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Fragility Analysis and Risk Assessment of Precast Concrete Frames with “Dry” Connections: A Comparative Study

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Abstract. Precast concrete frames (PCFs) with "dry" connections and self-centering capacity have been proposed as a new kind of seismic protective structural system with characteristics of damage controllable mechanism, easy-assembly and rapid repair speed. The damage mechanism of PCFs are concentrated at the panel zones under earthquake excitations, so as to avoid damage to beam and column components. Through reasonable design for the PCFs, not only the structural and life safeties can be guaranteed, but also the seismic loss and social impact can be minimized. This paper conducts a comparative study between PCFs with "dry" connections and conventional cast-in-situ concrete frame. A generalized beam-column connection analytical model is utilized to predict the seismic behaviour of PCFs with energy dissipation devices, with an emphasis on the opening behaviour at beam-column interfaces, the self-centering capacity provided by prestressed tendons and the hysteresis behaviour provided by energy dissipation devices. Prototype PCFs or cast in situ frame structures are designed to achieve similar deformation capacities in Chinese highly seismic fortification zone. Probabilistic seismic capacity analyses (PSCA) are conducted based on the results of probabilistic pushover analyses and Latin Hypercube Sampling. Incremental dynamic analysis method combined with nonlinear time history analyses are utilized to conduct probabilistic seismic demand analyses (PSDA). Fragility functions of different structural systems are derived based on the convolution of PSCA and PSDA. Finally, the seismic risk is evaluated based on the fragility functions and the developed Chinese seismic code compliant hazard functions. The results indicate that PCFs with energy dissipation devices can have lower seismic risk than conventional cast-in-situ frames.

1. Introduction

With the development of seismic codes and application of ductile structural systems, structural safety and life safety can be guaranteed under earthquake excitations. However, the seismic loss and social impact may far exceed the construction cost or time of the structures themselves. According to the post-earthquake reconnaissance of 2011 Christchurch earthquake [1], many structures had to be demolished and reconstructed due to unacceptable residual displacements, even if the structures had not yet reach collapse limit state. Bruneau et al. [2] firstly proposed the concept of seismic resilience, which is a measurement that comprehensively evaluate the recovery and functionality of the structures before, during and after earthquake. Many researches have been conducted around the topic of resilience henceforth, including developing innovative seismic protective structural systems, proposing new resilience-based assessment methodology for infrastructures with different functions, etc. Among which



the earthquake resilient structures have the capacity of rapidly recovery to pre-earthquake functionality, with slight or even no repairs [3].

Precast concrete frames (PCFs) have been widely adopted in engineering practices in China. The advantages lie in more rapid construction speed, materials and labor saving, and improved quality of assemblage. According to their beam-column connection types, the PCFs can be further divided into “wet” connected and “dry” connected structures. As for the “wet” connected structures (also known as emulative precast systems in IBC-2006 [4]), the precast beam and column components are assembled on site with cast-in-situ concrete within the range of beam-column connection. While for the “dry” connected structures (also denoted as “non-emulative” precast system in Morgen et al. [5]), the precast components are connected through bolts, welded plates, prestressed tendons (PTs) among other connection measures. Compared with the “wet” one, the “dry” connections need no cast-in-situ concrete during the process of on-site construction, which can be beneficial for speeding up construction speed and saving labor consumptions. The earliest development of PCFs with “dry” connections was carried out by Priestley and his coworkers in the “PRESSS” program [5-8]. It was found that the lateral displacement of PCFs without configuring additional hysteresis devices may exceed that of cast-in-situ reinforced concrete frames (RCFs) due to the lack of energy dissipation capacities within the connection regions, which might be a great setback to promotion PCFs with “dry” connections. Therefore, in the past two decades, in order to reduce the lateral deformation demands of PCFs with unbonded PTs only, many researchers proposed various types of energy dissipation devices located at their beam-column interfaces to enhance seismic performance (e.g., passive dampers [5], mild steel reinforcements [6], web-friction devices [9] among others). Accounting for their reduced lateral deformation demands and satisfactory seismic behaviors, “dry” connected precast structures can be served as a kind of earthquake resilient structure, which can not only ensure structural safely under severe earthquake events, but have the potential of restoring structural functionality rapidly and mitigating seismic loss to a large degree.

It should be noted that most of the studies focus on conducting tests on PCFs with “dry” connections (Morgen et al. [10]; Song et al. [11]), developing brand new design methodology for self-centering structures (Lu et al. [3]; Song et al. [9]); and conducting seismic response analyses of these structures (Priestley and Tao [7]; Song et al. [9]). To the best of the author’s knowledge, there have been few studies investigating and comparing PCFs with conventional RCFs in terms of fragility and risk. Therefore, an in-depth fragility and risk-based study is conducted to compare the performance between RCFs and PCFs which are designed to satisfy the Chinese seismic requirements at seismic zones with high intensity. A generalized beam-column connection analytical model is utilized to predict the seismic behavior of PCFs with energy dissipation devices. The predictability and applicability of the analytical model are validated through the experimental results from previous studies. Two typical types of PCFs are designed based on the displacement-based design methodology to achieve similar deformation capacities to RCF. To evaluate the seismic capacity of the corresponding three structures under different damage states, probabilistic seismic capacity analyses (PSCA) are conducted based on the results of probabilistic pushover (PO) analyses. Incremental dynamic analysis (IDA) method combined with nonlinear time history analyses are utilized to conduct probabilistic seismic demand analyses (PSDA). Then, fragility functions of different structures can be determined through the convolution of the results of PSCA and PSDA. Finally, the seismic risk of different structures can be derived based on the fragility functions and analytical seismic hazard functions. The study shows that the PCFs with web friction devices can reduce seismic risk in Chinese highly seismic fortification zones, combined with potential economic benefits and rapid construction speed.

2. Methodology for analytical seismic fragility and risk formulations

One of the most important steps in performance-based seismic assessment of a structure is to estimate the mean annual frequency (MAF), λ_{DS} , of the structure to exceed a specified damage state (DS), which can be expressed as follow, based on the total probability theorem [12]:

$$\lambda_{DS} = \int_{im} F_R(im) |d\lambda_{IM}(im)| \quad (1)$$

The terms appearing in the equation are the rate of exceeding a certain value of ground motion intensity measure (IM), λ_{IM} ; and the structural fragility function $F_R(im)$. The term λ_{IM} is a measure of seismic hazard at a given site and can be evaluated per seismic hazard analysis. The fragility function, which defines the conditional probability of attaining or exceeding certain DS as a function of varying IM value, generally adopts the lognormal distribution model as [12]:

$$F_R(im) = \int_{edp} G_C(d) dG_{D|IM}(d|im) = \Phi \left(\frac{\ln m_{D|IM} - \ln m_c}{\sqrt{\beta_{D|IM}^2 + \beta_C^2 + \beta_M^2}} \right) \quad (2)$$

where $\Phi(\cdot)$ is the CDF of standard normal distribution; $G_C(d)$ represents the seismic capacity model and demand model with median capacity m_c and dispersion β_C , and $G_{D|IM}(d|im)$ represents the seismic demand model with conditional median value $m_{D|IM}$ and dispersion $\beta_{D|IM}$. β_M is the modeling dispersion. The proper values of these parameters can be generated through probabilistic seismic analysis or obtained from relative technical guidelines (e.g., GB 50011-2010 [13], HAZUS [14] among others). The former approach is suitable for PCF structural systems since few data is available in literature.

The probabilistic seismic demand analysis establishes the relationship between seismic demand and IMs, which is assumed to follow a power law [12]:

$$m_{D|IM} = a im^b \quad (3)$$

Eq. can be rewritten in logarithmic form:

$$\ln(m_{D|IM}) = \ln a + b \ln(im) \quad (4)$$

where the fitting parameters a and b can be established along with $\beta_{D|IM}$ by loglinear regression based on the results from some probabilistic seismic method (e.g., incremental dynamic analysis (IDA)).

The hazard curve can also be approximated by a power law relationship [12,15]:

$$\lambda_{IM}(im) = k_0 \cdot im^{-k} \quad (5)$$

where k_0 and k are fitting parameters based on given seismic hazard data. It has been demonstrated that the power law generally fits seismic hazard data satisfactorily, but may overestimate the hazard beyond the range of hazard data [15]. Eq. 5 is often applied when only limited hazard data are available or an analytical formulation is desired for risk assessment. For example, substitute the fragility function Eq. 2 and hazard function Eq. 5 into Eq.1, the closed-form expression for the annual probability of exceeding certain DS can be derived [16]:

$$\lambda_{DS} = k_0 \left(\frac{m_c}{a} \right)^{-\frac{k}{b}} \exp \left[\frac{k^2}{2b^2} (\beta_{D|IM}^2 + \beta_C^2 + \beta_M^2) \right] \quad (6)$$

3. Modelling the beam-column connections in PCFs

The most notable difference between PCFs with “dry” connections and RCFs lies in the nonlinear behaviours within the beam-column connection range. In this section, a generalized “dry” connection model is developed by OpenSees [17] for simulating the nonlinear response under earthquake excitations which is similar to the models proposed in previous studies [3,10]. Numerous experimental results are compared with the numerical results, to verify the accuracy and applicability of the proposed model.

3.1 Analytical model for “dry” beam-column connection

The keys to simulate nonlinear behaviours of “dry” connections mainly lie in: i) the gap opening behaviour at the beam-column interfaces. ii) The self-centering behaviour provided by unbonded PTs, and iii) The hysteresis behaviour provided by supplementary energy dissipation devices (web friction devices are adopted in current study). These three typical behaviours of “dry” connections contribute to the so-called “flag-shaped” hysteresis behaviour.

Due to the relaxed constraints between precast components, the beam-column interfaces can only undergo compression without tension. When the moment demand at interfaces exceeds the gap-opening moment, the opening is expected to occur. After earthquake event, all the gaps will close under the self-centering forces provided by unbonded PTs, and the precast components will restore their original positions. The “gap-opening” behaviour at the beam-column interfaces is the most predominant characteristic of such connection, through which the damage can be transformed from beam column elements to beam-column connection regions.

In order to simulate the opening mechanism at beam-column interface, concrete and reinforcements fibers at the beam ends are modelled with *Concrete01* and *ElasticPPGap* materials, respectively, to represent the no-tension behaviour of the beam-column interfaces. Unbonded PTs are modelled by *truss* elements with *Steel02* constitutive model, which can provide initial stress accounting for initial prestressing force (with proper consideration of initial prestress loss). Fiber-based beam-column elements are used to simulate precast components. The effect of the supplementary energy-dissipation devices located at the beam-column interface are equivalently arranged on the four corners of the connection region, using *zero-length* rotational elements assigned with hysteresis models. Different hysteresis rules should be considered to allow for different nonlinear behaviours of energy-dissipation behaviours. For example, *ElasticPP* materials will be utilized to represent the behaviour of web-friction devices proposed by Song et al. [9, 11], considering their plump and stable hysteresis behaviours. The schematic representation of the analytical model is shown in Figure 1.

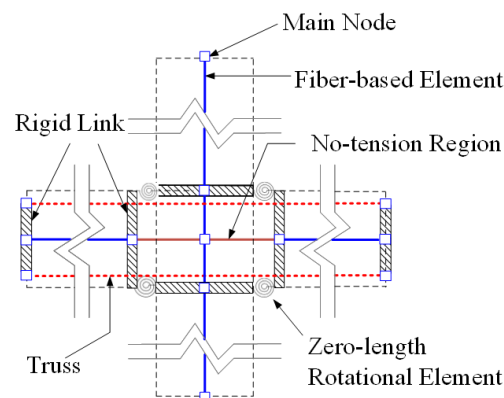


Figure 1. Schematic representation of the analytical model for the “dry” connection.

4. Design and modelling of prototype frame structures

A five-story official building structure is designed and modelled in this section. Three alternative structural systems are considered for better comparison: i) a conventional ductile RCF; ii) Precast concrete frame with unbonded PTs only (PCF-I) and iii) Precast concrete frame with unbonded PTs and web friction devices (PCF-II). The building is assumed to be located in China with seismic cautionary intensity of eight (equivalent to a design PGA of 0.2g at design based earthquake (DBE) level). According to the site classification method proposed in Chinese seismic code [13], a hypothetical site of stiff soil, class II category with a design characteristic period of 0.35s is considered. This site condition can be regarded to straddle the boundaries of NEHRP C and D site class [18]. Due to the lack of design codes for PCFs with “dry” connections, the displacement-based design method proposed by Lu et al. [3] is utilized to design the PCFs with “dry” connections. According to Lu’s design method, there are two design phases (i.e., elastic and nonlinear design phases) involved, in which the latter follows the displacement-based seismic design similar to Priestley et al. [19]. During the elastic design phase, plan configurations and section sizes of precast components are determined, while for the nonlinear design phase, the design of local reinforcement, PT steel and energy-dissipation devices can be determined accordingly, to make the structure meet the predesignated target deformation under MCE earthquake level, $\theta_{max,tar}$. The design method for web friction devices according to Song et al. [9] is utilized to determine the numbers of friction bolts associated with the connection resisting moment, M_{fj} , of web friction devices, based on prescribed energy dissipation ratio, β_E . In order to achieve similar levels of deformation capacities, the target maximum inter story drift (MIDR) at MCE earthquake level, $\theta_{max,tar}$ is set to be 1.8% for all of the structural systems. The reinforcements configurations of components are checked to satisfy the seismic demand requirements. Trial and error procedure is performed to achieve ultimate design results to meet the target displacement requirement for both of the structures. The beam-column connections of PCF-II are designed with web friction devices at each side of beam ends, with constant friction forces. Figure 2 depicts the plan and elevation view of the prototype buildings.

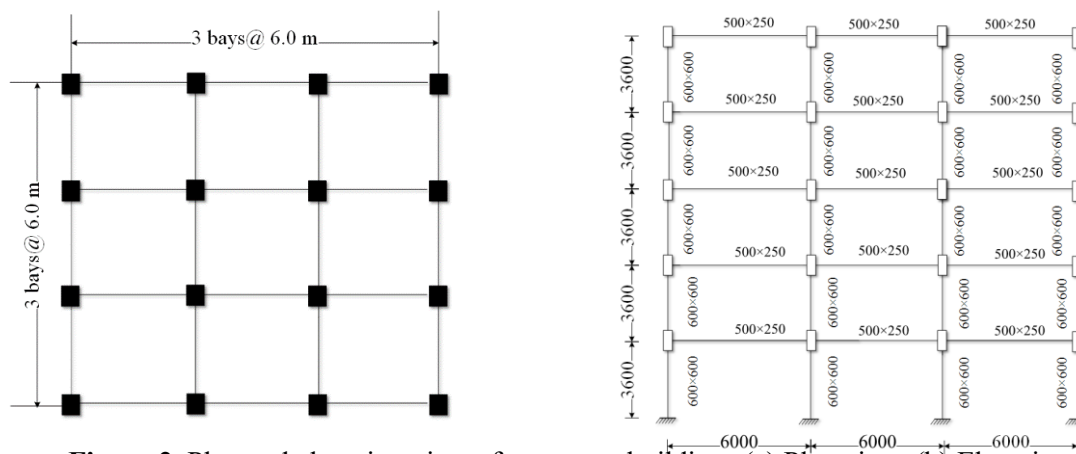


Figure 2. Plan and elevation view of prototype buildings (a) Plan view. (b) Elevation and member sizes.

For better comparison, the elongation of the effective damping due to inelastic deformation is not considered, 5% Rayleigh damping is applied to the 1st and 2nd period. 20 ground motion records selected by Lu et al. [3] are reused here, each record is scaled to PGA = 0.2g, 0.4g and 0.6g, corresponding to design-based earthquake (DBE), maximum considered earthquake (MCE) and extremely rare earthquake (ERE) at seismic cautionary intensity of eight defined at Chinese seismic codes [13, 20]. Nonlinear dynamic analyses are conducted based on the selected and scaled ground motions of both three structural systems. The MIDRs of the corresponding three structural systems under

the selected 20 ground motions at MCE levels are shown in Figure 3, the median MIDRs of the RCF, PCF-I and PCF-II are all controlled to be 1.8%.

Figure 4 shows the MIDR and residual drift ratio (RDR) profiles along the story height at DBE, MCE and ERE hazard level, correspondingly. The precast concrete frames have similar distribution of median MIDRs profiles compared with the RCF counterpart. The PCFs have significantly reduced RDRs even at the most intense hazard level, due to the self-centering capacity provided by PTs. The PCF-II structure has smaller MIDR profile along higher story levels than PCF-I, through reasonable web friction devices configuration. Compared with PCF-I, the PCF-II shows a relatively larger residual deformation at ERE level. This is because that in order to ensure comparability among the three structural systems, limited and controlled amounts of PTs are assigned to across the beam sections of PCF-II. However, the RDR level is far from that the PCF-II needs to be demolished. It should be noted that in order to ensure a similar seismic performance of the three structural systems, the configurations of component sizes, local reinforcements, PTs and web friction devices might not represent an optimal design.

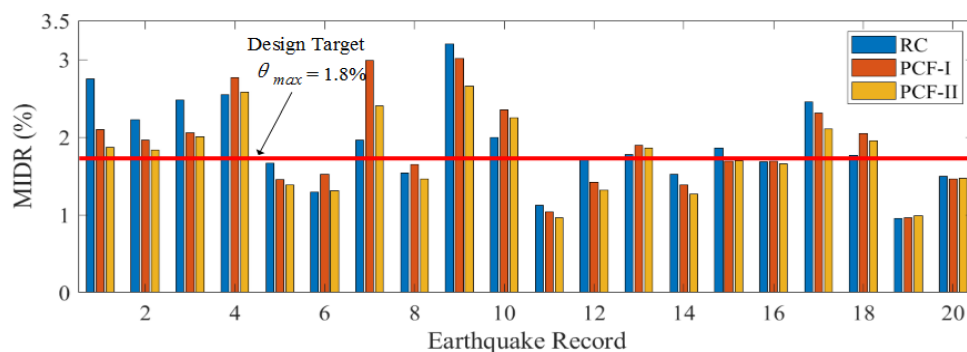


Figure 3. Median MIDR distribution of each earthquake record under MCE level.

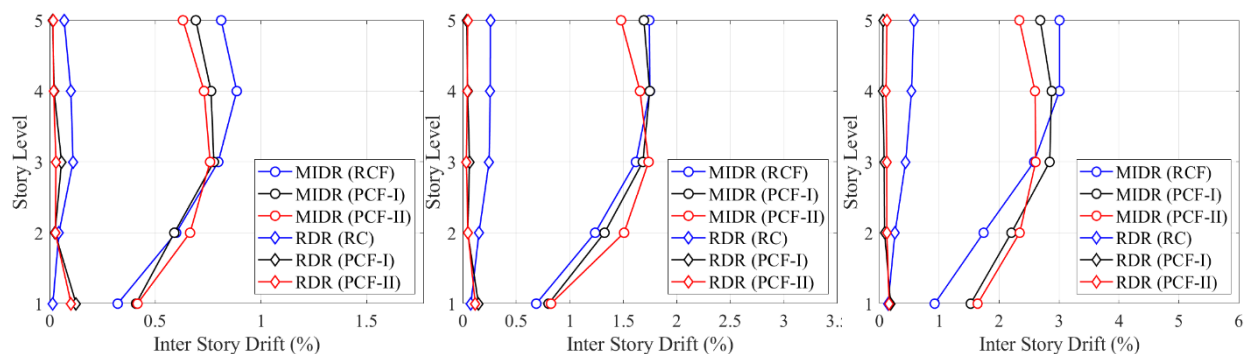


Figure 4. Median MIDR and RDR profile of different structural systems under (a) DBE hazard level, (b) MCE hazard level and (c) ERE hazard level.

5. Fragility analysis and risk assessment

In this section, the three structural systems designed in the previous section are utilized to evaluate their seismic fragility and risk. The fragility analysis is disaggregated into two steps: PSCA and PSDA. The uncertainty of material property and static loads are considered in PSCA, while for PSDA record-to-record variability is considered. To generate the parameters of seismic capacity, probabilistic PO analyses model based on Latin hypercube sampling (LHS) are conducted to identify the structural capacity parameters corresponding to different damage states. To obtain the parameters of seismic demand, IDA method is carried out, then the parameters can be identified via logarithmic linear fitting. Based on the parameters identified in PSCA and PSDA, the seismic fragility function of the

corresponding three structural types can be determined. Finally, convolute the fragility function with the analytical seismic hazard function, seismic risk of prescribed damage states can be calculated.

5.1 Probabilistic seismic capacity analysis

The predominant purpose of PSCA is to judge whether a structural system attain of exceed a specified damage state at given seismic demands, which can be served as an ingredient of fragility function. Currently, different codes or specifications (e.g., GB50011-2010 [13], HAZUS [14] among others) recommend different damages stages and their corresponding thresholds for conventional structural systems. However, these recommended thresholds are always determined by engineering judgements and tend to be conservative, which might not applicable to new structural systems. Therefore, a probabilistic PO analysis proposed by Lu et al. [16] are utilized to conduct PSCA on the three structural systems.

In order to consider the uncertainty of material properties along with static loads, 100 groups of random samples are selected based on LHS method. Twelve groups of random variables that affect structural capacity, including permanent and variable loads, concrete and steel properties are considered herein. The statistical parameters of the corresponding 12 random variables are summarized in Table 1, where the mean value of the parameters correspond to the nominal values specified in Chinese load code [21]. Other material properties not listed in Table 1 are deemed deterministic. No correlation is considered expected that the six material variables of concrete proposed by Barbato et al. [22]. Thus, a hundred groups of probabilistic PO analyses based on inverted-triangular lateral load pattern can be conducted on the probabilistic structural models. After the completion of each PO analysis, a capacity curve (i.e., the MIDR vs. base shear curve) can be derived.

Table 1. Statistical parameters of the selected material properties and static loads (in nominal values)

Uncertainty Class	Random Variable	Mean	Standard Deviation	Distribution
Static Loads	Permanent Load [21 ,23]	26.5kN/m ³	1.8497	Normal
	Variable Load [21 ,23]	0.98kN/m ²	0.4018	Gamma
Grade C30 Concrete	$f_{cp,cover}$ [13,16]	26.1MPa	3.6540	Lognormal
	$\varepsilon_{cp,cover}$ [22, 24]	0.004	0.0008	
	$f_{cp,core}$ [24, 25]	33.6MPa	7.0560	
	$\varepsilon_{cp,core}$ [24, 25]	0.0022	0.0004	
	$f_{cu,core}$ [24, 25]	22.2MPa	4.6620	
	$\varepsilon_{cu,core}$ [24, 25]	0.0133	0.0069	
Grade HRB400 Steel	f_y [13, 26]	452MPa	26.46	Lognormal
	E_s [13, 26]	200,000MPa	4000	
Prestressed Tendon ^a	f_p [26]	1934.4MPa	48.36	Normal
	E_p [26]	195,000MPa	3900	

^a f_p = ultimate tensile strength of PT; E_p = Elastic Module of PT.

Four damage states (DSs) are defined based on MIDR: slight damage (SD), moderate damage (MD), extensive damage (ED) and collapse prevention (CP). The feature points corresponding to the four DS will be determined from each capacity curve following the rules as follow: i) the SD state feature point is defined as the linear limit of the capacity curve [16]; ii) the MD state feature point is defined as the yield point of the equivalent perfectly elastoplastic system based on energy equivalent method [27]; iii) the ED state feature point is defined as the peak plateau of capacity curve and iv) the CP damage state feature point is defined as the capacity point at which 15% maximum base shear is reduced [28]. According to the definition of collapse-prevention damage states from JGJ/T101[28], the structures are

close to their ultimate strength capacity, and the CP feature point can be regarded as a near-collapse damage state. It is assumed that the rules of capturing the four typical feature points can reflect the whole process of damage development of concrete frames, which are also adopted by relevant studies (Lu et al. [16]; Cao et al. [29] among others).

The probabilistic capacity curves of the three structural systems are depicted in Figure 5, along with the captured feature points of the four DSs. The three structural types show similar fundamental period with median values of 0.94s, 0.92s and 0.94s for RCF, PCF-I and PCF-II, respectively. Statistical parameters of PSCA (i.e., the median capacity m_c and dispersion β_c) are summarized in Table 2. It can be observed that the median capacity of the SD and MD states for RCF are larger than that of PCFs. This is mainly because of the energy dissipation mechanism differ from RCF to PCF. For RCF, plastic hinges develop at local reinforcements to help the structure obtain ductility, while for PCF, the energy is dissipated through the gap opening and closing behavior at beam-column interface. Due to the reduced configurations of local reinforcements, the yield of PCFs is achieved earlier than RCF. But the collapse prevention capacities for PCFs are stronger than that of RCF, this is because PTs can go through long elasticity and will not yield until the structures experience displacement close to collapse. The median capacity identified in current study for both for the three structural types in DS1 and DS3 are larger than the deformation thresholds at the end of elastic and plastic phases specified in Chinese seismic code [13] (i.e., 0.18% and 2% respectively), which demonstrate the design methodology can meet the code-compliant seismic requirement.

Table 2. Summary of statistical parameters for the structural fragility analysis

Structural Type	DS	PSCA		PSDA			Fragility	
		$m_C(\%)$	β_C	a	b	$\beta_{D IM}$	$m_R(\text{g})$	β_R
RCF	DS1	0.273	0.212	0.033	1.002	0.383	0.084	0.480
	DS2	1.310	0.057				0.400	0.435
	DS3	2.694	0.059				0.822	0.435
	DS4	4.190	0.073				1.277	0.437
PCF-I	DS1	0.196	0.051	0.030	1.056	0.424	0.075	0.446
	DS2	1.010	0.122				0.355	0.458
	DS3	2.340	0.078				0.788	0.450
	DS4	4.700	0.119				1.525	0.458
PCF-II	DS1	0.230	0.050	0.031	1.114	0.505	0.097	0.490
	DS2	1.160	0.072				0.414	0.492
	DS3	2.990	0.194				0.968	0.518
	DS4	5.910	0.110				1.785	0.498

5.2 Probabilistic seismic demand analysis

The PSDA is to predict the potential EDPs attain or exceed a certain EDP level at given IM levels. The relationship between seismic demand for EDP and IM (associated with the regression coefficients in Eq. 4) serves as an interim step in deriving fragility functions. In this section, the MIDR and spectral acceleration at fundamental period [$S_a(T_1)$] are selected as the EDP-IM pair. Only the record-to-record variability is considered herein. Thus, the 22 pairs of far-fault ground motions recommended in FEMA P-695 combined with the IDA method [30] are utilized to develop the seismic demand model.

The scatter plots of MIDR vs. $S_a(T_1)$ in logarithmic coordinate, along with the log linear fitting results by Eq. 4 are shown in Figure 6. The regression coefficients a and b as well as the dispersion $\beta_{D|IM}$ are also labeled in the figures, and are summarized in Table 2. It can be seen that the PCF-II has smallest MIDR demands than RCF and PCF-I between the DBE and ERE hazard level.

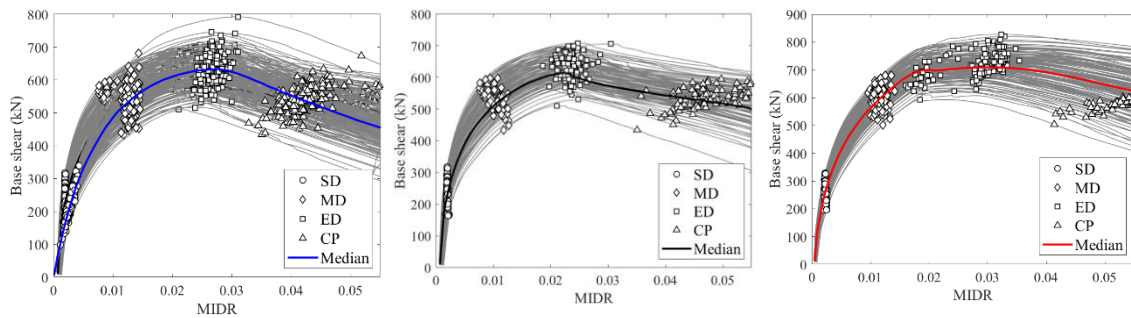


Figure 5. Probabilistic capacity curves for (a) RCF, (b) PCF-I, and (c) PCF-II. (d) is the normalized median capacity curve for the three structural systems.

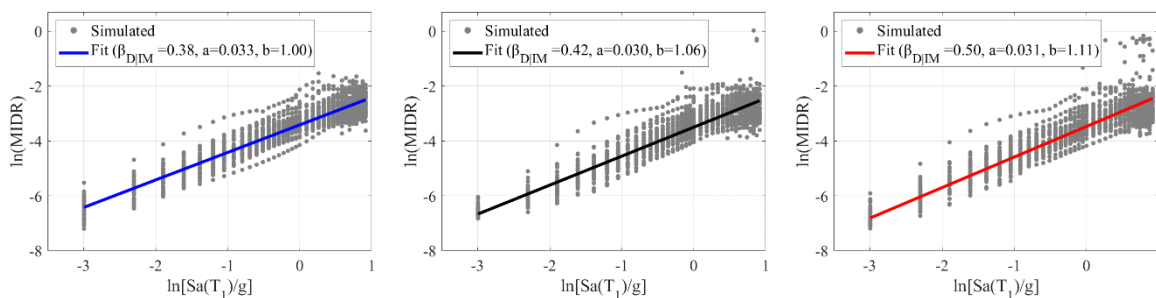


Figure 6. Probabilistic seismic demand models for (a) RCF, (b) PCF-I, and (c) PCF-II.

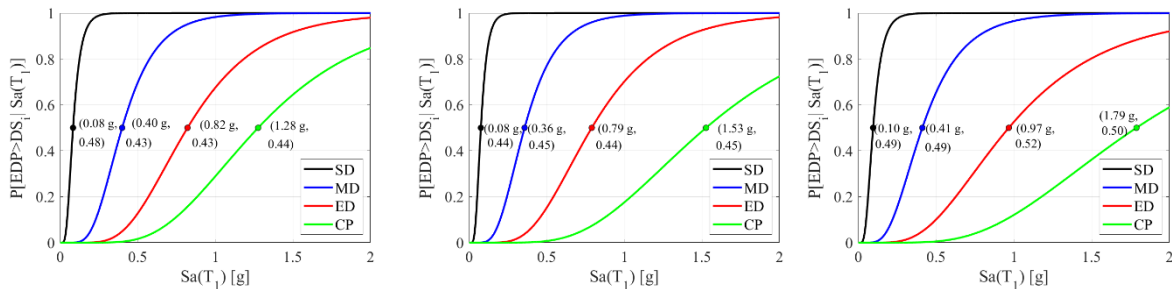


Figure 7. Comparison of fragility curves for (a) RCF, (b) PCF-I, and (c) PCF-II. (The data provided in parenthesis are log median and log standard deviation of IMs at different DSs)

5.3 Probabilistic seismic fragility analysis and risk assessment

As defined in Eq. 2, the seismic fragility functions can be derived based on the statistical parameters determined through the convolution of PSCA and PSDA. The modeling uncertainty β_M is taken as 0.2 herein [31]. Fragility functions of the three structural systems can then be derived for the four damage states (SD, MD, ED and CP) and are shown in Figure 7. The parameters of fragility functions, as well as that of PSCA and PSDA defined before are summarized in Table 2. As shown in Figure 7, the PCF-I with PTs only has an increase of median values of fragility curve at the CP damage state. This is mainly because that the PTs will not fracture until they undergo a long elastic stage, thus the collapse prevention resistance of PCFs are larger than RCF. The increment of m_R is more significant for PCF-II at all damage states, revealing that the configuration of web friction devices can facilitate a higher structural seismic resistance, and can ensure higher structural resistance at all damage states. No obvious variation in the dispersions of different DSs are observed.

The seismic risk represents the probability of exceedance of a specified damage state over a specified time period. The MAF of exceeding a specified damage state corresponds to a period of 1 year. In order to derive seismic risk of the structures, the seismic hazard functions in Eq. 5 should be determined first.

The fitting parameters in hazard function (Eq. 5) can be fitted through constraining the hazard curve to seismic hazard data at DBE and MCE level specified in Chinese seismic code [13]:

$$k = \frac{\ln(v_{DBE}/v_{MCE})}{\ln[S_{a,MCE}(T_1)/S_{a,DBE}(T_1)]}; \quad k_0 = v_{DBE}(IM_{DBE})^k. \quad (7)$$

where $v_{DBE} = 1/475$, $v_{MCE} = 1/2475$, $S_{a,DBE}(T_1)$ and $S_{a,MCE}(T_1)$ are annual rates of exceedance and design spectral acceleration value at fundamental period at DBE and MCE levels respectively. The design spectral acceleration at fundamental period corresponding to the RCF, PCF-I and PCF-II structural systems can be evaluated via Chinese seismic code, where the fundamental periods of the three structures are all taken as 0.9s. Substitute these values into Eq. 5 leads to

$$\lambda_{Sa} = 4.15 \times 10^{-5} x^{-2.38}; \quad (8)$$

Then, substitute seismic fragility function and hazard curve into Eq. 6, the annual mean seismic risk can be calculated (Table 3). Also, the failure probabilities of PCF-II are lower than the RCF at all damage stages expect for DS2. It can be concluded that adding PTs and energy dissipation devices can have lower seismic risk than conventional cast-in-situ frames.

Table 3. Comparison of the MAF of exceeding different DSs

	λ_{DS}			
	DS1	DS2	DS3	DS4
RCF	2.54×10^{-2}	6.12×10^{-4}	1.16×10^{-4}	4.20×10^{-5}
PCF-I	3.45×10^{-2}	8.85×10^{-4}	1.30×10^{-4}	2.75×10^{-5}
PCF-II	2.14×10^{-2}	6.75×10^{-4}	9.60×10^{-5}	2.11×10^{-5}

6. Conclusion

This paper investigates the seismic performance, fragility and risk of precast concrete structures with “dry” connections. A displacement-based design methodology is utilized to design the PCFs, in the light of the lack of existing design criteria for them in China. It is concluded that compared with the cast-in-situ concrete frame with the same design level, the PCFs can undergo less damage and negligible residual displacements at the same displacement demand of RCF. Through the additional configuration of web friction devices, reduced seismic fragility and risk are also observed at prescribed damage states. The good performance of self-centering and energy dissipation can facilitate rapid repair and cost saving for post-earthquake maintenance.

Comparison of the PSCA results with RCF shows PCF and higher initial stiffness and comparable peak strength and deformation capacity. Due to the application of web friction devices, increases of median capacity of all damage stages can be observed. Meanwhile, the collapse prevention resistance of PCFs are higher than RCF. According to the results of fragility analysis, the PCF with additional web friction devices can facilitate higher structural resistance and lower annual risk at all damage states. The additional configurations of web friction devices provide the structural system with large resisting moments, which might lead to the design of reduced component sizes and reinforcements configurations to meet the code requirements. The PCFs can not only ensure the structural safety under severe earthquake events, lower economic loss associated with shorter downtime can also be guaranteed. This study demonstrates that PCF with “dry connections” can serve as a resilient structural type, and provide a basis for further promotions in Chinese seismic fortification zones. It should be noted that the conclusions derived are applicable to the specific structural layout and design information used in this case study, further studies would be extended to PCFs with higher story levels and more spans.

The next step for this ongoing project includes the seismic economic loss and downtime of the PCFs, to highlight their construction simplicity, rapid repair speed associated with easy assemblage using the results of fragility analyses established in current study.

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