Dynamic and Probabilistic Seismic Performance Assessment of Precast Prestressed Rcfs Incorporating Slab Influence Through Three-Dimensional Spatial Model

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Dynamic and probabilistic seismic performance assessment of precast prestressed RCFs incorporating slab influence through three-dimensional spatial model

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Abstract

The dynamic and probabilistic seismic performances of precast prestressed RCFs are assessed in this paper, and the slab influence in the overall structural behavior is considered during the process. The three-dimensional spatial model is established to provide the numerical basis, and the slab is modelled through L-/T-section beam-slab fiber-sections considering the effective width and centroid positions. The adopted model is verified with the experimental data, and the slab influence in hysteresis curves is investigated by parametric study. Then, two groups of precast prestressed RCFs are well designed to evaluate the slab influence in dynamic responses through seismic excitations, and the modal analysis, roof displacement analysis, maximum and residual drift ratio analysis are conducted for discussion. Moreover, the incremental dynamic analysis and fragility analysis are also conducted to investigate the probabilistic performance of precast prestressed RCFs with or without slabs. In general, different demand parameters may result in the variability of analyzing results, and ignoring the slab influence may underestimate the structural capacity under the frequent earthquakes (i.e., elastic stage) and overestimate the structural capacity under the rare earthquakes (i.e., plastic stage). In a sense, the research proves the significance of slabs in the seismic performance of dry-connected precast prestressed RCFs, and meanwhile provides the reference for the further explorations of slab factors in precast concrete structures.

Keywords: Precast, Prestressed, Slab, Dynamic, Reinforced concrete, Probabilistic performance

1. Introduction

Prefabricated buildings possess the advantages of high-efficiency, ensured-quality, less-manpower and environmental-protection, which have been developed for half a century and have pointed out the future direction of construction industry for all countries [1]. With the promotion of precast theory and industrial process, the prefabricated technology continues to develop with the appearance of multiple novel forms, and the proportion of prefabricated buildings to the overall number continues to rise in the world range. According to the government proposal in China, the precast-assembly rate in the next 10 years is required to reach 30 %, thus, the academia and industry are paying great attention to this field from the aspects of micro mechanism to macro configuration [2, 3, 4].

Among all the precast structural types, the reinforced concrete frame (RCF) structure is the most commonly used one, which contains the superiority of flexibility in arrangement and convenience in standardization. Meanwhile, among all the components of precast RCFs, the beam-column connection is the
most important part that affects the seismic performance of the whole structure significantly, thus the research focuses are mainly on it. At this stage, the beam-column connection of precast RCFs can be divided into two types (e.g., wet-connection and dry-connection) according to whether there is cast-in-place concrete in the core area of the beam-column connection [5, 6]. In the early stage of precast technology, the wet-connection is widely studied and broadly applied because of its excellent performance equivalent to monolithic operation.

Im et al. [3] investigated the seismic performance of emulated precast wet-connection of beams to columns. Six full-scale specimens were adopted with U-shaped beam shells, and the parameters of reinforcement ratio as well as interface details were compared specifically. Besides, the headed reinforcing bars were selected at the connections to prevent the energy dissipation and stiffness degradation, which proved an effective approach in a sense. Choi et al. [7] developed a new type of precast wet-connection that was controlled using engineered cementitious composite (ECC) as casting material. All specimens indicated the flexural failure mode with plastic hinge appearance at beams, and ideal seismic behaviors were achieved with ACI experimental procedure. Ghayeb et al. [8] experimentally studied the hybrid precast beam-column connection under cyclic loadings. The steel bars were not yielding in the critical section of precast beams, and seismic performance can be improved obviously through stiffening the steel angles. Moreover, the concrete was not crushed at the connection of precast specimens. Nzabonimpana et al. [9] established a novel numerical model to reflect mechanical characteristics of precast beam-column connections. The joint performance and failure patterns can be accurately represented based on the extensive experimental validations. Besides, A variety of parameters (e.g., dilation angles and concrete damage) were considered and analyzed. Yan et al. [10] performed the experimental study on the precast wet-connection of beams to columns that was connected by grout sleeves. The results concluded that similar seismic responses can be acquired between the precast and cast-in-place connections, and the energy dissipation capacity or shear stiffness of the connection can be enhanced under a larger axial compressive ratio. Zhang et al. [11] proposed a new type of hybrid precast beam-column connection with ideal energy dissipative capacity, and six specimens were experimented under reversed loading. The results indicated satisfactory stiffness and strength of the precast connection, and the plastic hinge was transferred outside the core regions as designed. Fan et al. [12] proposed a novel precast self-sustaining wet-connection of beams to columns, and the experimental research was carried out based on the large-scale specimens, especially on the parameters of bar length and area. The test data demonstrated the satisfactory seismic performance compared with the monolithic connection, and the corresponding mechanical model incorporating the component yielding mechanism was also given.

Compared with the wet-connection, the dry-connection of beams to columns eliminates the post-pouring work, and is commonly formed with welded plates, tightening bolts, prestressed tendons and energy dissipators (Fig. 1). Meanwhile, the plastic deformation is mainly concentrated at the dissipators (i.e., outside the core region), while the prefabricated components always maintain good integrity and elasticity, which allows
the precast dry-connection to be quickly replaced and repaired after hazards [13]. Thus, the dry-connection is in line with the essence of prefabricated technology better, and can further maximize the characteristics of prefabricated buildings in the future. At this stage, the dry-connection researches are mainly focused on the configurational investigations and experimental explorations of two primary types: (1) Bolt-connected semi-rigid dry-connection, (2) Prestressed unbonded dry-connection.

As for the first type of dry-connection (i.e., bolt-connected semi-rigid type), Jiang et al. [14] experimentally investigated the anti-seismic behaviors of bolt-connected wave-web joints based on four full-size specimens under repeated quasi-static loading. The results showed that the plastic hinges were transferred outwards and the connections reflected certain energy dissipation capacity, which provided data basis for the further theoretical explorations and engineering applications. Cao et al. [15] proposed the precast bolt-connected dry-connection for the performance improvement of existing buildings, and cyclic tests were performed for three groups. The results indicated the enhanced stiffness and reduced displacement by this dry-connection sub-structure, and the bearing capacity was almost four times that of the non-retrofitting condition. Li et al. [16] experimentally analyzed the seismic behavior of high-strength bolt-connected joints with slot bolt holes, and six specimens with different bolt diameters or pretensions were well designed for comparison. Satisfactory energy dissipation capacity was realized by slot holes in the joints, and ideal lateral stiffness was achieved for dual-function improvement in the practical application. Chen et al. [17] researched on the static performance of bolt-assembled precast panel buildings based on a two-story half-scale specimen, and during the manufacturing process, bolted connections were properly adopted for precast components. Experimental results denoted the high bearing capacity and perfect structural integrity, and the shear failure mode appeared on the edge of concrete panels near bolt holes. Xue et al. [18] discussed the seismic performance of precast beam-column dry-connections based on four specimens, and longitudinal reinforcing bars were connected through bolt connectors in precast columns. Experimental results revealed the flexural failure pattern of all the specimens at the beam edge while no distinct damage was observed at the core regions. Moreover, a simplified degraded four-linear restoring model was given for this kind of dry-connection with bolt connectors in precast columns.

As for the second type of dry-connection (i.e., prestressed unbonded type), Yan et al. [19] experimentally analyzed a novel precast prestressed beam-column concrete connection by changing the concrete strength at the core regions, and the corresponding phenomena (e.g., damage development and capacity tendency) were captured during the cyclic loading. The results indicated that the prestressed tendons can limit the crack development in the precast components. Compared with the cast-in-place joints, similar failure modes and comparable bearing capacities of this prestressed connection can be well realized. Fan et al. [12] developed a novel precast prestressed dry-connection with unbonded dissipation bars, and three full-size specimens were experimented under reverse loading to explore the static behaviors. The results showed that greater bearing capacity and structural stiffness can be acquired by the proposed joint type, and comparable energy dissipation capacity to the cast-in-place specimen can be realized when the dissipation bars were located at the precast columns. Li et al. [20] performed the quasi-static pushdown tests to study the collapse mechanisms of prestressed precast beam-column joints, and different failure pattern was presented compared with the monolithic specimen. The test results demonstrated that the prestressed precast connection contained a higher collapse-resisting capability, and the tensile catenary effects in prestressed specimen were improved to some extent. Zhi et al. [21] experimentally investigated the shear-flexural performance of prestressed precast joints with high strength reinforcement, and the parameters of joint type, span ratio, initial prestress and longitudinal ratio were analyzed. The results indicated that the crack behaviors and shear capacities can be well enhanced with the control of prestress, and the equivalent performance to the monolithic specimen can be satisfied. Kim et al. [22] experimentally evaluated the seismic behaviors of a hybrid post-tensioned precast concrete connection by four full-scale specimens, and the variables of prestress length were considered during the process. The prediction model of shear strength was given and validated with the test data, and the acceptable static performance was proved quantitatively on the basis of the ACI 374 criteria.

With the development of connection technology, researchers find that the structural slab has a non-negligible effect on the seismic performance of beam-column joints, and the omission of slabs in experimental specimens may commonly underestimate the actual structural capacity in the practical engineering. To be specific, the slabs can enhance the compression capacity in negative moment zones from a macro perspective.
and reflect the shear-lag phenomenon of stress distributions from a micro perspective, which means an important structural factor in performance assessment. Gao et al. [23] researched on the seismic performance of precast steel reinforced concrete connection with or without slabs, and seven specimens were experimentally tested under cyclic loading to analyze the variation of structural capacities. The results signified better performance for the specimen with slabs, and the peak strength was decreased under the influence of bolt-slip. Santarsiero and Masi [24] discussed the effects of slab action on the structural behavior of beam-column connections, and the tension flange functions of slabs were evaluated specifically. The results showed the significant influence of reinforcing ratio in slabs, and the suggestions about slab effective width were also provided. Feng et al. [25] investigated the seismic performance of precast reinforced concrete beam-slab assembly in light of layered shell models, and the results showed the significant influence of slab thickness in the hysteretic behaviors. Besides, the slab width was found to have an obvious effect under a certain limit, but the effect was decreased distinctly after the slab width exceeded the boundary. Wang et al. [26] developed a prestressed beam-column connection with damage-free slabs, and experimental performance was analyzed compared to conventional connection. The results indicated satisfactory self-centering and energy dissipating capacities of the proposed joint, and the slab was damage-free even under large displacements. Moreover, the slab cracks were well reduced and the slab reinforcements were kept in elastic stage. Park and Mosalam [27] experimentally studied the non-ductile behavior of beam-column-slab connections without transverse reinforcement at the core regions, and two parameters (i.e., reinforcement ratio and aspect ratio) were mainly analyzed based on four specimens under static loading. The results presented the shear failure of connections as a result of transverse absence, and the effects of slabs on the shear strength were further verified with the recommendations in ASCE/SEI 41-06 [28].

From the literature review above, we can find that although extensive researches on dry-connection have been conducted, the existing approaches mainly focus on the experimental explorations of different configurations on the joint-level. The theoretical analyses, dynamic evaluations or engineering applications on the structural-level are not enough at present stage. Besides, although the significance of slabs on structural behaviors has been valued, the existing researches mainly focus on the cast-in-place monolithic joints or precast wet-joints. Due to the complicated mechanism and multiple forms of precast dry-connections, the evaluations of slab effects on this joint type are necessary. Moreover, the existing researches of precast beam-column-slab dry-connections mainly focus on the two-dimensional plane sub-assembly, while the corresponding researches from the aspect of three-dimensional spatial system are still in great need.

In this paper, the dynamic and probabilistic seismic performance assessments of precast prestressed RCFs are performed, and the slab influence in the overall structural behavior is considered during the process. The three-dimensional spatial model is established to provide the numerical basis, and the slab is modelled through L-section or T-section beam-slab fiber-sections considering the effective width and centroid positions. The adopted model is verified with the experimental data and layered shell model, and the slab influence in hysteresis curves is investigated by parametric study. Then, two groups of precast prestressed RCFs (four models) are well designed to evaluate the slab influence in dynamic responses through seismic excitations, and the modal analysis, roof displacement analysis, maximum and residual drift ratio analysis are conducted for discussion. Moreover, the incremental dynamic analysis and fragility analysis are also conducted to investigate the probabilistic seismic performance of precast prestressed RCFs with or without slabs. Two response indicators, three fractile curves and four limit states are incorporated, and the slab influence in the exceeding probabilities as well as the engineering decision is well compared.

2. Simulation approach incorporating slab influence

2.1. Three-dimensional numerical model

The three-dimensional numerical model of precast prestressed RCFs incorporating the slab influence is established based on the OpenSees software, as presented in Fig. 2. The beam and column components are modelled using nonlinear beam-column elements, which are formed based on the flexibility theory and are inserted with a force-type interpolation function to describe the internal force distribution. The stability of nonlinear beam-column elements and macro-fitness of mechanical behavior have been studied and verified.
by many scholars [29, 30, 31]. Even under the condition of strong nonlinearity, the static balance condition of the element is strictly satisfied, thus the mechanical characteristics of the entire beam and column can be described with fewer elements. The fiber sections are assigned to the nonlinear beam-column elements, and the whole section is divided into the concrete fiber and steel fiber to reflect the uniaxial characteristics of the materials. To be specific, the concrete fiber is defined by Concrete02 material and the steel fiber is defined by Steel02 material. Worth mentioning is the confined effects of stirrups on the concrete compressive strength, and an amplification factor of $k$ can be introduced to enhance the corresponding stress-strain relationship for the concrete fibers within the stirrup ranges, as expressed in Eq. 5. Scott et al. [32] conservatively took the ultimate compressive strain of the concrete in the core area as the value when the first stirrup is broke. In calculation, the ultimate compressive strain of unconfined concrete cover is determined as 0.004, and the ultimate compressive strain of the confined concrete core is calculated according to the Eq. 6.

$$k = 1 + \frac{\rho_s f_{yh}}{f'_c}$$

$$\varepsilon_{\text{max}} = 0.004 + 0.9\rho_s \cdot \frac{f_{yh}}{300}$$

where $\rho_s$ denotes the stirrup ratio in volume, $f_{yh}$ denotes the stirrups yielding strength, $f'_c$ denotes the cylinder compressive strength, and $\varepsilon_{\text{max}}$ denotes the ultimate compressive strain of confined concrete.

To reflect the bending moment-rotation characteristics of beam-column joints, the zero-length elements are adopted in the plane X-Y and plane Y-Z, respectively. The zero-length element defines two nodes at the same coordinates with the unit length of one, and the mechanical properties in different directions can be considered by assigning different constitutive materials. As the structure is excited with loads, the zero-length elements deform with two nodes separating, thus the constitutive materials can be functioned. In this modelling, the hysteresis material is selected in the rotation direction of joints, which can reflect pinching effects, stiffness degradations and damage accumulations under cyclic reloading and unloading. Three input points are required for both positive and negative zones in hysteresis material, and the specific data (i.e., bending moment-rotation relationship) are transformed from the shear stress-strain relationship which is acquired in light of the modified compression field theory and Membrane software [33, 34].

To reflect the gap opening-closing characteristics at the beam ends of the prestressed structures, the ElasticPPGap and Concrete01 materials are used to substitute the Steel02 and Concrete02 materials at the...
plastic hinge length from the beam ends. Both the materials do not consider the force effects in the tensile stage, thus the mechanical properties of single compression with no-tension can be realized when the gap at the beam ends opens. The prestressed tendons are modelled by the truss element, which is a two-node element that only incorporates the axial force-deformation influence. The Steel02 material can be defined with the initial stress and is then assigned to the truss element to reflect the prestress functions. For the core regions of the connections, the equalDOF constraints are adopted to combine the beam-column components together, and the horizontal-vertical constraints are connected to the two nodes of the zerolength element at the joint center, respectively.

To reflect the slab effects on the structural performance, some researchers utilized the layered shell model to prove the ideal results [35, 25], but also a few drawbacks were found such as the complicated modeling process to couple the degrees-of-freedom between layered shell floor elements and beam-column elements. Moreover, the calculation is often prone to non-convergence in the middle or later stages, even for the two-dimensional plane models. For simplification, some researchers utilized the rigid assumptions to consider the slab effects, such as the rigidDiaphragm command in OpenSees to construct a number of multi-point constraints. This command limits the degrees-of-freedom of slave nodes to the master node as if in a rigid plane. This approach reflects the spatial effects of slab in a sense, but also overestimates the structural capacity with idealization. Thus, in this modelling, the T-section and L-section fiber configurations are adopted to reflect the slab influence in the interior joints and exterior joints, respectively. To characterize the effective flange width of slabs, the recommendation criteria provided by GB50010 in Chinese design code of concrete structures are selected [36], as displayed in Tab. 2 and Fig. 3. Meanwhile, the cross-sectional centroid is also determined according to the T-sections and L-sections, and the axes of nonlinear beam-column elements in modelling are set in line with the centroid. The variations of centroid will result in different mechanical behaviors and displacement responses for asymmetric joints or structures subjected to asymmetric force, thus the step is important. The corresponding calculation equation is shown in Eq. 4.

\[
(S_{rec} + S_{sb}) \cdot y_{tol} = S_{rec} \cdot y_{rec} + S_{sb} \cdot y_{sb}
\]

(3)

where \(b_f\) denotes the effective flange width of slabs, \(l_0\) denotes the beam span in calculation, \(s_n\) denotes the beam (rib) net-distance, and \(h_f\) denotes the flange height. \(b\) represents the beam width, \(h_0\) represents the effective section height, and \(h\) represents the total section height. \(S_{rec}\) and \(S_{sb}\) signify the section areas of rectangular beams and flange slabs, respectively. \(y_{rec}\) and \(y_{sb}\) signify the distance from the centroid of rectangular beams and flange slabs to the section bottom, and \(y_{tol}\) signifies the total distance from the centroid of T-sections or L-sections to the section bottom. It is worth noticing that the final \(b_f\) used in modelling is chosen as the minimum value of multiple conditions in Tab. 1.

Figure 3: Strategy for determining the width of T-section and L-section in beam-slab assembly
Table 1: The selection strategy of slab width for different conditions

| Number | Condition                                      | T-section / I-section | L-section |
|--------|-----------------------------------------------|-----------------------|-----------|
| 1      | Using beam span in calculation $l_0$          | $l_0/3$               | $l_0/6$   |
| 2      | Using beam (rib) net-distance $s_n$           | $b + s_n$             | $b + s_n/2$|
| 3      | Using flange height $h_f$                     | $b + 12h_f$           | $b + 5h_f$|

2.2. Model validations

To verify the effectiveness of the proposed numerical model, the static loading data of two precast prestressed beam-slab assemblies (i.e., specimen A2 in Wang et al. [26] and specimen DP1 in Kaya and Arslan [35]) are adopted. Because the joint behaviors can reflect the main characteristics of the precast prestressed system, thus only the joint models are established for validation. The photograph view of specimen A2 and dimension view of specimen DP1 are displayed in Fig. 4, and the specific parameters for material selection in modelling are listed in Tab. 2.

![Photograph and Dimension Views](image_url)

(a) Photograph view of specimen A2 in Wang et al. [26]  (b) Dimension view of specimen DP1 in Kaya and Arslan [35]

Figure 4: Information of specimen A2 and DP1 in model verification

Fig. 5(a) to 5(c) present the comparing results with the experimental data of specimen A2, and Fig. 5(d) to 5(f) present the comparing results with the numerical data by layered shell models of specimen DP1. Fig. 5(a) and 5(d) show the hysteresis curves, Fig. 5(b) and 5(e) show the skeleton curves, and Fig. 5(c) and 5(f) show the energy dissipation tendency. Generally speaking, the overall numerical results by the proposed model indicate ideal agreement with the experimental data and the layered shell model data. The similar trends of bearing capacity, stiffness degeneration, structural ductility and residual deformation can be observed, especially with the experimental data in specimen A2. Take the energy dissipation capacity for example, the total value of the proposed model in specimen A2 reaches 101.24 kN·m after the seventh cycle, and the result of the experimental data is 89.01 kN·m with the relative difference of 12.08 %. As for the maximum capacity, the corresponding values in specimen DP1 are 105.6 kN and 115.7 kN for the proposed model and the layered shell model, and the relative difference is controlled as 8.73 %. Other performance indices have also been summarized in Tab. 3. The comparison with the two specimens
Table 2: The specific parameters and corresponding values for material selection in model verification

| Component Material | Specimen number | Initial prestress (MPa) | Ultimate stress (MPa) |
|---------------------|-----------------|-------------------------|-----------------------|
| Prestressed tendon  | A2              | 730.6                   | 1860.0                |
|                     | DP1             | 1074.0                  | 1790.0                |
| Beam-slab concrete  | A2              | 26.8                    | 0.0025                |
|                     | DP1             | 0.004                   | 8.2                   |

| Component Material | Specimen number | Steel type | Diameter (mm) | Yield stress (MPa) | Tangent slope |
|---------------------|-----------------|------------|---------------|--------------------|---------------|
| Reinforcing steel   | A2              | HRB400     | 6             | 400.0              | 0.01          |
|                     | 8               | 400.0      | 0.01          |
|                     | 12              | 400.0      | 0.01          |
|                     | 22              | 400.0      | 0.01          |
|                     | DP1             | S500B5     | 8             | 585.0              | 0.005         |
|                     | 10              | 585.0      | 0.005         |
|                     | 12              | 585.0      | 0.005         |
|                     | 14              | 585.0      | 0.005         |
|                     | 16              | 585.0      | 0.005         |

indicates the effectiveness of the proposed model in a sense, which lays the foundation for the further dynamic evaluation and probabilistic assessment of the precast prestressed system with slabs in the following sections.

![Hysteresis curve with specimen A2](image1)
![Skeleton curve with specimen A2](image2)
![Energy dissipation with specimen A2](image3)

(a) Hysteresis curve with specimen A2  (b) Skeleton curve with specimen A2  (c) Energy dissipation with specimen A2

![Hysteresis curve with specimen DP1](image4)
![Skeleton curve with specimen DP1](image5)
![Energy dissipation with specimen DP1](image6)

(d) Hysteresis curve with specimen DP1  (e) Skeleton curve with specimen DP1  (f) Energy dissipation with specimen DP1

Figure 5: Numerical verification with experimental data and layered shell model

2.3. Slab influence investigation

In this modelling, the slab strategy and width adoption are the critical factors that can affect the result accuracy. To further analyze the slab influence in the whole hysteresis tendency, a total of eight slab widths of specimen A2 are chosen to perform the parametric comparison (i.e., $d=640$ mm, 840 mm, 1040 mm, 1240 mm, 1440 mm, 1640 mm, 1840 mm and 2040 mm). Among the widths, the value of 1440 mm is calculated according to the slab strategy in this paper for benchmark (Tab. 1), and the other values are determined at an interval of 200 mm for mutual comparison, respectively. To better reflect the hysteresis curves clearly, $d=640$ mm and 840 mm are set as group 1, $d=1040$ mm and 1240 mm are set as group 2, and $d=1640$ mm,
Table 3: The comparison of performance indices between the proposed model and the test data / layered shell model

| Specimen | Source          | Initial stiffness | Bearing capacity | Secant stiffness degradation |
|----------|-----------------|-------------------|------------------|-----------------------------|
|          | Value (10^3kN·m^-1) | Ratio (%)       | Value (10^3kN)   | Ratio (%)                  | Value (%) | Ratio (%) |
| A2       | Test data [26]  | 130.90           | 619.9            | 93.7                       | 2.35      |
|          | Proposed model  | 118.40           | 600.3            | 91.5                       |           |
| DP1      | Layered shell model [35] | 38.11          | 115.7            | 98.6                       | 0.51      |
|          | Proposed model  | 41.58            | 105.6            | 98.1                       |           |

1840 mm and 2040 mm are set as group 3. Fig. 6(a) to 6(c) present the hysteresis curves of the three groups, and Fig. 6(d) to 6(f) present the skeleton curves of the three groups. For each group, the experimental data of specimen A2 and the modelling results by d=1440 mm are incorporated for comparison.

It can be observed that when the flange width of T-section is less than 1440 mm, the initial stiffness and bearing capacity of the model are significantly reduced with the decrease of flange width during the loading process. Meanwhile, the energy consumption mode, self-centering capability and stiffness degradation trend show a distinct difference with the original test data. When the value of d reaches 1440 mm, the modelling results of both hysteresis and skeleton curves are in good agreement with the original test data. When the values of d are greater than 1440 mm, the data of hysteresis and skeleton curves reflect limited changes. Although the structural stiffness and peak capacity increase slightly, the overall energy consumption mode, self-centering behavior, displacement ductility, and stiffness degradation are basically the same from the macro perspective. Only in the later stage of loading process, the skeleton contour and covering area of hysteresis loops present slight enlargement to some extent, with the increase of flange width. The reason is that when the loading protocol comes to the large displacement ratio, most of the materials have entered...
into the plastic developing stage. At this time, the concrete and steel at the slab regions further function to provide substantial negative-direction bending capacity in the structural failure period, which improves the energy dissipation capacity of the joint at the last few hysteresis loops to some extent. The parametric investigations indicate the feasibility of the slab strategy adopted in this paper, and the approach can effectively characterize the hysteretic performance of the relevant specimens in a sense, which provides the model basis for the following structural-level evaluations.

3. Dynamic performance evaluation

3.1. Building information

Two groups of precast prestressed RCFs are designed according to the concrete design code (GB50010) [36] and precast concrete structure specification (JGJ1) [37] in China. The first group represents the frames with five storeys (i.e., 5F) and the second group represents the frames with ten storeys (i.e., 10F), and both the groups include two comparing scenarios with or without slabs, respectively. All the scenarios are designed as spatial frames with three spans in the plane (length of 5 m) and two spans out of the plane (length of 4 m), as presented in Fig. 7. The storey height for the first group is 3.9 m and the storey height for the second group is 3.6 m. The fortification seismic intensity for the above scenarios is selected as seven degrees with the peak ground acceleration (PGA) of 0.1 g in China, which represents a 10% exceeding probability in 50 years and is similar to the conception of design basis earthquake (DBE) in ASCE/SEI 41-06 [28]. The design site condition is adopted as class-II with the shear velocity between 250 m/s and 500 m/s, and the design seismic group is type-I to incorporate the influence of far-/near-field earthquakes. The corresponding characteristic site period \( (T_g) \) for the target acceleration response spectra is adopted as 0.35 s. The dead load for design is calculated through bulk density of reinforced concrete (25 kN/m\(^3\)), and the live load for design is chosen as 2.5 kN/m\(^2\) and 2.0 kN/m\(^2\) for the roof and other storeys, respectively.

Considering the structural height and relevant specification requirements, the concrete grades for the first and second groups are selected as C30 and C40, respectively. The reinforcing steel for the first group is chosen as HRB335 and the corresponding prestressed strands are determined with the ultimate strength of 1720 MPa \( (f_{ptk}) \). Similarly, the reinforcing steel for the second group is chosen as HRB400 and the corresponding prestressed strands are determined with the ultimate strength of 1860 MPa \( (f_{ptk}) \). Worth mentioning is that the above material strength refers to the characteristic value with an 95% guarantee ratio. For both the groups, the slab thickness is uniformly taken as 150 mm. In addition, according to GB50010, when the thickness of the floor slab is 150 mm, the diameter of the steel bars should not be less than 8 mm, the spacing should not be greater than 200 mm, and the reinforcement within the unit width should not be
less than 1/3 of those at the slab bottom in the span middle. Based on the slab strategy as mentioned before (Tab. 1), the slab widths for T-section (i.e., middle span) and L-section (i.e., side span) are given as 1666.7 mm (minimum=\(l_0/3\)) and 833.4 mm (minimum=\(l_0/6\)), respectively. The diameters for steel bars in slabs are chosen as 18 mm with an interval of 200 mm. Fig. 7 presents the detailed cross-sectional information of columns, main beams, secondary beams and slabs for the two design groups.

### 3.2. Modal analysis

Based on the simulation approach adopted in Section 3 and Fig. 2, the four numerical models for the two design groups are established. Tab. 4 lists the natural vibration periods for the first six modes of the four models, in which 5F and 10F represent the models without slabs, while 5F-slab and 10F-slab represent the models with slabs. Comparing the fundamental period \((T_1)\), the varying tendency for the first and second group attains to 0.679\% and 0.162\%, respectively, and the models with slabs show a smaller period value, which is in agreement with the theoretical assumption that the structure with slabs shall have a larger stiffness than that without slabs. Besides, a smaller period commonly corresponds to a larger spectral acceleration in the response spectrum, thus a larger external load shall be considered in the relevant assessment and design. Comparing the other periods \((T_2 \text{ to } T_6)\), the gap difference enlarges from \(T_2\) to \(T_6\), especially for the torsional periods of \(T_3\) and \(T_6\), which indicates that the models with slabs obviously improve the integrated stiffness and reduce the spatial effects than the models without slabs.

The spatial displacement patterns for the first three modes of 5F-slab and 10F-slab are displayed in Fig. 8. It can be seen that the main modes corresponding to the first and second periods are characterized with translational deformation in the \(y\) direction and the \(x\) direction, respectively. The third mode corresponding to the third period appears to be torsional deformation in the \(x-y\) plane. In addition, the shape of the first and second vibration modes exhibits the shear-type deformation features, which satisfies with the mechanical properties in the traditional cast-in-place concrete frame structures. In a sense, the modal results conform to the primary requirements of seismic design in China, and verifies the reliability of the design process and simulation approach. The relevant settings of structural parameters can be used for the subsequent seismic performance analysis in the following subsection.

#### Table 4: The natural vibration periods for the first six modes of the four models

| Model     | \(T_1\) (s) | \(T_2\) (s) | \(T_3\) (s) | \(T_4\) (s) | \(T_5\) (s) | \(T_6\) (s) |
|-----------|-------------|-------------|-------------|-------------|-------------|-------------|
| 5F-slab   | 0.9130      | 0.6950      | 0.5393      | 0.3012      | 0.2591      | 0.2196      |
| 5F        | 0.9192      | 0.7684      | 0.7003      | 0.3949      | 0.3597      | 0.2723      |
| 10F-slab  | 1.7308      | 1.2351      | 0.9694      | 0.5338      | 0.4144      | 0.3191      |
| 10F       | 1.7336      | 1.4109      | 1.2755      | 0.5421      | 0.4512      | 0.4210      |

Figure 8: Building mode decomposition for precast prestressed RCF-5F and RCF-10F with slabs
### 3.3. Ground motion selections

The time history analysis method is a commonly used analysis method in structural assessment, and many seismic codes have proposed that this approach can be used to check the structural dynamic characteristics under the action of an earthquake. According to the seismic design code of buildings in China (GB50011), three or seven sets of ground motions are needed for structural dynamic assessment, and the average values shall be discussed when the seven sets of ground motions are adopted. Thus, in this subsection, seven earthquakes are selected according to the design information of fortification intensity and the corresponding target acceleration response spectra under the maximum considered earthquake (MCE), as presented in Fig. 9(a). Besides, for the probabilistic analyses in the next section, 22 ground motions are selected under the same condition, as presented in Fig. 9(b). The target response spectrum, mean response spectrum and individual response spectrum are depicted in red, blue and gray in Fig. 9, respectively. Worth mentioning is that when selecting ground motions, the difference of spectral acceleration between the target and mean spectrum should not be larger than 20% at the main considered periods. Tab. 5 lists the earthquake information of the selected 22 waves for probabilistic analyses, among which the Nos. 3, 10, 11, 13, 14, 15 and 16 are the selected 7 waves for time history analyses.

| Number | Motion | Year | Station | Intensity | Component |
|--------|--------|------|---------|-----------|------------|
| 1      | Helena-Montana-01 | 1935 | Carroll College | 6 | A-HMC180 |
| 2      | Helena-Montana-02 | 1935 | Helena Fed Bldg | 6 | B-FEB000 |
| 3      | Imperial Valley-02 | 1940 | El Centro Array | 6.5 | I-ELC180 |
| 4      | Northwest Calif-02 | 1941 | Ferndale City Hall | 6.6 | C-FRN045 |
| 5      | Northern Calif-01 | 1941 | Ferndale City Hall | 6.4 | C-FRN225 |
| 6      | Borrego | 1942 | El Centro Array | 6.5 | B-ELC000 |
| 7      | Kern County | 1952 | LA-Hollywood Stor FF | 7.36 | PEL-PEL090 |
| 8      | Kern County | 1952 | Pasadena-CIT Athenaeanum | 7.36 | PAS180 |
| 9      | Kern County | 1952 | Santa Barbara Courthouse | 7.36 | SBA042 |
| 10     | Kern County | 1952 | Taft Lincoln School | 7.36 | TAF202 |
| 11     | Southern Calif | 1952 | San Luis Obispo | 6 | SLO234 |
| 12     | Northern Calif-03 | 1954 | Ferndale City Hall | 6.5 | C-FRN044 |
| 13     | El Alamo | 1956 | El Centro Array | 6.8 | ELC180 |
| 14     | Parkfield | 1966 | Cholame-Shandon Array | 6.19 | C12050 |
| 15     | Parkfield | 1966 | Cholame-Shandon Array | 6.19 | C0589 |
| 16     | Parkfield | 1966 | Cholame-Shandon Array | 6.19 | C0650 |
| 17     | Parkfield | 1966 | San Luis Obispo | 6.19 | SLO234 |
| 18     | Parkfield | 1966 | Tumblor pre-1969 | 6.19 | TMB205 |
| 19     | Borrego Mtn | 1968 | El Centro Array | 6.63 | A-ELC180 |
| 20     | Borrego Mtn | 1968 | LA-Hollywood Stor FF | 6.63 | A-PEL090 |
| 21     | Borrego Mtn | 1968 | LB-Terminal Island | 6.63 | A-TLE219 |
| 22     | Borrego Mtn | 1968 | Pasadena-CIT Athenaeanum | 6.63 | A-PAS180 |

Note: Nos. 3, 10, 11, 13, 14, 15 and 16 are the selected 7 waves for time history analyses.

### 3.4. Roof displacement analysis

Two levels of ground motions are adopted for the roof displacement analysis, which are frequent earthquake (FE) and rare earthquake (RE). FE and RE represent the exceeding probability of 63.2% and 2% for 50 years, and the corresponding PGAs for the seven fortification intensity are adjusted to 35 cm/s\(^2\) and 220 cm/s\(^2\), respectively. Fig. 10 and Fig. 11 display the roof time history displacements under frequent earthquake and rare earthquake for No.3-I-ELC180, No.10-TAF021 and No.11-SLO234, respectively. The red lines represent the models with slabs while the black lines represent the models without slabs.

Generally speaking, the roof displacement of the models without slabs is greater than or close to that of the models with slabs, although there exists reversal phenomenon at some time points. Take the Fig. 10(a) and Fig. 10(d) for example, the maximum roof displacements for the 5F and 10F without slabs are 14.1 mm and 20.8 mm under the FE condition, and the values for the 5F-slab and 10F-slab reduce to 11.7 mm and 16.2 mm, respectively. The corresponding decreasing ratios attain to 19.1% and 22.1% for this ground.
motion under the FE condition, and the average dropping percentages for the seven selected ground motions approximate to 25.9% and 28.6%, respectively. The same tendency can be obtained for the RE condition in Fig. 11. The reason is that the model with slabs commonly possesses a smaller fundamental period, and a smaller period tends to have a larger spectral acceleration since it is in the descending part of the response spectrum. However, the model with slabs commonly possesses a larger stiffness, and the influence of stiffness improvement is distinctly more obvious than the influence of acceleration increment, thus the roof displacement can be well controlled. Besides, the time point for the maximum roof displacement is different between the models with or without slabs, and the displacement amplitude for the model without slabs can be quite strong in a certain interval. Take the Fig. 11(b) and Fig. 11(e) for example, the time points of the maximum roof displacement for the 5F and 10F without slabs are 6.2 s and 18.8 s under the RE condition, and the time points for the 5F-slab and 10F-slab change to 3.8 s and 12.4 s, respectively. This means that the acceleration response process of the structure is not consistent, and ignoring the slab effects may misjudge the time point of the largest structural deformation in the performance evaluation. In addition, residual displacement patterns are different for the models with or without slabs, and the models with slabs generally present a smaller roof residual displacement, which can be observed from the gaps between the final roof displacements of red and black lines in Fig. 10 and Fig. 11. In a sense, the results demonstrate the importance to consider the slab influence in structural performance assessment, and indicate that ignoring the slab effects may overestimate the maximum and residual vibration responses of the overall structure.

3.5. MIDR and RIDR analysis

Fig. 12 and Fig. 13 display the maximum interstory drift ratio (MIDR) and residual interstory drift ratio (RIDR) distributions along storey height under the FE and RE conditions, respectively. According to the seismic design code of buildings in China (GB50011) [38], the MIDR thresholds for RC frames are recommended as 1/550 and 1/50 for FE and RE conditions, and the RIDR thresholds for RC frames can be adopted as 1/2000 and 1/200 for FE and RE conditions.

It can be observed from Fig. 12(a) and Fig. 12(c) that the individual MIDRs of both the 5F and 5F-slab models are smaller than threshold, and the average values show the obvious redundancy, which indicates the design rationality and structural safety of the precast prestressed frames under the FE condition. The same conclusions can be drawn for the 10F and 10F-slab models in Fig. 12(b) and Fig. 12(d), but one ground motion in 10F model shows the MIDR that exceeds the threshold. For the four models in Fig. 13 under the RE conditions, all the individual and average results are within the limitations of 1/50, which demonstrates the satisfactory seismic performance of this structural form. However, the MIDRs of models
Figure 10: Roof time history displacement under frequent earthquake

Figure 11: Roof time history displacement under rare earthquake
without slabs are commonly larger than the models with slabs, and the weak storeys (i.e., MIDR storeys) of the two types are different. To be specific, among the seven selected ground motions for 5F-slab model under the FE condition, two result in the weak storeys at the first floor and five result in the weak storeys at the second floor. In comparison, for the 5F model without slab, three result in the weak storeys at the first floor and four result in the weak storeys at the second floor. The same difference can also be found in 10F and 10F-slab models. The above phenomenon demonstrate that the models with slabs can accurately simulate the rigidity of the whole structure, and ignoring the slab influence in precast prestressed frames may overestimate the structural responses and misestimate the position of the weak storey, which may lead to incorrect judgments in engineering design and applications.

![Figure 12: MIDR and RIDR distributions along storey level under frequent earthquake](image)

Compared with the traditional cast-in-place RCFs, one of the most obvious characteristics in precast prestressed frames is the excellent self-centering capacity and recovering function in the elastic stage. Thus, the RIDR is an important index in performance assessment of this structural type. Worth mentioning is that
Figure 13: MIDR and RIDR distributions along storey level under rare earthquake
RIDR value should be calculated by subtracting the initial drift ratio caused by gravity load and prestress from the final drift ratio. It can be seen from Fig. 12(e) and Fig. 12(g) that the models with slabs commonly reflect a smaller RIDR than the models without slabs, and two conclusions can be acquired correspondingly. First, the structural type is almost non-destructive under the FE condition, and the self-centering efficiency of this structural type is well exerted. Second, the models with slabs commonly have larger structural rigidity and better spatial integrity, thus the plastic development of the whole structure is not as fast as the models without slabs. As a result, the self-centering and recovering capacities of the models with slabs commonly present a better tendency. In addition, from Fig. 12(e) and Fig. 12(f), it can be observed that the individual RIDR results for the models with slabs are obviously less discrete and have smaller deviation than the models without slabs. Moreover, as for the average RIDR distributions under the RE condition in Fig. 13(g) and Fig. 13(h), the maximum RIDRs appear at different weak storeys. To be specific, storey-two is for 5F-slab model, storey-three is for 5F model, storey-one is for 10F-slab model, and storey-three is for 10F model, which denotes that the models with or without slabs may change the results of structural damage assessments and self-centering assessments under the RE condition. The above discussions indicate that the slab influence is important to consider in the simulation of precast prestressed frames and the accuracy of this analyzing approach can be realized with more value for reference.

4. Probabilistic performance evaluation

4.1. Incremental dynamic analysis

The incremental dynamic analysis (IDA) approach is proposed by researchers in 1970s [39], and has gradually developed in terms of its macroscopic and comprehensive advantages in studying the dynamic performance of novel structural types. Up to date, the IDA approach has been used as a method to evaluate the overall collapse resistance of structures by FEMA356 [40], and has been applied into practical projects broadly to prevent serious damage in weak structural components. The main principle of IDA approach is to continuously adjust the ground motions to different intensity levels, and during the process the intensity measure (IM) is used as reference for evaluation. Then a series of time history analyses can be performed to analyze the structural responses, and the mechanical development from linear elasticity to plasticity and even collapse can be well reflected and captured [41]. During the process the engineering demand parameter (EDP) is used to characterize the structural performance. The IDA curves can be then depicted by matching and connecting the IM-EDP pairs, as shown in Fig. 14 and Fig. 15. Besides, in order to consider the data discreteness and to effectively predict the statistical tendency, fractile curves (e.g., 16%, 50% and 84%) are also commonly depicted, as presented in blue, black and red in Fig. 14 and Fig. 15.

The commonly used IM indices include the PGA and the spectral acceleration under the fundamental period \( S_a(T_1) \), and according to the comparison analysis of the two IMs in Vamvatsikos et al. [42], the \( S_a(T_1) \) presents less dispersion than PGA under the same conditions. Thus, the \( S_a(T_1) \) is chosen as the IM in this paper. As for the EDP indices, multiple selections can be adopted such as peak displacement and peak shear force. Considering the mechanical properties and self-centering functions of the precast prestressed frames, the MIDR and RIDR indices are adopted in this paper for assessment. In addition, limit states should be defined in the IDA approach based on the selected EDP indices, which are also required in the subsequent fragility analyses. At present there is no comprehensive and unified standard for the specific description of each state and the determination of indices. Thus, combining the recommendations in FEMA356 [40] and GB50011 [38], four limit states are determined to reflect the damage development in this paper, and the corresponding thresholds for MIDR and RIDR are listed in Tab. 6.

Fig. 14 presents the IDA curves and fractile curves of the four models using the EDP of MIDR. Fig. 14(a) and Fig. 14(d) reflect the results of 5F and 10F without slabs, and Fig. 14(b) and Fig. 14(e) reflect the results of 5F and 10F with slabs. Fig. 14(c) and Fig. 14(f) are the fractile comparison of the two conditions. Generally speaking, the IDA curves are more concentrated at the early stage for the models with slabs, and the corresponding \( S_a(T_1) \) under the same limit states are obviously larger than the results of models without slabs. In the later stage of curve development, the models without slabs show the serious irregularity than the models with slabs, and there exists the dropping tendency in MIDR during the continuous increment of
Table 6: The four limit states to reflect the damage development and the corresponding thresholds for MIDR and RIDR

| Number | Limit state          | MIDR threshold | RIDR threshold |
|--------|----------------------|----------------|----------------|
| 1      | Normal Operations (NO) | 1/550          | 1/2000         |
| 2      | Immediate Occupancy (IO) | 1/100          | 1/1000         |
| 3      | Life Safety (LS)      | 1/50           | 1/200          |
| 4      | Collapse Prevention (CP) | 1/25           | 1/100          |

IM values. This phenomenon supports the conclusion that the models with slabs possess higher calculation reliability and analysis accuracy to some extent. The similar conclusions can be drawn from the comparison of fractile curves. For the four selected limit stats (i.e., NO, IO, LS and CP), the models with slabs require larger $S_a(T_1)$ than the models without slabs, which means that the models with slabs can undertake larger seismic intensity corresponding to the same MIDR thresholds and indicate better seismic performance under the same intensity level. Take the LS states in model 5F and 5F-slab for instance, the $S_a(T_1)$ of 16%, 50% and 84% fractile curves reaches 0.78g, 1.22g and 1.72g in model 5F-slab, respectively, and the $S_a(T_1)$ in model 5F reaches 0.54g, 1.02g and 1.51g, respectively. The increasing percentages are 44.4%, 19.6% and 13.9%, respectively, and the average improving ratio attains to 23.8% among all the limit states and fractile conditions. However, in the later stage after the CP limit state, the results of the two fractile curves intersect and the models without slabs show larger $S_a(T_1)$ comparatively. It reflects that the models with slabs degenerate more seriously and deteriorate more rapidly when entering into the plastic stage. In a sense, ignoring the slab influence will overestimate the seismic performance of structures in the plastic stage, and it is necessary to adopt the models with slabs for performance evaluation.

Fig. 15 presents the IDA curves and fractile curves of the four models using the EDP of RIDR. In contrast
to the results from the EDP of MIDR, the fractile curves of model 5F without slabs are almost above the curves of model 5F with slabs during the whole IDA process (Fig. 15(c)). For the 50% fractile curve, the development of RIDR accelerates sharply when the $S_a(T_1)$ is over 3.0g. Although the difference between the 10F and 10F-slab is smaller in Fig. 15(f), the results of 10F commonly denote larger $S_a(T_1)$ at the main considered RIDRs and limit states. For the 50% fractile curve, the development of RIDR accelerates sharply when the $S_a(T_1)$ is over 0.7g. The curve trends indicate that different EDPs may result in the variability of results in the IDA approach, and selecting multiple EDPs for comprehensive comparison is significant. In addition, the factors of slabs may obviously affect the self-centering function and the recovering capacity of the considered RIDRs and limit states. For the 50% fractile curve, the development of RIDR accelerates sharply when the $S_a(T_1)$ is over 0.7g. The curve trends indicate that different EDPs may result in the variability of results in the IDA approach, and selecting multiple EDPs for comprehensive comparison is significant. In addition, the factors of slabs may obviously affect the self-centering function and the recovering capacity of the precast prestressed frames by changing the residual deformation. The models without slabs show better seismic performance than the models with slabs in terms of the RIDR indicator, which is mainly related to the restraining effects of the slabs on the prestress transferring and RIDR decreasing to some extent. Thus, incorporating the slab influence in structural assessment is important and can lead to different engineering decisions in actual projects.

Figure 15: IDA and fractile curves for all conditions with threshold $\theta_{res}$

4.2. Fragility analysis

Seismic fragility analysis aims to give the conditional probability of structures to exceed a certain limit state under the earthquake hazards. Compared with the empirical expression acquired from the history records, the analytical expression by fragility curve through numerical data is a more efficient way [43, 44]. In this paper, the data from IDA approach are used to perform the fragility analysis. In addition, two indicators are needed in the analytical fragility, which are the seismic demand and structural capacity. Based on the assumptions that the two indicators are in line with the lognormal distributions, the seismic fragility can be expressed in the reliability way as Eq. 4:

$$P_f[Demand > Capacity|IM] = \Phi \left[ \ln\left(S_d/IM/S_c\right)/\sqrt{\beta^2_{d/IM} + \beta^2_c} \right]$$

(4)
Moreover, incorporating the slab influence in fragility assessment can affect the variability of fragility results, and selecting multiple EDPs for comprehensive comparison is significant. At the initial growing stage of the LS and CP states, the models with slabs are commonly on the right of the models without slabs under the same conditions. For the four limit states, the corresponding exceeding probability of LS limit state under the same exceeding probability. In a sense, if the slab influence is neglected in the linear regression (Eq. 6), \(\beta_\text{IM} \) is the logarithmic standard deviation for demand and can also be acquired from the logarithmic regression of EDP-IM pairs in the logarithmic scale. All the four models with or without slabs and two EDPs by MIDR or RIDR are well depicted. The black scattered point represents the individual seismic result and the red line represents the regression result. Fig. 17 displays the seismic fragility curves for all conditions.

For the EDP of MIDR (Fig. 17(a) and Fig. 17(b)), the fragility curves move towards right from the NO to CP states, and the models with slabs are commonly on the right of the models without slabs under the same conditions. For the four limit states, the corresponding \(S_a(T_1)\) that approaches to the 100% exceeding probability is 0.4g, 1.3g, 2.2g and 4.0g for model 5F-slab, and 0.2g, 0.8g, 1.4g and 2.4g for model 10F-slab. Take the 10F and 10F-slab for example, the corresponding exceeding probability of IO limit state under the \(S_a(T_1)\) is 0.25g is 52.7% for model 10F, while the value drops to 35.4% for model 10F-slab. The corresponding exceeding probability of LS limit state under the \(S_a(T_1)\) is 0.5g is 59.6% for model 10F, while the value drops to 50.3% for model 10F-slab. In another word, through considering the influence of slabs in precast prestressed frames, the fragility curves show smaller exceeding probability under the same \(S_a(T_1)\) and larger \(S_a(T_1)\) under the same exceeding probability. In a sense, if the slab influence is neglected in the analysis, the structural capacity and seismic performance may be underestimated.

For the EDP of RIDR (Fig. 17(c) and Fig. 17(d)), the same curve tendency is observed from NO to CP states, but the positions of fragility curves between the two models with and without slabs show difference. At the initial growing stage of \(S_a(T_1)\), the exceeding probability of the models without slabs is slightly greater than than that of the models with slabs, especially for the NO and IO performance levels. This phenomenon is same as the results by the EDP of MIDR and indicates that under the excitation of small earthquakes, the stiffness contribution of slabs to the whole structure makes the components less likely to enter into the plastic range, so the results present slightly better self-centering capacity and stronger structural capacity. As the value of \(S_a(T_1)\) becomes larger, this trend is reversed. The exceeding probability of the models without slabs is obviously smaller than that of the models with slabs, especially for the LS and CP performance levels, and the maximum gap difference reaches about 35% under the CP state. The reason for this phenomenon is that when the EDP index is selected as RIDR, it reflects the self-centering capability of the whole structure to a certain extent. Under the large \(S_a(T_1)\) value, the entire structural materials have entered into plasticity, and the axial force of the prestressed tendons has also reached the limit state. Under the same prestressed forces, the larger sectional areas lead to smaller stress level, thus the RIDR indicator presents more obvious tendency. As a result, the slabs restrain the prestress transferring and limit the self-centering capability under the large \(S_a(T_1)\) condition relatively. The curve trends indicate that different EDPs may result in the variability of fragility results, and selecting multiple EDPs for comprehensive comparison is significant. Moreover, incorporating the slab influence in fragility assessment can affect the probability tendency and change the engineering decision especially for the LS and CP performance levels.
Figure 16: Linear regression for IM-DM pairs in logarithmic scale
Figure 17: Seismic fragility curves for all conditions
5. Conclusions

In this paper, the dynamic and probabilistic seismic performance assessments of precast prestressed RCFs are performed, and the slab influence in the overall structural behavior is considered through three-dimensional spatial model. The conclusions may be drawn as follows:

(1) The three-dimensional numerical model incorporating the slab influence is established based on the OpenSees software. To reflect the bending moment-rotation characteristics of beam-column joints, the zero-length elements are adopted in the plane X-Y and plane Y-Z, respectively. To reflect the gap opening-closing characteristics at the beam ends of the prestressed structures, the ElasticPipel and Concrete01 materials are used to at the plastic hinge length from the beam ends. The slab is modelled through L-section or T-section beam-slab fiber-sections considering the effective width and centroid positions, and the adopted model is verified with the experimental data and layered shell model, indicating the similar tendency in bearing capacity, stiffness degeneration, structural ductility and residual deformation. The comparison indicates the effectiveness of the proposed model in a sense, and lays the numerical basis for the dynamic and probabilistic assessment of the precast prestressed system with slabs.

(2) Two groups of precast prestressed RCFs are designed to evaluate the slab influence in dynamic responses through 7 seismic excitations. The modal analysis shows smaller vibration periods for models with slabs, which is in agreement with the theoretical assumption that the structures with slabs shall have larger stiffness than those without slabs. Beside, the roof displacement analysis reflects that the deformation of models without slabs is greater than that of the models with slabs, although there exists reversal phenomenon at some time points. The time point for the maximum roof displacement is different between the models with or without slabs, and ignoring the slab effects may misjudge the time point of the largest deformation in the performance evaluation. Moreover, in the MIDR and RIDR analyses, the plastic development of the models with slabs is not as fast as the models without slabs, and as a result the self-centering capacities of the models with slabs commonly present better tendency and smaller deviation. The positions of weak storeys also change after incorporating the slab influence, especially under the RE condition.

(3) The incremental dynamic analysis and fragility analysis are conducted to investigate the probabilistic seismic performance of precast prestressed RCFs with or without slabs through 22 seismic excitations. Two response indicators, three fractile curves and four limit states are incorporated, and the slab influence in the exceeding probabilities as well as the engineering decision is well compared. As for the EDP of MIDR, the models with slabs require larger $S_a(T_1)$ than the models without slabs, which means that the models with slabs can undertake larger seismic intensity corresponding to the same MIDR thresholds and indicate better seismic performance under the same intensity level. However, in the later stage after the CP limit state, the curves of the two conditions intersect and the models without slabs show larger $S_a(T_1)$ comparatively. It reflects that the models with slabs degenerate more seriously and deteriorate more rapidly when entering into the plastic stage. Besides, the fragility curves move towards right from the NO to CP states, and the models with slabs are commonly on the right of the models without slabs under the same conditions, indicating that the fragility curves show smaller exceeding probability under the same $S_a(T_1)$ and larger $S_a(T_1)$ under the same exceeding probability after considering the slab influence.

(4) As for the EDP of RIDR, this trend is reversed. The fractile curves of models without slabs are almost above the curves of models with slabs, and the models without slabs show better seismic performance than the models with slabs in terms of the RIDR indicator, which is mainly related to the restraining effects of slabs on the prestress transferring to some extent. Under the FE earthquakes for the NO and IO performance levels, the exceeding probability of the models without slabs is slightly greater than that of the models with slabs, which is in line with the conclusion through the EDP of MIDR. However, under the RE earthquakes for the LS and CP performance levels, the exceeding probability of the models without slabs is obviously smaller than that of the models with slabs. The curve trends indicate that selecting multiple EDPs for comprehensive comparison is important in the variability of IDA and fragility results, and incorporating the slab influence in structural assessment can affect the performance tendency and change the engineering decision. In a sense, the research proves the significance of slabs in the seismic performance of dry-connected precast prestressed RCFs, and provides the reference for the further explorations of slab factors in precast concrete structures.
Declarations

We declare that we have no financial and personal relationships with other people or organizations that can inappropriately influence our work.

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