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Progressive Collapse Analysis of Concrete-filled Steel Tubular Column to Steel Beam Connections Using Multi-scale Model

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ABSTRACT

The multi-scale model which combined the fiber beam element with the fine element was used to investigate the progressive collapse performance of steel beam to concrete-filled steel tubular (CFST) column connections. By using the nonlinear static analysis method and taking into account the influence of the adjacent framework of joints, the resistance of progressive collapse, the failure modes and the stress distribution revealed the resistance mechanism of these joints during the process of progressive collapse. And the vertical displacement time history curves of joints which displayed the progressive collapse resistance demands of these joints were obtained by using the nonlinear dynamic analysis method. The relationship between resistance capacity and resistance demand of these joints were obtained by analyzing the nonlinear static analysis results and the nonlinear dynamic analysis results. These analysis results showed that the frame structure with these joints which enabled to form the resistance mechanism and new alternate path of unbalanced loads can prevent the occurrence of progressive collapse after the failure of column connected to joints. And the adjacent framework can improve the ability of anti-progressive collapse of these joints.

1. Introduction

Since the 911 event, more and more researchers have focused on the investigation of the resist progressive collapse performance of structures [1–6]. It is important to predict the failure mechanism and the alternate load path of structures which occur progressive collapse under abnormal accidental events. Some researchers have found that the joint areas connected with the failure column can generate an unbalanced load when frame structures occur progressive collapse. Moreover the unbalanced load is able to disperse to the adjacent areas by the beam resistance mechanism and catenary resistance mechanism which provide by the beam-column joints [7,8]. Therefore the beam-column joints are key elements for frame structures to resist progressive collapse events.

Some experimental investigations have focused on the performance of steel beam-column joints [9–15] and reinforced concrete beam-column joints [16,17]. These study results suggest that the catenary resistance mechanism developed in the beams and connections plays a critical role in the resistance of structure progressive collapse. However, these experiments have only studied the joint parts and did not consider the effects of the other structural parts that are connected with these joints. Moreover, there are few researches focus on the study of progressive collapse of the steel beam to CFST column connections by now. Thus the performance of some steel beam to CFST column connections was investigated under a central-column-removal scenario in this paper. In order to improve the understanding of the behavior of these connections to resist progressive collapse, the multi-scale numerical model which could reflect the effect of the adjacent structures was used to investigate the performance of these joints. And these numerical models adopted the nonlinear static analysis method and the nonlinear dynamic analysis method to study the progressive collapse resistance capacity of the steel beam to CFST column connections.

2. Steel beam to CFST column connection model

2.1. Design model

In order to study the progressive collapse performance of the joints in steel beam to CFST tubular frames under a central-column-removal scenario, two 9-story and 4-span CFST columns with I-shape steel beams planar frames were designed. The circular and square section columns, respectively, were chosen as the frame columns. And the two frames have the same size in story height and span. The height of first story is 4.2 m, and the height of other stories is 3.6 m. The span is 6.6 m. The middle joints at the first story are the main objects to be investigated in this paper. Fig. 1 shows the four types of these joints which include steel beam to circular or square CFST column connections with outside stiffening ring plate or with penetrating ring plate. The steel

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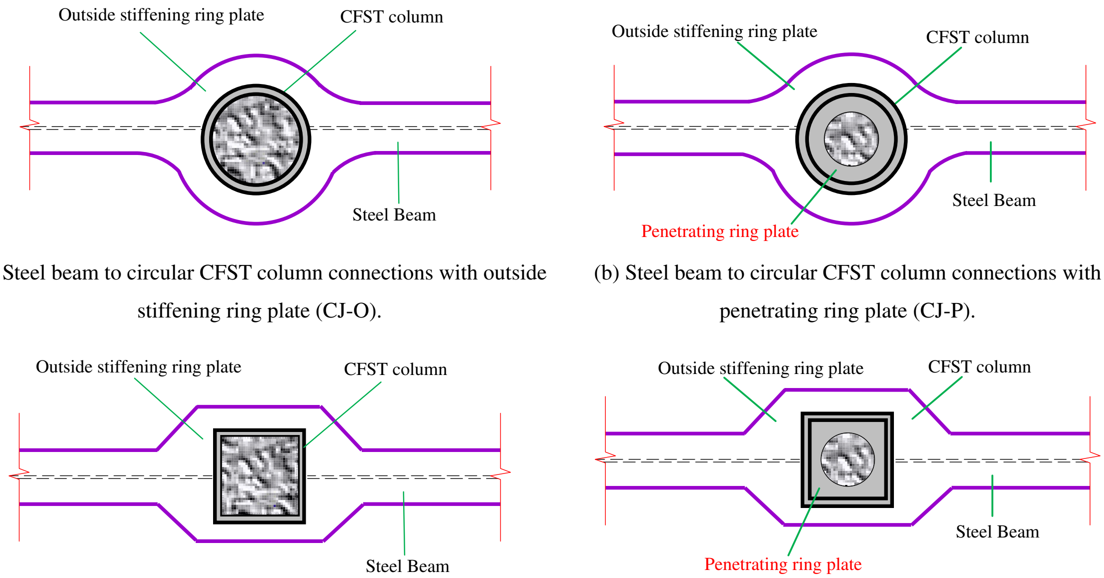

(a) Steel beam to circular CFST column connections with outside stiffening ring plate (CJ-O).

(b) Steel beam to circular CFST column connections with penetrating ring plate (CJ-P).

(c) Steel beam to square CFST column connections with outside stiffening ring plate (SJ-O).

(d) Steel beam to square CFST column connections with penetrating ring plate (SJ-P).

Fig. 1. Typical steel beam to CFST column connections.

beams and ring plates are welded to the steel tube to keep the connections firm. Table 1 shows the geometry and material information of steel beam to CFST column connections in the frame structures. The outer or inner plate size stands for its width outside or inside the column.

2.2. Finite element model

The finite element models for steel beam to CFST column connections were built in the finite elements software ABAQUS. And the multi-scale model was used for the finite element analysis. The multi-scale model is a kind of simulate method that combine different finite element types in one numerical model [18]. This model feature is illustrated in Fig. 2. The fine element is usually used for substructures or members simulation and has a high computational accuracy. In contrast, since the fiber beam element has a high computational efficiency, it is generally adopted by the global structural numerical calculation. Therefore the multi-scale model can take into account the high computational accuracy and the advantages of high computational efficiency at same time. But the interaction between these two elements is a key factor to consider when the multi-scale model is employed. The interface of these two elements need satisfy deformation compatibility condition, at the same time this interaction should keep the degree of freedom of fiber beam elements constant and do not increase additional constraints for fine elements. The coupling interaction in ABAQUS is employed for the fine element and fiber beam element in this paper.

Considering the advantage of the multi-scale model, it can be used in the finite element models for the analysis of the steel beam to CFST column connections. The joint parts which should be studied more thoroughly can use the fine element and other parts like beams and columns adopt the fiber model. This numerical model considers the tension behavior that is provided by the frame structures on the joints and reflects the deformation and failure mode of joint areas in detail.

Since the inflection point of the beam is located at the midpoint of the beam, only one-half span of the steel beam of the fine model is adopted. Fig. 3 shows the multi-scale model. For the fine joint parts, the four nodes shell element S4 was used for steel beam and tube and the concrete in columns employed the 8 nodes solid element C3D8. The fiber beam model iFiberLUT [19] which was developed in ABAQUS was employed by other parts in the frame structures. The detail information of concrete and steel material constitutive model could be found in the reference [20]. Fig. 4 shows the finite element models of these four designed joint parts.

The progressive collapse analysis of these joint models adopted the nonlinear static analysis method and the nonlinear dynamic analysis method. By using the nonlinear static analysis method, the progressive collapse mechanism, the failure modes and the stress distribution revealed the progressive collapse resistance capacity of these joints. And the vertical displacement time history curves of typical joints which displayed the progressive collapse resistance demand of these joints were obtained by using the nonlinear dynamic analysis method.

3. Numerical verification

Some experiments were chose to establish the numerical model in order to verify the validity of the finite element model. These experiments included CFST columns and steel joint models.

Table 1
The information of connections in the frame structure.

Joint Types		Column section/mm	Steel beam section/mm	Outer plate size/mm	Inner plate size/mm	Concrete cubic compression strength/MPa	Steel yield strength/MPa
Circular	CJ-O	Φ 500 × 12	I 450 × 250 × 12 × 16	125	0	40	345
	CJ-P	Φ 500 × 12		50	75		
Square	SJ-O	□ 500 × 12	I 500 × 300 × 12 × 16	150	0		
	SJ-P	□ 500 × 12		60	90		

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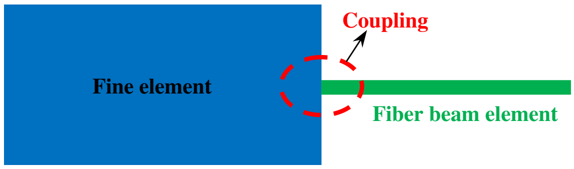

Fig. 2. The multi-scale model.

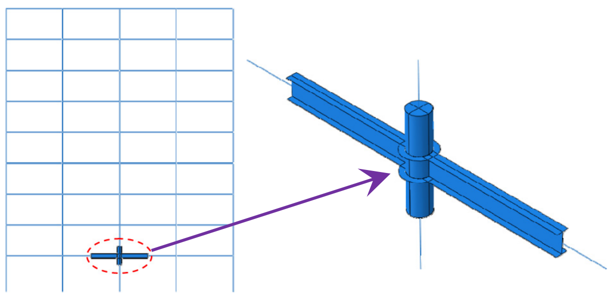

Fig. 3. The multi-scale model for steel beam to CFST column connections.

3.1. Multi-scale model verification

Hu et al. [21] studied the behavior of concrete-filled steel tube columns under axial compression. A circular section specimen numbered as CU-40 and a square section specimen numbered as SU-70 were chosen as the numerical models. Fig. 5 shows the basic geometry and material information and the multi-scale models of these specimens. In the multi-scale models, one half employed the fine element and the other part adopted fiber beam element. The axial force and strain curve that were simulated by numerical models matched the test result well as shown in Fig. 6. These calculation results showed that the multi-scale model could be used for CFST structures analysis.

3.2. Progressive collapse model verification

Sadek et al. [10] studied the progressive collapse performance of a steel moment connection under a column removal scenario by experiment. The vertical displacement was applied on the middle joint by static method in this experiment. Therefore this connection was modeled in ABAQUS by using the nonlinear static method. Fig. 7 shows the numerical model of steel connection. The horizontal force and displacement curve of calculation result agrees well with the experimental curve as shown in Fig. 8.

The experiment result showed that the failure region located at the weakening section of steel beams finally. The fracture started to occur at the bottom flange and then extended to the web. The numerical simulation result also showed the same failure mode which was showed in Fig. 9. Therefore these numerical models could reflect the collapse behavior and failure mode well in this paper.

4. Nonlinear static analysis

The progressive collapse is a nonlinear dynamic process, but the failure mode and the resistance of progressive collapse could be obtained visibly by nonlinear static analysis. Therefore the nonlinear static method was used to investigate the steel beam to the CFST column joints firstly. In the nonlinear static analysis, the middle column was removed firstly, then the vertical displacement U was applied to the bottom of the residual column which connected to the middle joint until the middle joint lost its resistance. Fig. 10 shows the loading path of the nonlinear static analysis. Through the nonlinear static pushdown, the failure modes and the process of the anti-progressive collapse of these joints could be obtained.

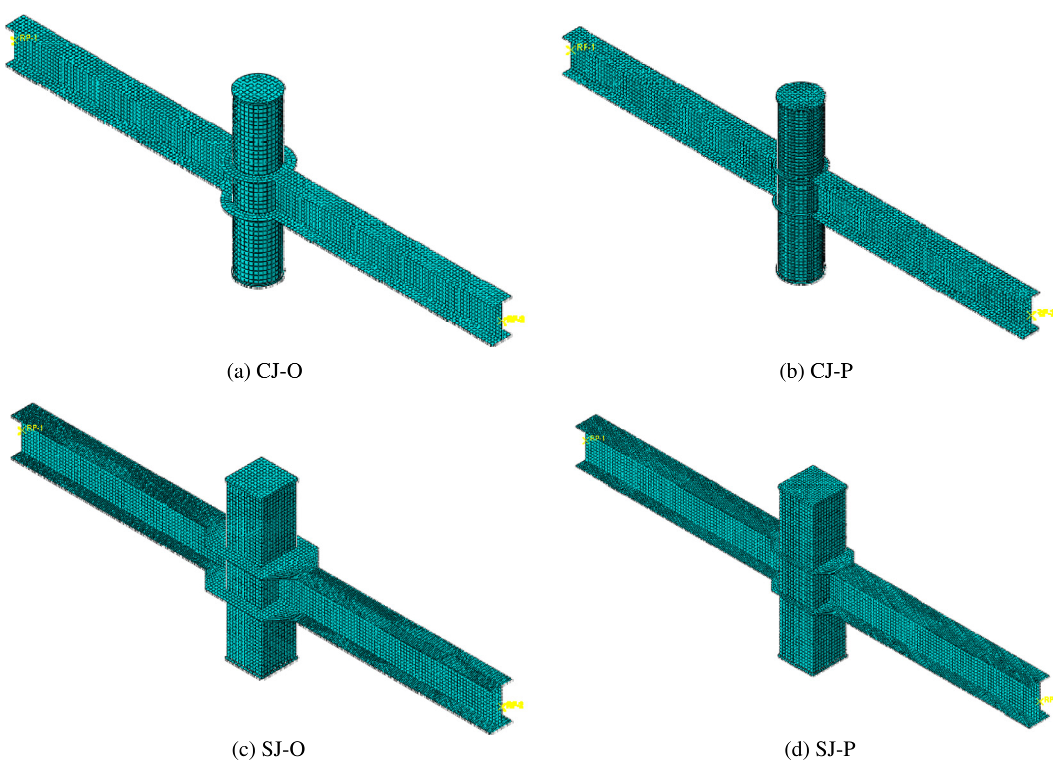

Fig. 4. The finite element model of joint parts.

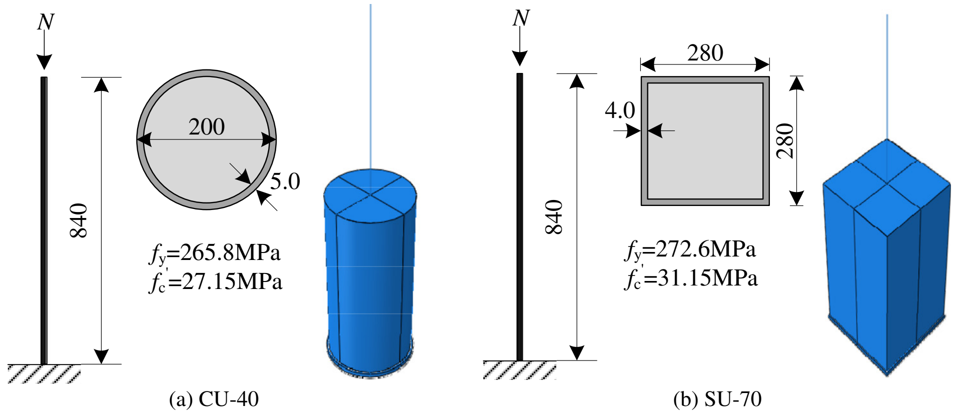

Fig. 5. CFST columns information and numerical models.

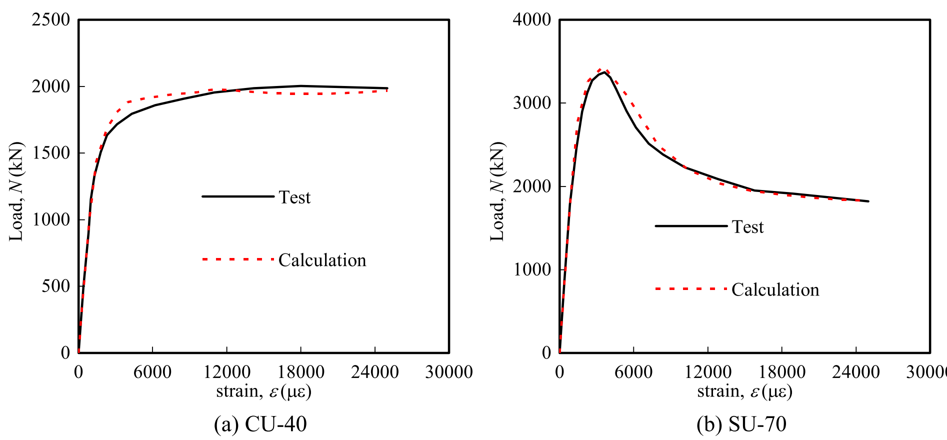

Fig. 6. Comparison results of CFST columns under vertical monotonic load.

4.1. The result of the nonlinear static analysis

Through the processes of the nonlinear static analysis, the vertical resistance of progressive collapse P and the vertical displacement U were obtained. Fig. 11 shows the curves of P—U which stands for the capacity of anti-progressive collapse of these joints connected to the frame structure. Since the square shape frame has steel beams with greater depth, the joints with the square shape column can provide better performance than the circular shape column. Moreover the outside stiffening ring plate joints shows better performance in the nonlinear static analysis when they compare the performance with the penetrating ring plate joints. This is because that the outside stiffening ring plates which have a wider ring. And these outside ring plates can

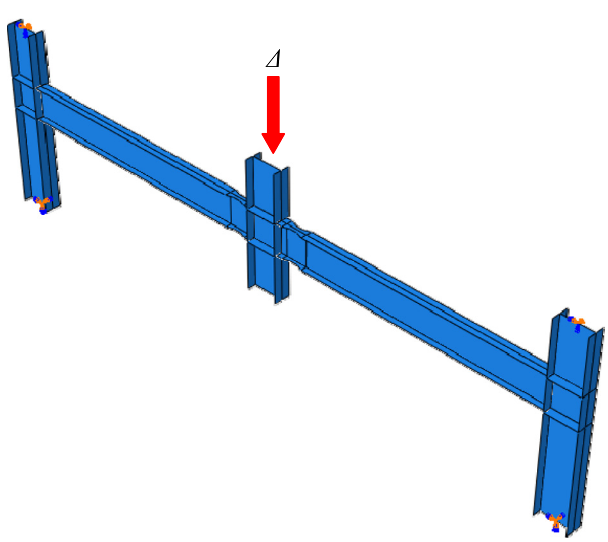

Fig. 7. The numerical model of steel connection.

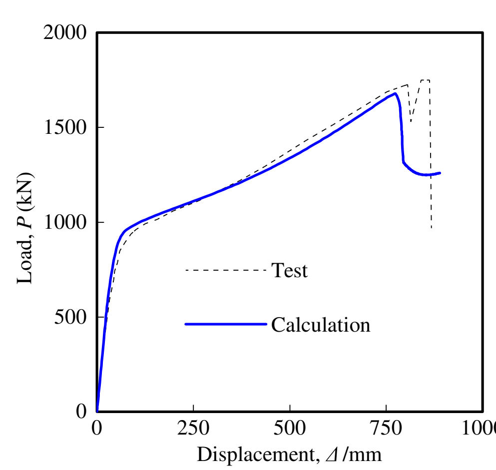

Fig. 8. Comparison of collapse resistant performance results of steel connection.

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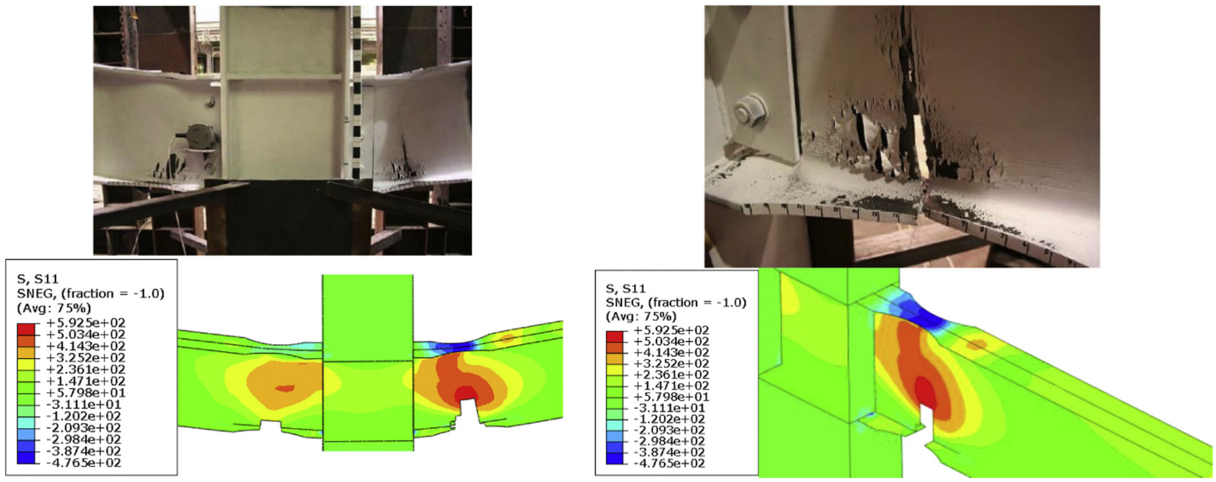

(a) Global view (b) Local view

Fig. 9. Comparison of the failure modes.

cause the unbalance energy that is triggered by the progressive collapse process dissipate to the adjacent frame structure efficiently.

The resistance mechanisms of these four joints present similar trend in the nonlinear static analysis result. In order to analyze the features of resistance mechanisms of these joints, the cross profiles of steel beams are chosen as the analysis region. Taking the SJ-O as an example, Fig. 12 shows the axial direction stress S11 distribution in the cross profile of the steel beams. Fig. 12(a) shows the beam mechanism phase. During this phase the upper flange of the beam section is in compression and the bottom flange is in tension. The neutral axis almost locates at the middle of the web. Fig. 12(b) shows the mixed mechanism phase. With the vertical motion of the middle joint, the neutral axis moves upward. The tension regions in the cross profile enlarge and the compression regions become smaller gradually. Fig. 12(c) shows the catenary mechanism phase. The whole section of the steel beam is in tension. Fig. 12(d) shows the failure phase. When the bottom flange and the web start to fracture, the stress of these regions become zero, and the upper flange remains some tension ability to provide resistance.

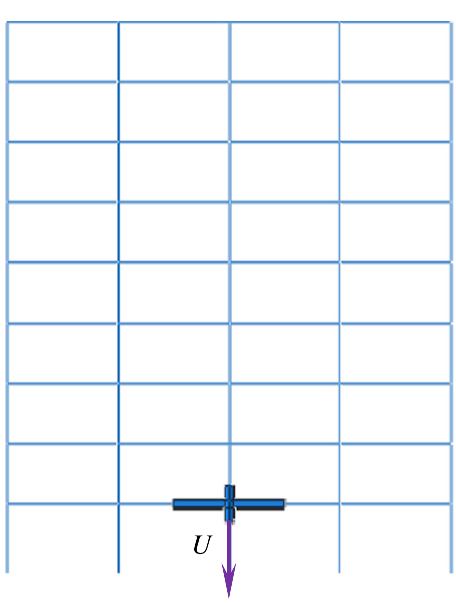

Fig. 10. The loading path of the nonlinear static analysis.

The resistance mechanisms which are provided by these joints change from the beam resistance mechanism to the catenary resistance mechanism until these joints fail and cannot contribute any resistance. The anti-progressive collapse mechanisms of these joints are included in Fig. 13. *OA* phase represents the beam mechanism which is provided by the *anti*-bend capacity of steel beams. *BC* phase stands for the catenary mechanism, the tensile ability of steel beams provides this resistance mechanism during this phase and the total cross profiles of the steel beam are in tension. *AB* phase, which is between the *OA* and *BC* phase, indicates the mixed mechanism. During the mixed mechanism phase, the beam mechanism reduces and the catenary mechanism enhances progressively. *CD* phase is named as failure, because the steel beams start to fracture and the joints lose the anti-collapse ability gradually during this phase.

In Fig. 13, the point *A* is the resistance of the beam mechanism phase. Going across this point, the tension regions of steel beam sections enlarge. The point *B* stands for the state that the total cross profile of the steel beam start to become tension. The point *C* indicates the resistance of the catenary mechanism phase and it is the starting point of fracture in steel beams. Combining the Fig. 11 and Fig. 13, the resistance of beam mechanism, catenary mechanism and the resistance ration of these two mechanism are listed in Table 2. The result shows that the catenary mechanism could improve the anti-collapse ability of joints in different degree. If the development of the catenary mechanism gives full play to the resistance of progressive collapse, the frame structure could provide

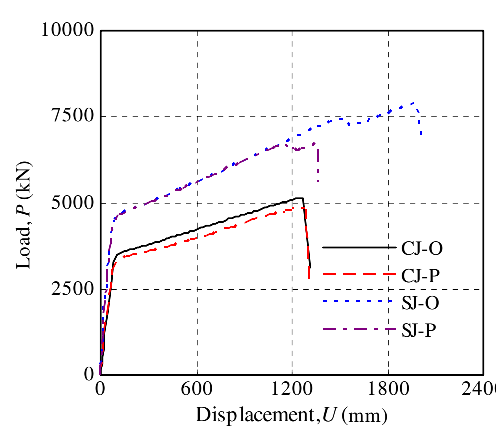

Fig. 11. The *P*—*U* curves for the anti-progressive collapse performance of these joints.

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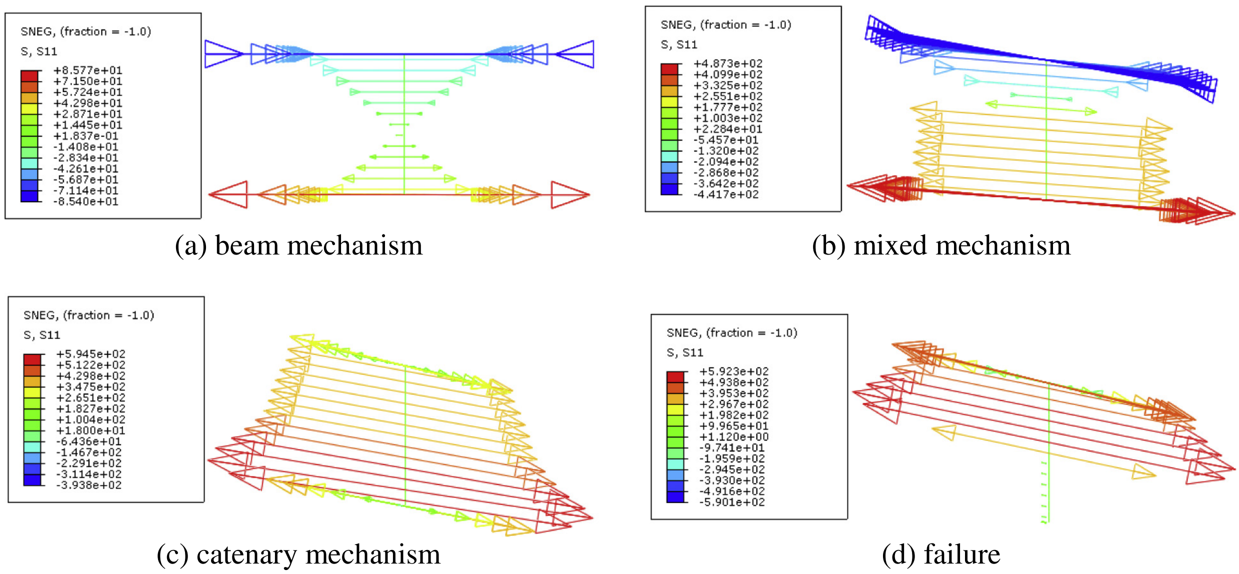

Fig. 12. The axial direction stress S11 distribution of the steel beams.

robustness steadily. Therefore the steel beams connected to joints play a vital role in structural resistance to progressive collapse.

4.2. Deformation and failure mode of joints

Fig. 14 shows the failure mode and the axial direction stress S11 distribution at these joints. The failure modes of CJ-O, SJ-O and SJ-P joint are alike, i.e., the connection between the steel beam and ring plate fractures at the bottom flange of the steel beam and extends to the web. The top flange buckles in different degrees. Moreover, the steel tube has a convex deformation under the tension from steel beams. The top flanges of steel beams in CJ-P joint are in compression. From the stress distribution in Fig. 14, the joint fractures at the connection between the web of steel beams and the tube. The full section of steel beam is not in tension which causes that the joint cannot contribute the catenary mechanism adequately. This is also the reason that the C/A value of CJ-P is minimum in Table 2.

During the process of the collapse analysis, the resistance of joints is mainly provided by the steel beam and outer ring plate connected to the steel tube. Even the concrete inside the steel tube is involved in the loading process, the concrete parts in the CJ-O and SJ-O joints contribute less resistance than the concrete parts in the CJ-P and SJ-P joints can provide. Fig. 15 shows the vector graphs of the concrete plastic strain at the ring plate sections. X direction stands for the axial direction of steel beams, while Z direction is the perpendicular direction of the axial direction of steel beams. PE11 is the concrete plastic strain in X direction and PE33 stands for the concrete plastic strain in Z direction. The concrete part is in tension along the X direction, while they are in compression along the Z direction. As the inner ring plates locate in the concrete part, the loads transferred to the concrete parts by the inner ring plates. This is also the reason that the plastic strain of concrete parts inside the steel tube with inner ring plate are in higher level.

Fig. 16 shows the Mises stress distribution and the deformation of the whole frame structure in multi-scale model. When the joint is pushed vertically downwards, the beam and column at the two spans and stories connected the middle joint have obvious stress redistribution. The Mises stress of other joints which are modeled by the fiber beam element are in a higher level. The columns above the middle joint which is modeled by the fiber beam element have a vertical rigid body displacement and do not carry obvious loads, therefore they are in lower stress level. The steel beams carry unbalanced loads and transfer them to the adjacent region in the frame structure, which caused the high stress level in the steel beams. The columns at the first story tilt to the middle of this frame structure with the vertical motion of the middle joint.

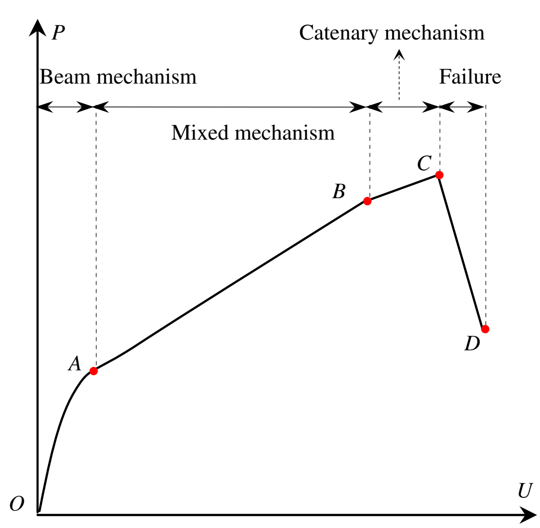

Fig. 13. The anti-progressive collapse mechanism.

5. Nonlinear dynamic analysis

5.1. Nonlinear dynamic analysis method

The nonlinear dynamic method could reflect the characteristic of progressive collapse realistically. Through this method, the progressive collapse resistance demand of these joints was obtained. In order to

Table 2
The resistance of joints.

Items	CJ-O	CJ-P	SJ-O	SJ-P
A/kN	3581.95	3383.85	4655.25	4631.7
C/kN	5110.66	4783.04	7899.81	6714.28
C/A	1.43	1.41	1.70	1.45

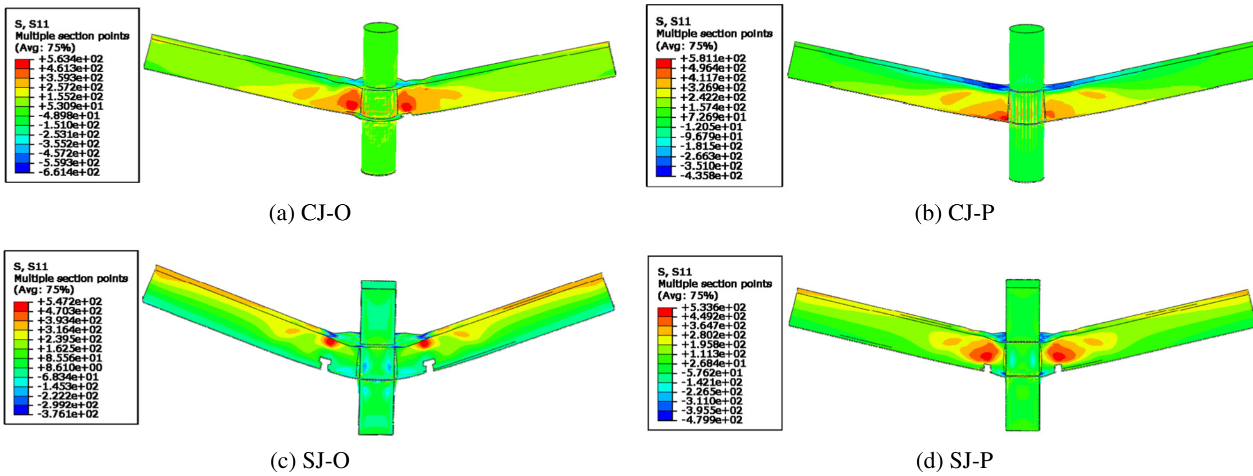

Fig. 14. The failure mode and S11 distribution during nonlinear static analysis.

achieve the nonlinear dynamic analysis, three analytical steps were applied as listed below.

(1) Apply the entire regular load on the frame structure;
(2) Remove the middle column at the first story rapidly during the column failure time; and.
(3) Use the nonlinear dynamic method to analyze the residual frame structure until the structure collapses or keeps smooth.

The GSA2003 (The United States General Services Administration 2003) [22] proposed a load combination calculation method. For dynamic analysis purposes the following vertical load shall be applied downward to the structure under investigation: $Load = DL + 0.25LL$. Where DL and LL mean dead load and live load, respectively.

The structure damping should be considered in the dynamic analysis. The Rayleigh damping is adopted in this paper. The damping ratio ξ of frame structure is usually intermediate between 0.02 and 0.05. The smaller the damping, the larger the dynamic respond. Therefore the damping ratio is taken as 0.02 conservatively.

The column failure time has a major influence on the dynamic response of the residual structure. However, the failure time of support column is different when frame structures suffer from different structural progressive collapse events. The column failure is in a slow process when frame structures subject to fire, whereas the column failure caused by explosion events is a transient process. Therefore a reasonable value need be confirmed for the column failure time in the nonlinear dynamic analysis. The GSA2003 proposed that the support column should be removed over a time period that is no more than 1/10 of the period associated with the structural response mode for the vertical column removal when using the dynamic analysis method. Based on this criterion and the mode analysis of the CFST frame structure, 0.01 s is chosen as the middle column failure time in the multi-scale model in this paper.

5.2. The result of nonlinear dynamic analysis

Analyzing the multi-scale model by using the nonlinear dynamic analysis method, the time-history curves of vertical displacement U of these joints are obtained. Fig. 17 shows the time-history curves of the nonlinear dynamic analysis results.

When the CFST column sections are same, the calculation results of joints with the outside ring plate and joints with the penetrating ring plate do not show the difference obviously. This is because that these multi-scale frame structure models do not collapse during the analysis, the frame structures dissipate the energy which is caused by the unbalanced load from the middle column failure process. The joints connected the failure column could not develop their *anti*-collapse ability at the catenary mechanism adequately.

From the calculation result, we could also find that the trend of the amplitude and variety are analogous for the same shape of the CFST columns. The displacements of joints connected to the circular shape column are greater than the square shape's. The maximum dynamic displacement to the ultimate displacement ratio is 3.694 for circular section and 4.0259 for square section. There are two reasons for this phenomenon, one is that the Rayleigh damping of the circular section CFST frame is lower even though the frames uses the same damping ration. The lower the damping is, the greater displacement arises. The other reason is that the square section columns have bigger section moment of inertia. Therefore CFST frames with square section columns are able to provide more horizontal constraint force which leads to smaller vertical displacement of this joint type.

5.3. Stress distribution and deformation of joints

Fig. 18 shows the axial direction stress S11 distribution at these joints after the vertical vibration process. The stress distributions of the steel beam near the joint are similar in these 4 joints. The top flange is in compression and the lower flange is in tension. Comparing the stress distribution of the joints with outside ring plate and with the penetrating ring plate, it is obvious to find that the zone of the maximum stress of these joints with the outside ring plate is smaller. When the outside ring plate become wider, it is more effective to transfer the unbalanced load during the progressive collapse.

The Mises stress distribution and the deformations of the whole frame structure in multi-scale model are shown in Fig.19. After the middle column is removed at the first story, the joints vibrate vertically and the Mises stress of the joints which are modeled by the fiber beam elements are in a higher level, which is similar to the result of the nonlinear static analysis. Moreover, the columns above the middle joint have a vertical rigid body displacement and bear lower loads.

Observing the deformation mode in Fig. 19, the joints connected with the circular CFST columns have a greater vertical displacement and present a bent shape mode obviously. However the joints connected with the square CFST columns keep smaller vertical displacement relatively. Moreover the nonlinear dynamic analysis of the multi-scale model demonstrates that the main resistance mechanism is the beam mechanism. The greater depth of the steel beam is, the more resistance of beam mechanism the joints in the frame could provide. The joints which can form the resistance mechanism and new alternate path of unbalanced loads prevent the occurrence of progressive collapse of frame structures after the failure of middle column.

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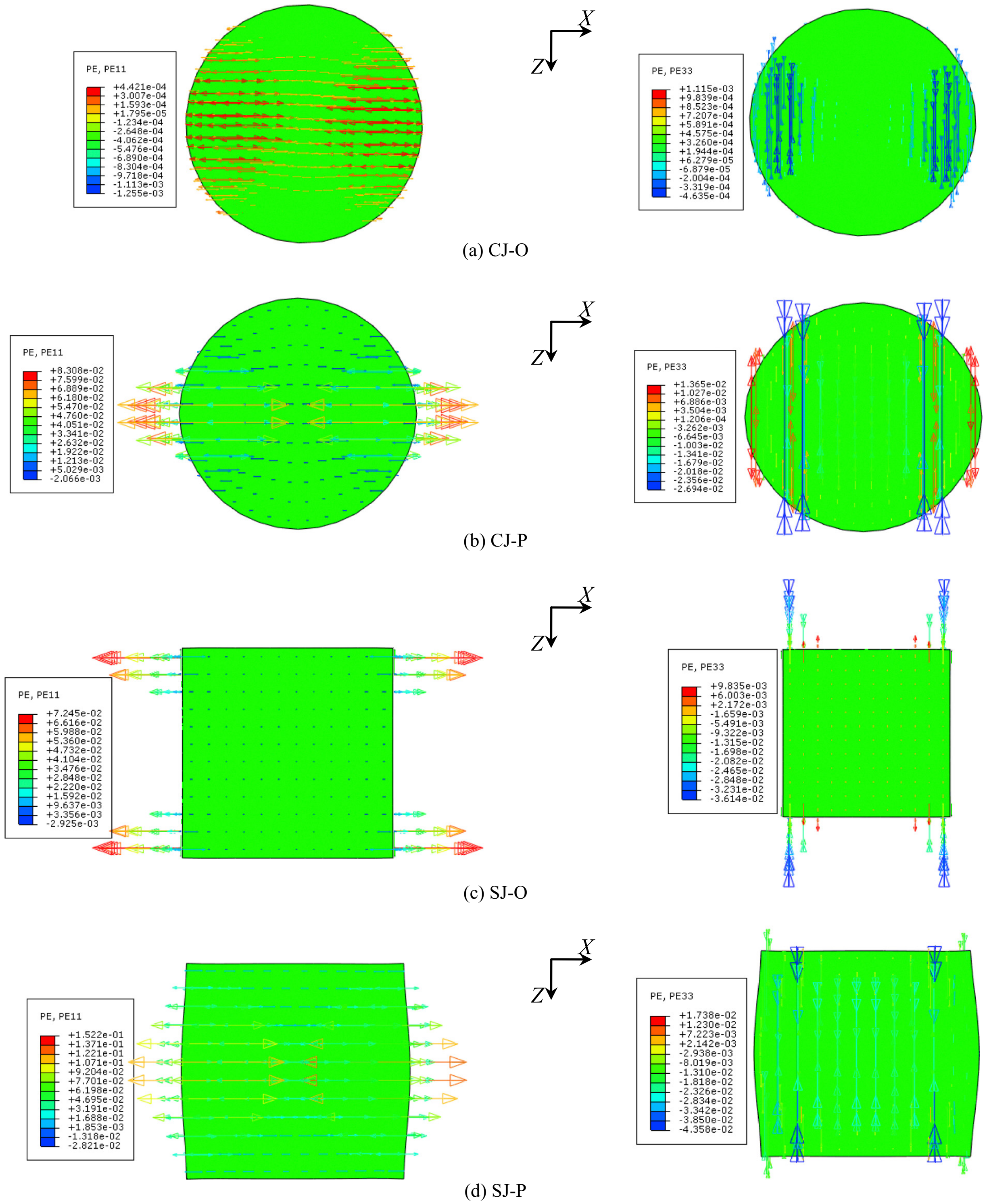
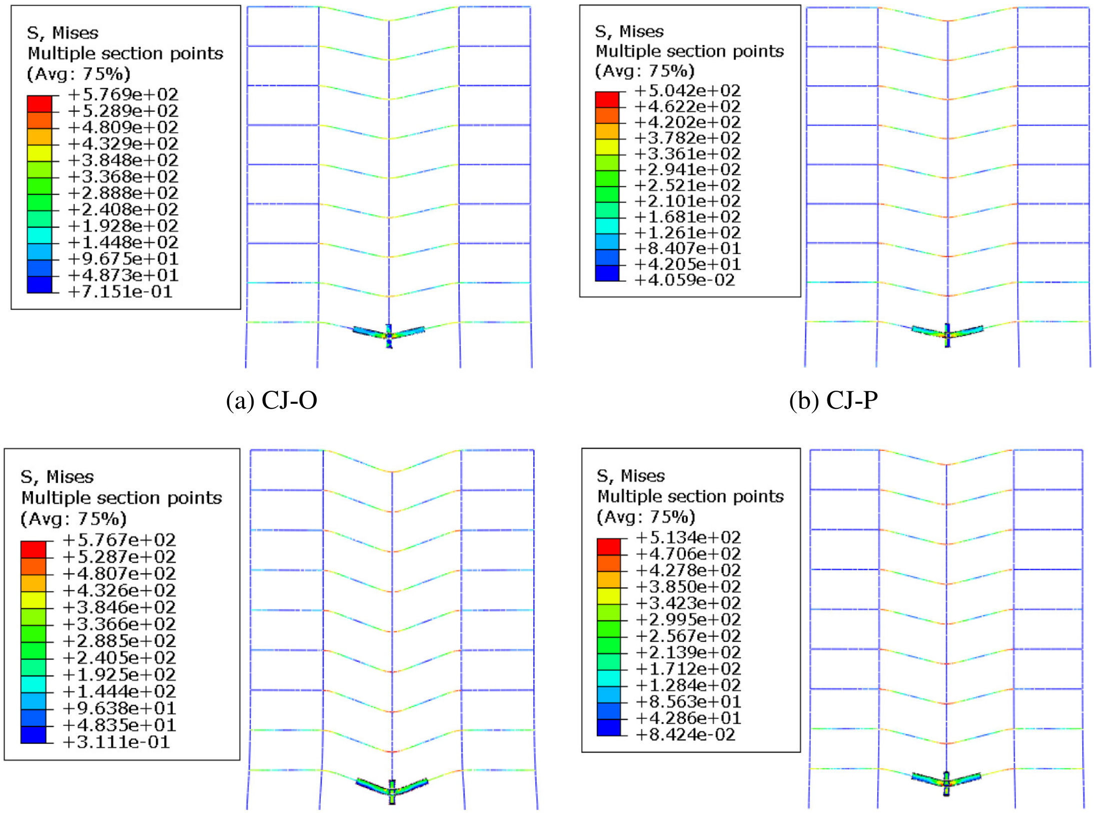


Fig. 15. The vector graphs of the concrete plastic strain.

6. Comparison between the resistance and resistance demand

By using the nonlinear static analysis method, the results present the bear capacity of progressive collapse, while by using the nonlinear dynamic analysis method, the vertical displacement time history curves of joints which indicates the resistance demand are obtained. The relationship between resistance capacity and resistance demand of these joints are obtained by comparing the nonlinear static analysis results with the nonlinear dynamic analysis results. The comparison results are shown in Fig. 20. Since the maximum dynamic displacements of joints at the frame with the same CFST columns section type have little difference, the maximum dynamic displacement of joints U-MAX chooses the bigger displacement value as the criterion in order to unify the resistance demand of beam mechanism for the same column section.



(a) CJ-O (b) CJ-P

(c) SJ-O (d) SJ-P

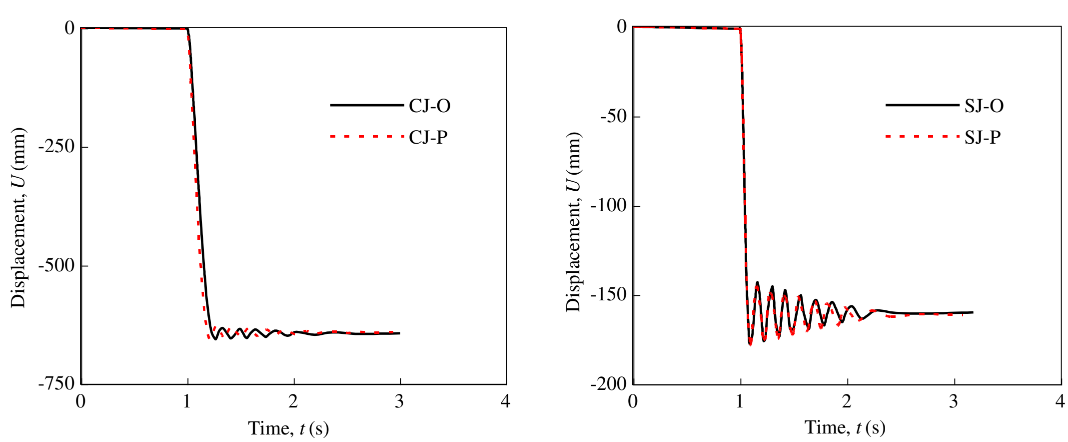
Fig. 16. The Mises stress distribution and deformation of the frame structure during nonlinear static analysis.

The Fig. 20 shows that the maximum dynamic displacement of joints CJ-O and CJ-P are located at the mechanism which is dominated by the catenary mechanism. And for the SJ-O and SJ-P joints, the maximum dynamic displacement are in the mixed mechanism phase that the beam mechanism declines and the catenary mechanism develops preliminarily. From the progressive collapse analysis results of these joints, the resistance demand is less than the resistant when the middle column is removed. This is also the reason why the frame structures do not occur progressive collapse during the nonlinear dynamic analysis.

These comparisons have also showed that the frame structure with these joints enables to form some new alternate paths of unbalanced loads. These new load paths delay and even avoid the occurrence of progressive collapse after the failure of column. The resistance mechanisms could reduce the probability of progressive collapse. In order to guarantee these resistance mechanisms develop effectively, the steel beam to CFST column connection should be firm and cannot failure before they offer sufficient resistance.

7. Conclusions

This paper presented the progressive collapse analysis of the steel beam to the CFST column connections by using multi-scale model. In



(a) CJ-O/P (b) SJ-O/P

Fig. 17. The time history curve of nonlinear dynamic analysis.

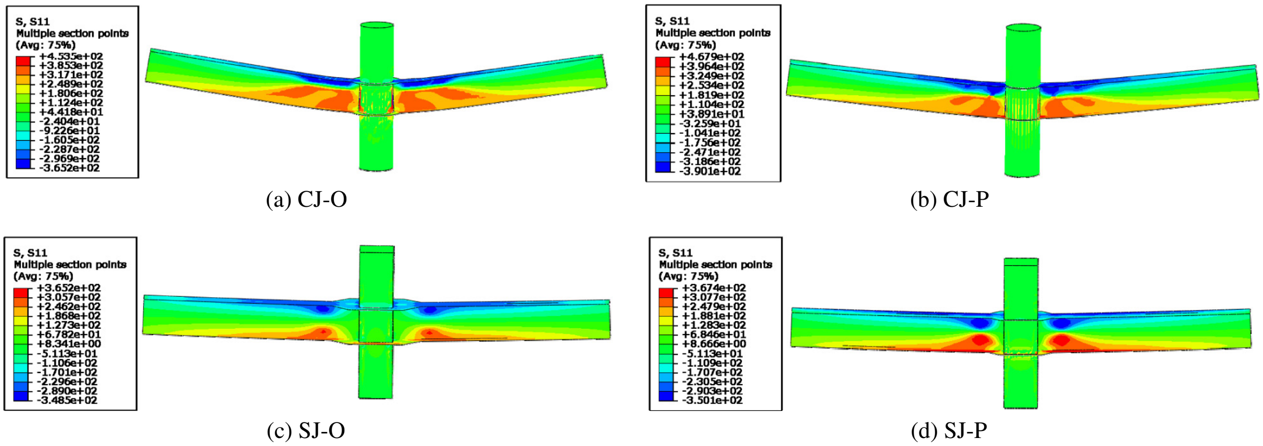


Fig. 18. The S11 distribution during nonlinear dynamic analysis.

these investigations, the authors used both the nonlinear static method and nonlinear dynamic method to conduct analysis. Several conclusions can be drawn in this paper:

(1) There are 4 phases of resistance mechanism of joints: beam mechanism, mixed mechanism, catenary mechanism and failure. The catenary mechanism plays a vital role in the resistance of progressive collapse.

(2) By using the nonlinear static analysis, the failures of joints are located at connection between the steel beam and ring plate during the progressive collapse process. Due to the tension function of the neighboring frame structure, these joints can provide more resistance to prevent the occurrence of progressive collapse.

(3) By using the dynamic analysis, the resistance demand can be obtained. Comparing the nonlinear dynamic analysis results with the nonlinear static analysis results, it has showed that joints should have a strong connection between the steel beam and CFST column. The quality of the connections is key factor for frame structure to resist the progressive collapse.

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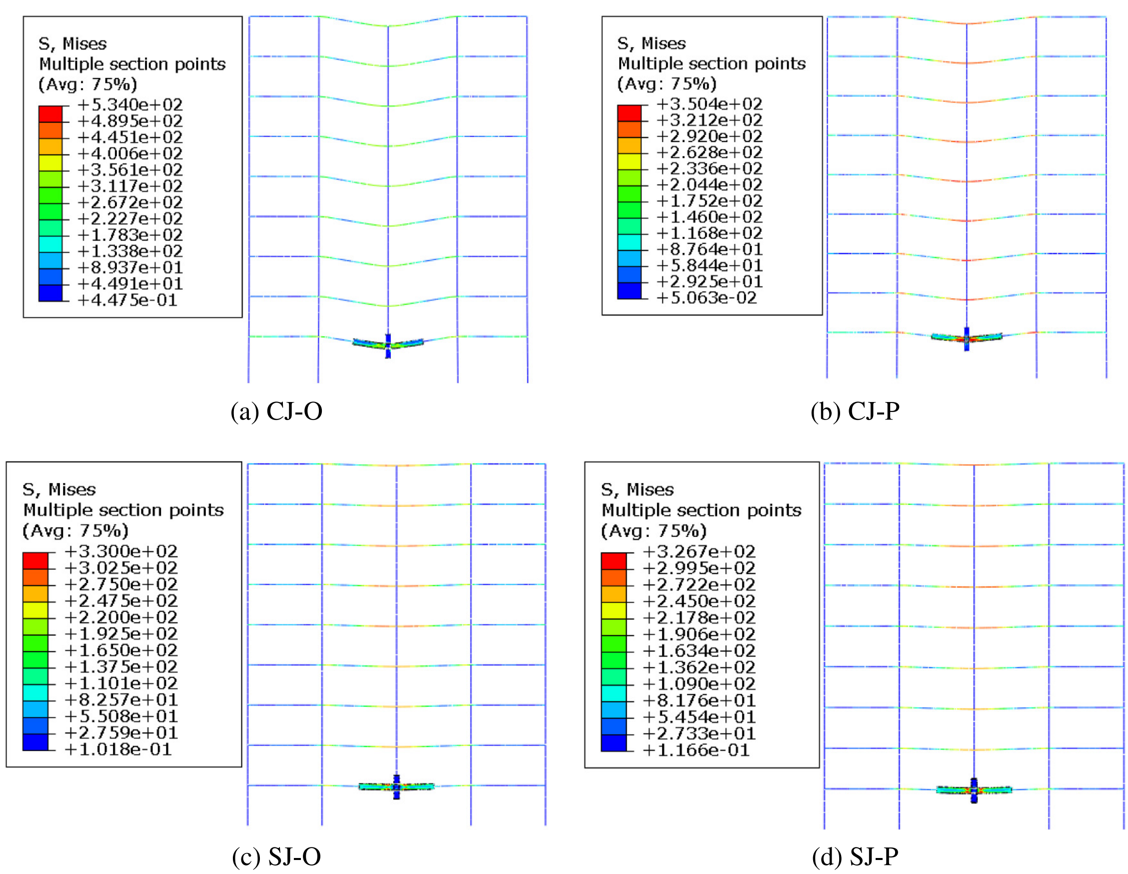

Fig. 19. The Mises stress distribution and deformation of the frame structure during nonlinear dynamic analysis.

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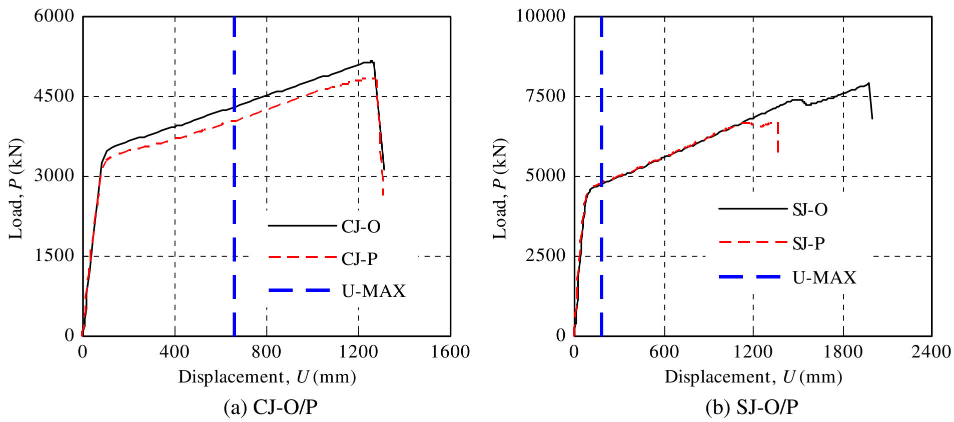

Fig. 20. The comparison between resistance and resistance demand.

References

[1] Sasani M, Bazan M, Sagiroglu S. Experimental and analystical progressive collapse evaluation of actual reinforce concrete structure. ACI Struct J 2007;104(6):731–9.

[2] Yi WJ, He QF, Xiao Y, Kunnath SK. Experimental study on progressive collapse- resistant behavior of reinforced concrete frame structures. ACI Struct J 2008;105(4): 433–9.

[3] Fu F. Progressive collapse analysis of high-rise building with 3-D finite element modeling method. J Constr Steel Res 2009;65(6):1269–78.

[4] Fu F. 3-D nonlinear dynamic progressive collapse analysis of multi-storey steel composite frame buildings-parametric study. Eng Struct 2011;32(12):3974–80.

[5] Fascetti A, Kunnath SK, Nistico N. Robustness evaluation of RC frame buildings to progressive collapse. Eng Struct 2015;86:242–9.

[6] Ren PQ, Li Y, Lu XZ, Zhou YL. Experimental investigation of progressive collapse resistance of one-way reinforced concrete beam–slab substructures under a middle-column-removal scenario. Eng Struct 2016;118:28–40.

[7] Li S, Zhao Y, Zhai CH, Xie LL. The influences of joint on the progressive collapse-resisting performance of RC frames. Engineering Mechanics 2012;29(12):80–7 (in Chinese).

[8] He Z, Huang GH. Progress in studies of catenary action in frame structures. Advances in Mechanics 2012;42(5):547–61 (in Chinese).

[9] Lee C, Kim S, Lee K. Parallel axial-flexural hinge model for nonlinear dynamic progressive collapse analysis of welded steel moment frames. J Struct Eng ASCE 2010; 136(2):165–73.

[10] Sadek F, Main JA, Lew HS. An experimental and computational study of steel moment connections under a column removal scenario. National Institute of Standards and Technology U.S. Department of Commerce Report; 2010.

[11] Yang B, Tan KH. Numerical analyses of steel beam-column joints subjected to catenary action. J Constr Steel Res 2012;70(3):1–11.

[12] Yang B, Tan KH. Experimental tests of different types of bolted steel beam-column joints under a central-column-removal scenario. Eng Struct 2013;54(9):112–30.

[13] Huo JS, Wang N, Chen Y. Experimental study on collapse resistance of welded beam-column connection substructure of steel frame based on seismic design. Journal of Building Structures 2014;35(4):100–8 (in Chinese).

[14] Wang W, Li L, Chen YY. Experimental investigation on progressive collapse behavior of WUF-B connections between SHS column and H beam. Journal of Building Structures 2014;35(4):92–9 (in Chinese).

[15] Wang W, Li L, Chen YY, Yan P. Experimental study on progressive collapse behavior of CHS column-to-H beam connections with outer-diaphragm. Journal of Building Structures 2014;35(7):26–33 (in Chinese).

[16] He QF, Yi WJ. Experimental study of the collapse-resistant behavior of RC beam-column sub-structures considering catenary action. Chin Civil Eng J 2011;44(4):52–9 (in Chinese).

[17] Qian K, Li B. Experimental and analytical assessment on rc interior beam-column subassemblages for progressive collapse. J Perform Constr Facil 2012;26(5):576–89.

[18] Lu XZ, Ye LP, Ma YH, Tang DY. Lessons from the collapse of typical RC frames in Xuankou School during the great Wenchuan Earthquake. Adv Struct Eng 2012; 15(1):139–53.

[19] Li HW, Wang WD. Application of ABAQUS secondary development in the finite element analysis of concrete-filled steel tubular structures. Journal of Building Structures 2013;34(Sup1):354–8 (in Chinese).

[20] Han LH. Concrete-filled steel tubular structures-theory and practice. 2nd ed. Beijing: China Science Press; 2007(in Chinese).

[21] Hu HT, Huang CS, Wu MH. Nonlinear analysis of axially loaded concrete-filled tube columns with confinement effect. J Struct Eng ASCE 2003;129(10):1322–9.

[22] The United States General Services Administration. Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects. Washington, D. C: The U.S. General Services Administration; 2003.