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Numerical analysis on seismic performance of precast concrete frame structure with intermediate-connected columns

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Abstract

Owing to a variety of advantages including high construction efficiency, environmental protection, short construction period and so on, precast concrete structure has been a major form of construction industrialization currently and will be the trend of building structures in the future in China. The improvement of the seismic performance of precast concrete structure is always difficult to study, and the quality of connection is one of the vital factors affecting the seismic performance of precast concrete structure. Compared to the bottom-connected columns, the intermediate-connected columns are more in line with the principle of the "strong column weak beam" mechanism. Therefore, based on the concept and method of seismic design suitable for China, seismic performance of the precast concrete structure with intermediate-connected columns (ICC) is researched in this study. The seismic performance of the precast concrete structure using ICC is improved compared with that using conventional bottom-connected columns. Firstly, based on finite element software SeismoStruct, a prefabricated column using half grout-filled sleeve connection and a prefabricated frame beam-column joint connected using grouted sleeves are analyzed, the accuracy and effectiveness of the software are verified by comparing the simulated results with experimental results of previous study, and the rationality of the modified Menegotto-Pinto steel constitutive model for simulating the half grout-filled sleeve connection is verified. Secondly, based on the above numerical simulation methods and assembly connection forms, two 3-span 6-story prefabricated frames model are designed, which the assembly connection points are placed in the end of the columns and in the middle of the columns respectively. Thirdly, according to the seismic collapse fragility analysis method of the incremental dynamic analysis(IDA) , through selecting appropriate data of the ground motion records, indicators of the ground motion strength and the structural damage, and structural collapse criteria, the IDA curve of the two prefabricated frame models is obtained and the collapse-resistant capacities of different frames are quantitatively evaluated and compared. It can be found that the precast concrete structure with ICC is better than that of the bottom-connected columns in collapse resistance performance. Finally, the elastic and elastoplastic time history analysis of the two prefabricated frame models is carried out by inputting three frequent-level ground motions and three rare-level ground motions, respectively. The maximum lateral displacement and drift angle of two prefabricated frame models are analyzed and compared. It can be seen that ICC can reduce the displacement response and enhance the lateral stiffness of the structure for the low-rise & multi-storey precast frames under earthquakes. However, a prefabricated column using half grout-filled sleeve connection will increase the number of connections and the difficulty of construction, the assembly connection forms and the assembly connection methods of beam-column joint still need to be studied.

Keywords: precast concrete frame structure; intermediate-connected in columns; numerical analysis; half grout-filled sleeve connection; seismic performance

1. Introduction

Industrialization is the direction of construction industry's development, thus the development of new prefabricated structural connection is an important measure to promote it. The maximum bending moment of the columns of the low-rise & multi-storey precast frames under earthquakes usually appear at the end of the column, meanwhile the inflection point often appears in the middle of the column, where the bending moment is small as well as the height about 1.5 meters is more convenient for construction. Thus in this paper, the assembly connection point has been moved to the central of the column where the bending moment is comparatively much lower under earthquakes. It is expected that the performance of the intermediate-connected columns is better than that of the bottom-connected columns.

Grout-filled sleeve connection is the most widely used technology for bar-splicing, and the numerical simulation of grout-filled sleeve connection is well investigated in previous studies [1-5]. In this study, two completed quasi-static experiments are modelled through SeismoStruct, and the accuracy are validated. Then the original model is chosen from a frame of Tsinghua University's experiment[13]. Simultaneously, based on the above numerical methods and assembly connection forms, two 3-span 6-story prefabricated frames model are designed, which the assembly connection points are placed in the end of the columns and in the middle of the columns respectively. Finally, the collapse-resistant capacities and the displacement response of different frames are quantitatively evaluated and compared, which provide a theoretical reference for the seismic performance of the precast concrete structure with ICC.

2. Numerical simulation method

2.1 Constitutive model

2.1.1 Concrete

The Mander nonlinear concrete constitutive model[6] is applied to the concrete in the cylinder model, which is a constitutive model of confined concrete under uniaxial cyclic load. As shown in Fig. 1, the transverse rebar, which has a certain constraint effect on the concrete, is also considered in the entire stress-strain range.

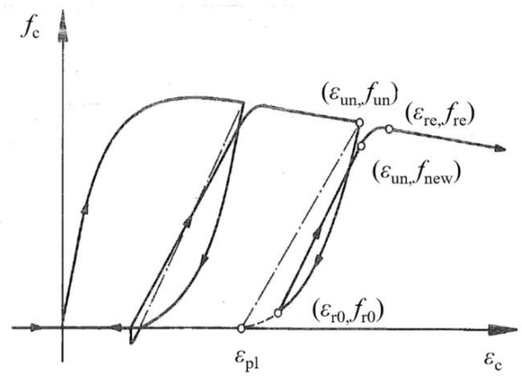

Fig. 1 – Unload and reload curve of concrete

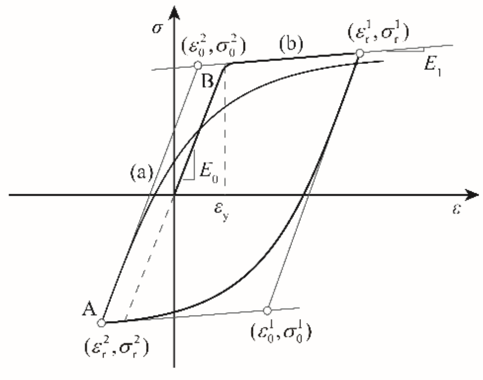

Fig. 2 – Menegotto-Pinto steel model

2.1.2 Reinforcement

The Menegotto-Pinto steel constitutive model[7] is adopted in the reinforcement, which is a uniaxial steel model with the greatest advantage of high numerical calculation efficiency and accuracy, and is suitable for the reinforcement materials in reinforced concrete members subjected to reciprocating load in complex working conditions. The constitutive stress-strain curve proposed by Menegotto-Pinto is shown in Fig. 2.

2.1.3 The grout-filled sleeve

A simplified simulation method for half grout-filled sleeve connection is proposed by Liu Liping[8], which considers the half grout-filled sleeve connection as a section of "equivalent steel bar", just as the steel bars on both sides of half grout-filled sleeve connection and its connection are equivalent to a steel bar with slightly increased strength, decreased stiffness and unchanged diameter within a certain length. The value rules of strength and stiffness are given in Eqs. (1) and (2). Meanwhile, in many grout-filled sleeve connection component models based on fiber element analysis[9], the ideas and values of the simplified treatment of fully grout-filled sleeve simulation are consistent with Liu Liping's research ideas. Therefore, the simplified simulation method and numerical correlation are adopted in the numerical simulation of half grout-filled sleeve connection and fully grout-filled sleeve connection in this paper, and the Menegotto-Pinto steel constitutive model is formelted as follows.

Yield stress of the half grout-filled sleeve connection

$$\sigma_A = 0.940 f_y + 43.375 \tag{1}$$

Where σ_A is the stress corresponding to the yield point of the modified Menegotto-Pinto steel constitutive model, f_y is the stress corresponding to the yield point of the longitudinal bar connecting the sleeve.

Yield strain of the half grout-filled sleeve connection

$$\varepsilon_A = -0.369\varepsilon_y + 6.993 \times 10^{-8} L + 0.006 \tag{2}$$

Where ε_A is the strain corresponding to the yield point of the modified Menegotto-Pinto steel constitutive model, ε_y is the strain corresponding to the yield point of the longitudinal bar connecting the sleeve, L is the length of grout-filled section.

2.2 Examples of components

2.2.1 The prefabricated columns with half grout-filled sleeve connection

The prefabricated columns YZZ2-1 with half grout-filled sleeve connection[10] is composed of ground beam, upper column and post-cast part. After the ground beam and prefabricated column are poured and maintained to meet the requirements of intensity, the grouting material is injected into the sleeve. The column height is 1220 mm, the section size is 350 mm×350 mm, and 8Φ18 vertical steel bars and Φ8@100 stirrups are provided. A constant vertical pressure of 265.92 kN is applied on the top of the column, the lateral loading point is located at the height of the column 1025 mm, the specific loading steps are described in the Code for seismic test methods of buildings [11].

2.2.2 The prefabricated frame joints with grout-filled sleeve connection

The prefabricated frame joint specimen P2[12] is composed of the lower prefabricated column, the upper prefabricated column, the two side prefabricated beams and the cast-in-situ parts. The height of the upper and lower prefabricated columns is 880 mm, and the section size is 300 mm×300 mm, equipped with 8Φ14 vertical steel bars and Φ8@100 stirrups. Eight longitudinal steel bars of the upper prefabricated column are arranged in the through-length. The half grout-filled sleeve is connected at the bottom of the column, and it is embedded at the bottom of the upper prefabricated column after pouring concrete. Eight longitudinal bars of the lower prefabricated column protrude from the top of the column, and the protruding length is 517 mm, which acts like "inserting bars". The length of prefabricated beam is 1000 mm, and the section size is 200 mm×350 mm, equipped with 6Φ12 longitudinal steel bars and Φ8@110 stirrups. 6 longitudinal steel bars are connected to the core area of precast joints through fully grout-filled sleeve. A post-cast section with a length of 500 mm is set at the beam end. The precast columns, beams and core areas of precast joints are connected by pouring concrete on site, and the loading method without considering $P-\Delta$ effect is adopted in the frame joint experiment.

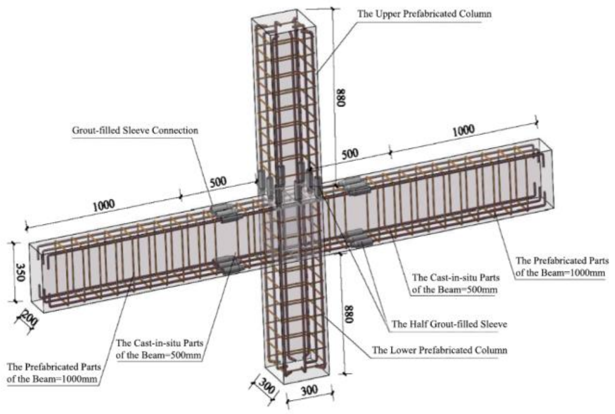

Fig. 3(a) Size of specimens P2

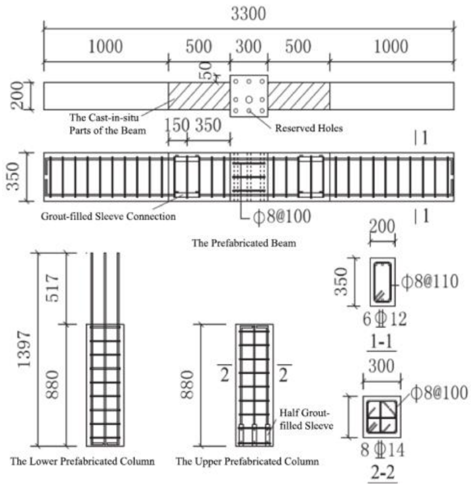

Fig. 3(b) Structure and reinforcement of specimens P2

Fig. 3 – Dimensions and reinforcement of specimens P2

2.3 The validation of simulated methods

The simulated and experimental hysteretic curves of the prefabricated columns YZZ2-1 with half grout-filled sleeve connection are shown in Fig. 4 below. In the first three stages of loading, the peak bearing capacity of the simulated results is slightly higher than that of the experimental results. In the second cycle of the last stage of loading, the experimental results show that the components have failed, so they are not included in the comparison. The results show that the relationship between force and displacement of the prefabricated columns with half grout-filled sleeve connection can be accurately simulated.

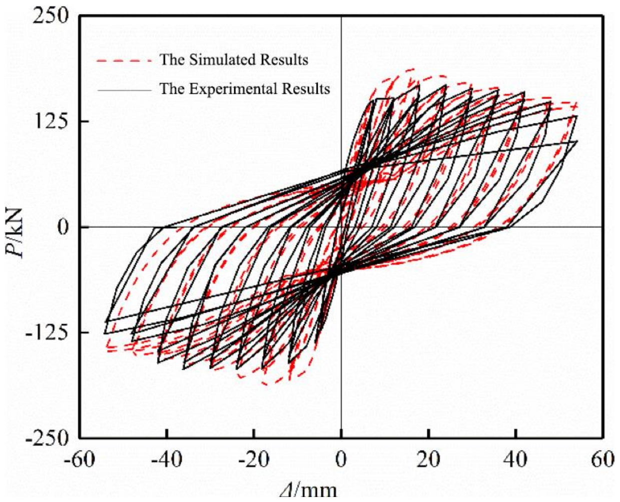

Fig. 4 – Comparison of simulated and measured
hysteretic curves of specimens YZZ2-1

The simulated and experimental hysteretic curves of the prefabricated frame joints considering the influence of joint area are shown in Fig. 5(a) below. The bearing capacity of the simulated value is lower than that of the experimental value in the early stage of loading, and it fits well in the later stage. Both the loading stiffness and unloading stiffness of the simulated value fit well with the experimental value, but the platform segment with reduced pinch phenomenon doesn't fit well due to the limitations of the Takeda model. The simulated and experimental hysteretic curves of the prefabricated frame joints without considering the influence of joint area are shown in Fig. 5(b) below. The overall bearing capacity is well fitted, the stiffness degradation of the simulated value in the early stage is faster than that of the experimental value, and the stiffness degradation in the later stage is better fitted. However, the shear failure and the bond slip of reinforcement in the joint area are not taken into account in the simulation, so the fitting of the reduced pinch phenomenon is poor. On the whole, both of them can well simulate the response of prefabricated frame joints under low reversed cyclic load. Thus, the following numerical simulation method without considering the influence of joint area is adopted to simulate the joint in the frame structure.

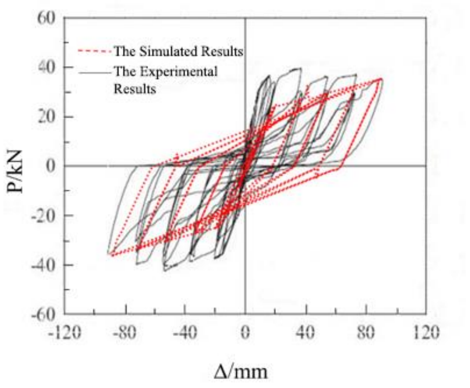

Fig. 5(a) Hysteretic curve of frame joints considering the influence of joint area

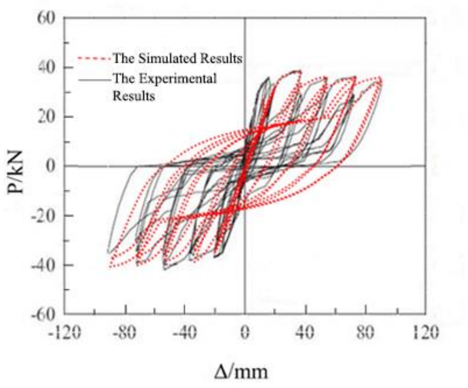

Fig. 5(b) Hysteretic curve of frame joints without considering the influence of joint area

Fig. 5 – Comparison of simulated and measured hysteretic curves of specimens P2

3. Finite element model

3.1 Overview of model prototype

In this paper, two 3-span 6-story prefabricated frames model in the completed frame quasi-static experiment[13] are selected as the original model to study the structural response and collapse fragility analysis of the prefabricated frame with different prefabricated joints under earthquake action. Its dimensions and reinforcement are shown in Fig. 6 below.

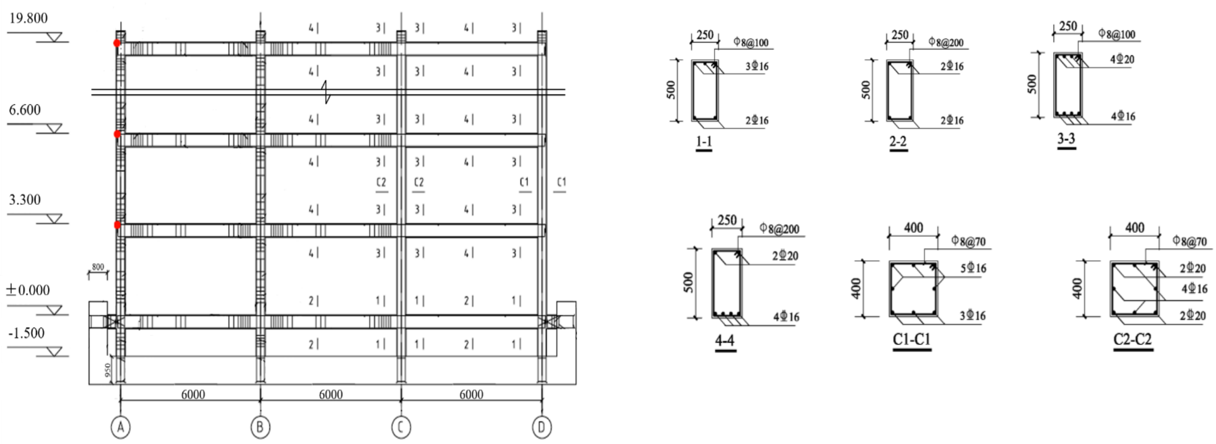

Fig. 6 – Dimensions and reinforcement of frame

The experimental seismic fortification intensity is 7, the site category is II, the framework seismic grade is 3, the floor constant load and live load are 4.6 kN/m^2 and 2 kN/m^2, respectively[14].

Concrete strength grade of beam and column is C30, meanwhile, type HRB335 and HPB235 rebars are adopted respectively for longitudinal reinforcement and stirrup. The material parameters measured by the material test are shown in Table 1 below.

Table 1 – Mechanical properties of frame

Measured concrete strength f_{cu}/Mpa				The yield strength f_y/Mpa	Modulus of elasticity E/Mpa
Pull beam layer	the first floor	the second floor	the third floor		
31.8	36.2	34.7	33.6	481	265433

3.2 Model design

Based on the above experimental frame model, two prefabricated frames model with different assembly connection points are designed, and their geometric dimensions and reinforcement are the same as the original model.

Model KJ-1 is based on the original experimental frame model, which transform cast-in-site frame column into prefabricated column. The assembly connection form of it is half grout-filled sleeve connection, and the assembly connection point is in the bottom of the column. Accordingly, all of the cast-in-place beam column points are transformed into prefabricated frame joints, the assembly connection form is fully grout-filled sleeve connection, and the assembly connection point is on the beam end.

The difference between model KJ-2 and model KJ-1 is the locations of the assembly connections (see Fig. 7).

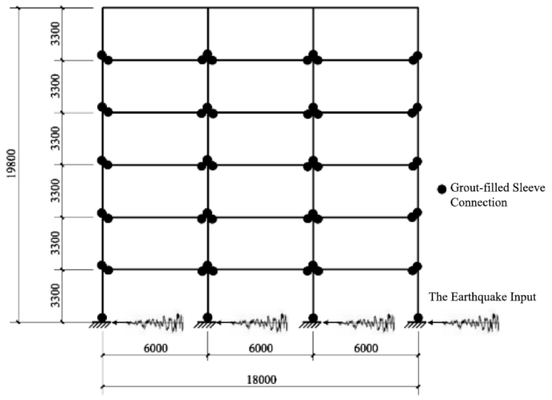

Fig. 7(a) the precast concrete structure KJ-1 with bottom-connected columns

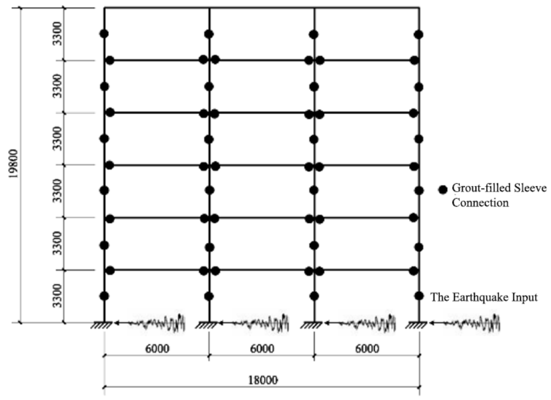

Fig. 7(b) the precast concrete structure KJ-2 with intermediate-connected columns

Fig. 7 – The elevation of the fabricated RC frame structure

The beam section and column section are defined according to the section and reinforcement information of the frame structure given by the frame experiment, and the influence of floor plate as flange is ignored in the model design.

3.3 Load input

Floor constant load and live load are 4.6 kN/m^2 and 2 kN/m^2 respectively. The ground motion records are defined as the working condition of the earthquake, which is applied to column bottom, meanwhile the amplitude modulation coefficient is used to scale the ground motion intensity of the selected ground motion records from a initial value, which keeps the frame model under the effect of the intensity of earthquake is in low elastic response status, then, amplitude modulation coefficient will be adjusted so that the ground motion intensity can be gradually enlarged until the model under a certain level of intensity of earthquake collapses.

4. Numerical simulation analysis

4.1 Overview of collapse vulnerability analysis

Seismic vulnerability is the conditional probability that the structure reaches or exceeds a certain limit state under the preset ground motion intensity. The probabilistic relationship between the seismic collapse-resistant capacities and ground motion intensity index established by the seismic vulnerability function can better reflect the seismic performance of the structure.

In this paper, the collapse probability of frame structures with different assembly connection points under different seismic intensities recorded by multiple seismic records is investigated by using incremental dynamic analysis method. A certain amount of seismic records are input into the frame models, then the seismic response of the whole process from low elastic level to collapse failure are recorded, and the vulnerability curve of the structure is calculated based on the simulated results.

4.2 Ground motion record selection

It is very important to select a certain number of seismic records that can reflect the ground motion characteristics of the target structure. In this paper, the method based on design response spectrum is adopted to select ground motion records, and the dual-frequency control method[15] is adopted and then 11 diverse ground motion records with magnitude greater than 6.3 were selected from the PEER seismic record database, as shown in Table 2 below.

Table 2 – Ground motion records

number	ground motion records	magnitude	number	ground motion records	magnitude
1	Imperial Valley	6.5	7	Mexico City Aftershock	7.8
2	Northridge	6.7	8	Friuli	6.5
3	Borrego Mtn	6.6	9	San Fernando	6.6
4	Campano-Lucano	6.8	10	Uttarkashi	7.0
5	Weber	6.3	11	Valpariso	7.2
6	Lima	7.6			

The dynamic amplification coefficient spectrum of acceleration with a damping ratio of 5% and the response spectrum of Chinese design are shown in Fig. 8 below. It is easily seen that the curves obtained by averaging the acceleration dynamic amplification coefficient spectrum curves of 11 ground motions are in good agreement with the response spectrum of Chinese design code [14].

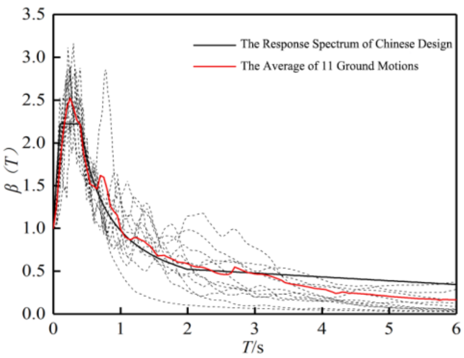

Fig. 8 – Acceleration dynamic magnification factor of selected earthquake records

4.3 Intensity index

With regard to the analysis of structural vulnerability, the ground motion intensity measure (*IM*) should comprehensively reflect the magnitude of the ground motion intensity. Based on *IM*, the original ground motion records can be normalized and amplified step by step. It found that the seismic response of structures has better correlation with amplitude modulated multiples when the first period spectral acceleration index $S_a(T_1)$ was selected as *IM*, and the selected ground motion records were normalized based on the first period spectral acceleration index. According to the equivalent incremental step size, the initial $S_a(T_1)$=0.1g is enlarged step by step, and then manually reduce the incremental step size in the case of poor convergence.

4.4 Damage index

The structural damage index *DM* can reflect the level of structural performance in the analysis of structural vulnerability. The research object of this paper is the collapse vulnerability analysis of the prefabricated frame with different connection points. Therefore, the maximum inter-layer displacement Angle is chosen as the structural damage index *DM*.

4.5 Criteria for the determination of structural collapse

In this paper, collapse determination criteria based on IDA method[16] are adopted to determine structural collapse. Structural collapse is defined as the lateral dynamic instability of the structure, which is considered as the overall collapse failure of the structure when the flat section of IDA curve occurs.

4.6 Collapse vulnerability analysis results

IDA analysis is performed on two prefabricated frame models, KJ-1 and KJ-2. The above selected structural damage indexes are taken as the horizontal coordinate and ground motions intensity indexes as the vertical coordinate. Combined with the above collapse determination criteria, IDA curves of the two models recorded in 11 ground motions are obtained. Due to the limited space in this paper, IDA curves of the five ground motions recorded from No.1 to No.5 are listed here, as shown in Fig. 9 below.

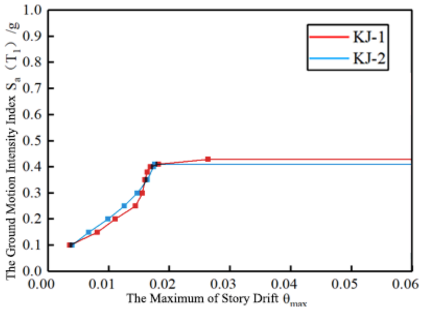

Fig. 9(a) Imperial valley KJ-1 and KJ-2

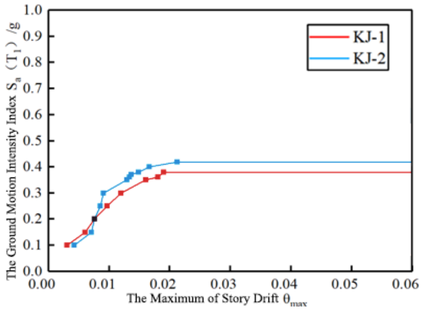

Fig. 9(b) Northridge KJ-1 and KJ-2

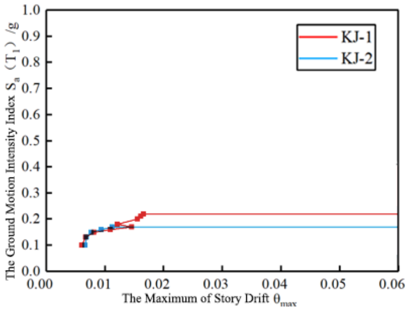

Fig. 9(c) Borrego Mtn KJ-1 and KJ-2

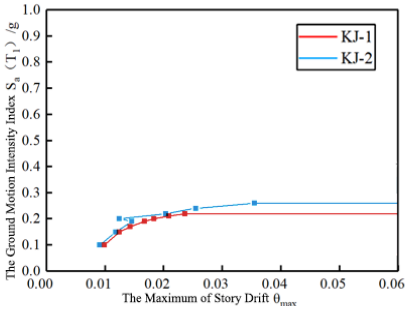

Fig. 9(d) Campano-Lucano KJ-1 and KJ-2

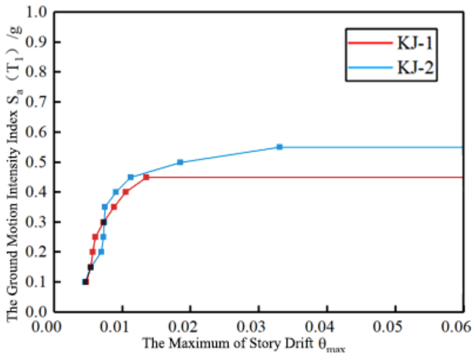

Fig. 9(e) Weber KJ-1 and KJ-2

Fig. 9 – IDA curve of different models

It can be seen that all IDA curves recorded by the two models in 11 ground motions show straight segments, which means the whole structures are collapsed.

Meanwhile, the collapse vulnerability curves of the two frame models are compared, as shown in Fig. 10.

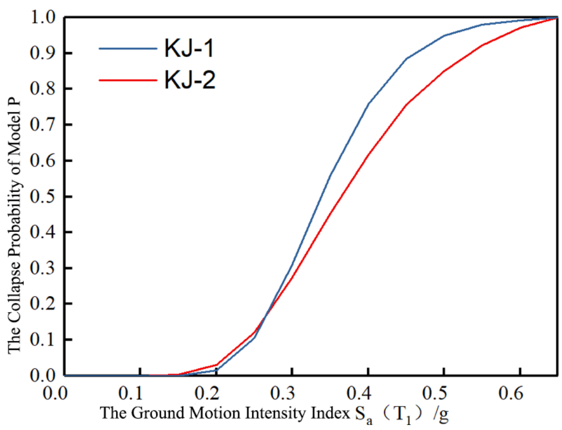

Fig. 10 – Comparison of collapse fragility curves

When the ground motion intensity index $S_a(T_1)$ is less than 0.28g, the collapse probability of model KJ-1 is similar to that of model KJ-2. With the increase of $S_a(T_1)$, the collapse probability of model KJ-1 is higher than that of model KJ-2, which is 18.89% higher than that of model KJ-2 at the maximum difference value, it indicates that the seismic collapse-resistant capacities of prefabricated structures can be effectively enhanced by changing joints from the bottom of the column to the middle of the column.

4.7 Structural seismic response analysis

Three ground motion records Imperial Valley, Northridge and Mexico City Aftershock are selected from the above 11 ground motion records to use in the seismic response analysis of structure, and the peak acceleration value is adjusted to 35cm/s^2 and 220cm/s^2, which corresponds respectively to the peak acceleration of frequent-level and rare-level ground motions in the region, then ground motion records after modulation are input respectively to the two prefabricated frame model KJ-1 and KJ-2. The peak value and mean value of story drift distributed along the floor are stimulated by different ground motions, see Fig. 11.

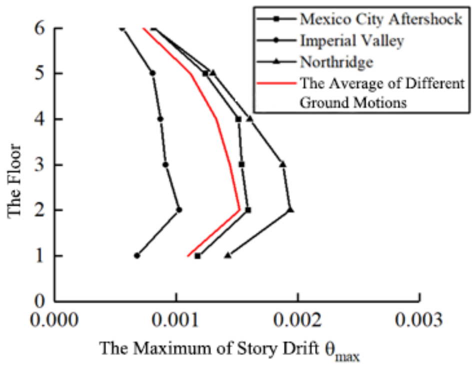

Fig. 11(a) maximum story drift of KJ-1 in frequent-level ground motions

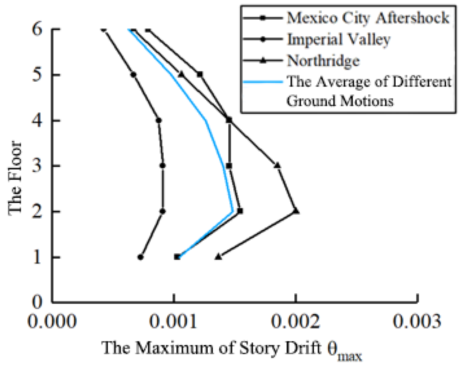

Fig. 11(b) maximum story drift of KJ-2 in frequent-level ground motions

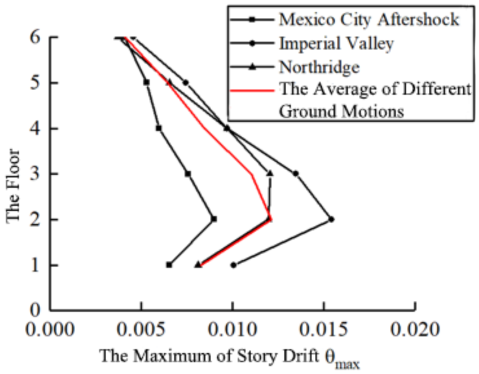

Fig. 11(c) maximum story drift of KJ-1 in rare-level ground motions

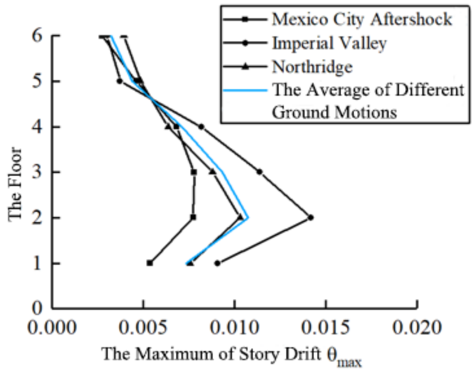

Fig. 11(d) maximum story drift of KJ-2 in rare-level ground motions

Fig. 11 – Distribution of maximum story drift

The mean values of maximum story drift distributed along the floor of the two models at different seismic levels are compared in Fig. 12.

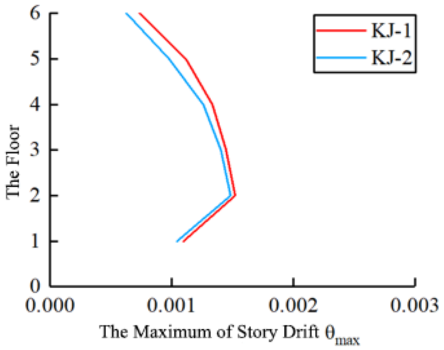

Fig. 12(a) in frequent-level ground motions

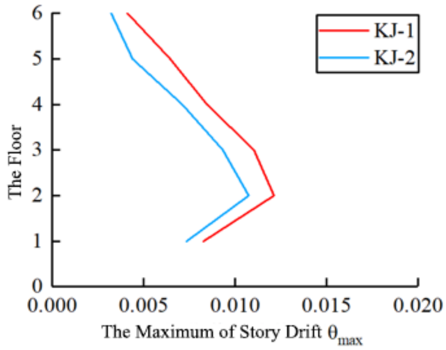

Fig. 12(b) in rare-level ground motions

Fig. 12 – Compare of maximum story drift

The maximum horizontal peak displacement of the top in frequent-level and rare-level ground motions are shown in Table 3.

Table 3 – Maximum Horizontal Peak Displacement

		The maximum horizontal peak displacement of the topΔ/mm		
		Imperial Valley	Northridge	Mexico City Aftershock
KJ-1	frequent-level	13.3	26.6	23.1
	rare-level	183.6	167.0	101.4
KJ-2	frequent-level	12.8	23.5	22.1
	rare-level	174.8	130.6	91.9

As is seen from the above, the maximum story drift of model KJ-1 and model KJ-2 has a similar variation trend along the floors. The maximum story drift both appears on the second floor of the structure, and meets the limit value of story drift stipulated in the Code for seismic design of buildings (GB50011-2010) [14].

The peak values of maximum story drift distributed along the floor of the two models are compared at different seismic levels. Under frequent-level ground motions, the peak value of maximum story drift of model KJ-2 is slightly smaller but similar compared to that of model KJ-1. Under rare-level ground motions, the peak value of maximum story drift of model KJ-2 is significantly smaller than that of model KJ-1, this is because the hysteretic curve of the intermediate-connected columns under low reversed cyclic load is fuller than that of the bottom-connected columns, which indicates that it can better exert its energy dissipation capacity in the earthquake. In addition, its lateral stiffness is also significantly enhanced, so it can effectively weaken the displacement response of the structure under the action of earthquake.

The maximum horizontal peak displacement of the top in frequent-level and rare-level ground motions of model KJ-1 are both larger than that of model KJ-2. Under the excitation of three frequent-level ground motions, the maximum horizontal peak displacement of the top of model KJ-1 is 3.9%~13.2% higher than that of model KJ-2. Besides that, under the excitation of three rare-level ground motions, the maximum horizontal peak displacement of the top of model KJ-1 is 5.1%~27.9% higher than that of model KJ-2. It can be seen that the maximum horizontal peak displacement of the top can be significantly reduced by improving the assembly connection points to the middle of the columns.

5. Conclusion

Based on the above numerical simulation methods and SeismoStruct, two 3-span 6-story prefabricated frames model are designed and simulated, which the assembly connection points are placed in the end of the columns and in the middle of the columns respectively. In addition, collapse vulnerability analysis and seismic response analysis are carried out to study the impact of different assembly connection points on seismic performance from the perspective of the overall structure, and the following main conclusions are obtained:

The hysteretic curves obtained by the finite element analysis of the prefabricated column fit well with the experimental results. The accuracy of this numerical simulation method is verified, and the rationality of the modified Menegotto-Pinto steel constitutive model for simulating the half grout-filled sleeve connection is verified, which provides a reference for the numerical simulation of seismic performance of prefabricated columns with half grout-filled sleeve connection under low reversed cyclic load.

The intermediate-connected columns can reduce the displacement response and enhance the lateral stiffness of the structure for the low-rise & multi-storey precast frames under earthquakes. According to the seismic response analysis, the maximum horizontal peak displacement and story drift of the precast concrete structure with intermediate-connected columns is smaller than that of the bottom-connected columns whether in frequent-level or rare-level ground motions.

It can be found that the precast concrete structure with intermediate-connected columns is better than that of the bottom-connected columns in seismic collapse-resistant capacities. According to the seismic collapse fragility analysis, the collapse probability of the two models are similar when the ground motion intensity index $S_a(T_1)$ is less than 0.28g, with the increase of $S_a(T_1)$, the collapse probability of the precast concrete structure with intermediate-connected columns is smaller than that of the bottom-connected columns.

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