the damage state of the column by the displacement ductility ratio is shown in Table 3.

Table 3 Definition of the failure state

Failure state	Failure criterion
Nondamage	$\mu_d \leq \mu_{cyl}$
Slight damage	$\mu_{cyl} \leq \mu_d \leq \mu_{cy}$
Medium damage	$\mu_{cy} \leq \mu_d \leq \mu_{c4}$
Serious damage	$\mu_{c4} \leq \mu_d \leq \mu_{cmax}$
Complete damage	$\mu_{cmax} \leq \mu_d$

The expression of pier displacement ductility ratio is as follows:

$$\mu_d = \frac{D}{D_{cyl}} \quad (12)$$

where, μ_d denotes the displacement-ductility ratio, D is the relative displacement of the column top under seismic conditions (mm), D_{cyl} is the relative displacement of the column top when the steel bar of the column bottom section yields for the first time (mm).

Fig.6 Time-varying moment curvature analysis

To obtain the damage index, the section moment-curvature is analyzed. Under offshore environments, the RC bridge columns in the service period are heavily affected by chloride ion erosion, and the mechanical properties of the materials will degrade, which can influence the moment-curvature change of the column section. The column moment-curvature under different service periods is shown in Fig. 6. In the whole life cycle, the mechanical properties of the reinforced steel are decreased due to the corrosion of the steel bar. The slope degradation rate of PSBC is greater than that of CPBC, and the ultimate curvature of PSBC decreases more obviously with the extension of bridge service time due to the effects of chloride ion erosion. The equivalent and ultimate bending moments of CPBC and PSBC decreased by 19%, and 22% and 30%, and 39%, respectively, when the column reached the service life.

After the moment curvature analysis, the displacement ductility ratio can be calculated as follows:

$$\mu_{cy1} = 1 \quad (11)$$

$$\mu_{cy} = \frac{\Delta_{cy}}{\Delta_{cy1}} = \frac{\phi_y}{\phi_y'} \quad (12)$$

$$\mu_{c4} = \frac{\Delta_{c4}}{\Delta_{cy1}} \quad (13)$$

$$\Delta_{c4} = \Delta_{cy} + \Delta_{p4} = \frac{1}{3} \cdot \phi_y \cdot L^2 + \theta_{p4} \cdot (L - \frac{1}{2} \cdot L_p) \quad (14)$$

$$\mu_{cmax} = \mu_{c4} + 3 \quad (15)$$

where, Δ_{cy} is the relative displacement of the column top when the column section yields equivalently(mm); Δ_{c4} 、 Δ_{p4} 、 θ_{p4} is the relative displacement of the column top when the strain of concrete for column protection is 0.004(mm), ideal elastic-plastic displacement(mm), permissible angle, respectively; L is the height of the cantilever; L_p is the length of the equivalent plastic hinge region(mm).

Table 4 Time-varying damage indexes of CPBC and PSBC

Time/a	Δ_{cy1}/mm		Δ_{cy}/mm		Δ_{c4}/mm		μ_{cy1}		μ_{cy}		μ_{c4}		μ_{cmax}	
	CP	PS	CP	PS	CP	PS	CP	PS	CP	PS	CP	PS	CP	PS
	BC	BC	BC	BC	BC	BC	BC	BC	BC	BC	BC	BC	BC	BC
0	195.6	166.3	244.5	269.1	533.4	409.0	1	1	1.25	1.62	2.73	2.46	5.73	5.46
25	195.0	167.3	243.5	265.3	530.1	407.3	1	1	1.25	1.59	2.72	2.43	5.72	5.43
50	187.7	170.8	238.3	257.5	532.9	402.3	1	1	1.27	1.51	2.84	2.36	5.84	5.36
75	179.5	174.3	229.7	250.9	537.4	396.1	1	1	1.28	1.44	2.99	2.27	5.99	5.27
100	171.5	158.3	219.4	236.2	537.8	389.0	1	1	1.28	1.49	3.14	2.46	6.14	5.46

Considering the service environment of the investigated columns and the related parameters in Table 1, the displacement ductility ratio can be obtained. The time-varying damage indexes of CPBC and PSBC are shown in Table 4. In the whole life cycle, the displacement of the column top of CPBC and PSBC decrease with the extension of the service time under four damage damages; under the slight damage stage, the damage index remains unchanged, under the moderate damage, severe damage and complete damage stages, the damage index of CPBC increased with the extension of service time, and the damage index of PSBC decreased with the extension of service time.

4.3 Time-variant fragility analysis

(d) 75a

(e) 100a

Fig.8 Probabilistic seismic demand model in different periods of service

Based on the 150 seismic waves obtained by amplitude modulation, 150 relative displacement ductility ratios corresponding to every column are selected for logarithmic linear regression analysis, the probabilistic seismic demand model between seismic demand (D) and seismic intensity parameter (I_M) is obtained, as shown in Fig.8. Then the exceeding probability under different damage states in the whole life cycle of the column can be calculated, the time-varying vulnerability curve can be drawn.

In the whole life cycle, the time-variant fragility surfaces of CPBC and PSBC are shown in Fig. 9 and Fig. 10. The comparison indicates that under the four damage stages, the exceeding probability of PSBC is higher than that of CPBC. Under the same stages, the growth rate of the exceeding probability of PSBC with the extension of service time is greater than that of CPBC. Therefore, PSBC is more prone to damage than CPBC in the long-term service process.

Fig.9 CPBC time-varying fragility surface

(a) Slight damage

(b) Moderate damage

(c) Serious damage

(d) Complete damage

Fig.10 PSBC time-varying fragility surface

In order to directly compare the exceeding probability of PSBC and CPBC under different PGAs in the whole life cycle, the time-varying fragility curve and the exceeding probability difference under the four damage states are drawn, as shown in Fig.11 and Fig.12.

(a) Slight damage

(b) Moderate damage

(c) Serious damage

(d) Complete damage

Fig.11 Fragility curve

The exceeding probability curve under the four damage states are shown in Fig. 11. It can be seen that the exceeding probability of PSBC is always greater than that of CPBC in the whole life cycle. When the designed service life is reached, the maximum exceeding probabilities of PSBC and CPBC are 99.6% and 95.7% under the slight damage stage, which were 0.7% and 4.1% higher than that of the completed bridge, respectively. The maximum probabilities were 97.2% and 89.1% in the moderate damage stage, which were 6.5% and 7.2% higher than those in bridge completion, respectively. The maximum probability was 82.2% and 28.4%, which were 17.3% and 4.8% higher than that of the bridge completed. When the bridge was completely destroyed, the maximum exceeding probability was 25.1% and 2.8%, which increased by 75.6% and 53.6%, compared to the bridge completed, respectively. In the four damage states, the exceeding probability of CPBC and PSBC increases with the extension of bridge service time and the increase of PGA. The initial corrosion time of the first row of CPBC longitudinal bars is 22 years, and that of the second row longitudinal bars is 50 years. The damage of the CPBC bridge column is small in 0-25 years. Due to the discontinuity between segments, the corrosion time in PSBC is earlier than that in CPBC, the first row of ED bars at the PSBC segment began to rust in the 8th year, and the second row began to rust in the 14th year. The exceeding probability of PSBC increased rapidly with the extension of the service time of the bridge. Taking the medium damage with $PGA=0.5g$ as an example, compared with that of the bridge when it has been in service for 25, 50, 75 and 100 years, the exceeding probability of the CPBC bridge column increased by 1.25%, 8%, 18% and 33%, and the exceeding probability of PSBC increased by 5%, 17%, 28% and 46%.

Fig.12 Exceeding probability difference of vulnerability curve

Fig.12 shows the difference of exceeding probability between CPBC and PSBC under four damage states. In the four damage conditions, when PGA is less than 0.3g, the difference of exceeding probability is small. With the increase in PGA, the exceeding probability difference reached the maximum value around PGA=0.6g, and the maximum exceeding probability difference was approximately 32%. When PGA=1.0g, the exceeding probability difference gradually decreased to approximately 10%. In the condition of severe damage and complete damage, the exceeding probability difference increases all the time and reaches the maximum value around PGA=1.0g. In the condition of severe damage, the exceeding probability difference reaches 54%, and in the condition of complete damage, the exceeding probability difference reaches 22%.

Supposing that the vulnerability curve follows a logarithmic positive distribution, linear regression analysis was carried out for different exceeding probabilities of PGA to calculate the median PGA, which is defined as the PGA value corresponding to exceeding 50% exceeding probability of specified damage state. The time-varying damage index of the column in different damage states is taken as the mean value of the structural seismic capacity μ_C replacing the mean value

of the structural seismic demand in equation (8) μ_D , the time-varying median PGA of the columns under different damage states can be obtained:

$$I_M = e^{\frac{\ln(\mu_C) - \ln a}{b}} \quad (16)$$

Table 5 Time-varying median PGA

Time/a	Slight Damage		Moderate Damage		Serious Damage		Complete Damage	
	CPBC	PSBC	CPBC	PSBC	CPBC	PSBC	CPBC	PSBC
0	0.521	0.346	0.64	0.537	1.322	0.786	2.633	1.627
25	0.518	0.342	0.637	0.522	1.316	0.772	2.626	1.607
50	0.497	0.338	0.621	0.492	1.289	0.739	2.582	1.561
75	0.473	0.335	0.596	0.465	1.248	0.702	2.521	1.502
100	0.448	0.298	0.563	0.426	1.190	0.663	2.427	1.347

Then, the median time-varying PGA of CPBC and PSBC under different damage states is shown in Table 5. In the whole life cycle, under the four damage states, the median PGA of CPBC and PSBC decreased with the bridge service time extension and increased with the aggravation of damage state; The median PGA of PSBC is less than CPBC, and the reduction ratio is greater than CPBC with the extension of bridge service time; The corrosion time of the first row of longitudinal bars of CPBC is approximately 22 years, while the corrosion time of the second row of longitudinal bars is approximately 50 years, the median PGA at 25 years of service is not different from that at the completion of the bridge. Taking the medium damage state as an example, the median PGA at 50, 75 and 100 years of CPBC is reduced by approximately 3%, 7% and 12%, respectively compared with that at the completion of the bridge; the corrosion time of the first row of ED bars of PSBC is approximately 8 years, while the corrosion time of the second row of ED bars is approximately 14 years, which is far earlier than that of the reinforcement in CPBC, the durability damage of PSBC is more serious than that of CPBC, the median PGA of PSBC after 0, 25, 50, 75 and 100 years of service is approximately 16%, 18%, 21%, 22% and 24% lower than the corresponding value of CPBC, respectively.

5 Conclusions

Based on a comprehensive comparative study of the time-varying seismic fragility analysis of the PSBC and CPBC over the whole life cycle, the main conclusions are as follows:

(1) In the whole life cycle, considering the influence of chloride ion erosion, the diameter, yield strength and ultimate strength of the reinforcement decrease with the extension of the service time. After 100 years of service, the diameter, yield strength and ultimate strength of the CPBC reinforcement are reduced by 11%, 10% and 14%, respectively; the diameter,

yield strength and ultimate strength of the PSBC are reduced by 32%, 26% and 35%, respectively.

(2) When the bridge is formed, the exceedance probability of PSBC is similar to that of CPBC in the four states, and the difference in fragility is small. With the aggravation of the damage state, the exceeding probability decreases gradually. When the damage state was relatively light, there was a small difference between PSBC and CPBC exceeding probability. When the damage state was severe, the exceeding probability of PSBC was greater than that of MR due to the discontinuity between PSBC segments. The exceeding probability of PSBC and CPBC was 99.6% and 95.7%, respectively, the maximum value was 97.2% and 89.1% in moderate damage, the maximum was 82.2% and 28.4% under severe damage stage, the maximum was 25.1% and 2.8% under complete damage.

(3) In the whole life cycle, the PGA median value of CPBC and PSBC decreases with the extension of bridge service time and increases with the aggravation of the damage state. The PGA median value of PSBC is less than that of CPBC, and the durability damage of PSBC is more serious than that of CPBC. Taking the medium damage condition as an example, the median PGA of PSBC is 6%, 18%, 21%, 22% and 24% lower than the corresponding median PGA of CPBC when the column has been in service for 0, 25, 50, 75 and 100 years.

The comprehensive seismic performance of PSBC is weaker than that of CPBC, which is not conducive to popularization and application in medium- and high -intensity areas. In the follow-up, the influence of adding high-performance materials such as high-strength steel strands and UHPC on the time-varying seismic vulnerability of PSBC should be considered. At the same time, it is also necessary to further study the impact of bridge maintenance on the seismic performance and time-varying seismic vulnerability of PSBC.

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Author Contributions

All authors contributed to the study conception and design. Material preparation, data collection and analysis were performed by [Yan Liang], [Jing-Xiao Shu] and [Yang Cao]. The first draft of the manuscript was written by [Qing-He Li], [Pan-Jie Li] and all authors commented on previous versions of the manuscript. All authors read and approved the final manuscript.

Data Availability

The datasets generated during and/or analysed during the current study are available from the corresponding author on reasonable request.