SEISMIC BEHAVIOR ANALYSIS OF STEEL-CONCRETE COMPOSITE FRAME STRUCTURE SYSTEMS

LIU Jingbo[1] and LIU Yangbing[2]

[1] *Professor, Dept. of Civil Engineering, Tsinghua University, Beijing. China*
[2] *Candidate for Ph. D., Dept. of Civil Engineering, Tsinghua University, Beijing. China*
Email: liujb@tsinghua.edu.cn, liu-yb05@mails.tsinghua.edu.cn

ABSTRACT :

To investigate seismic behavior of steel-concrete composite frame structures, based on researches of composite beams and connections between beams and columns, a 4 poly-line plastic hinge mode is suggested for the inelastic analysis of composite frames. And CL-CFST (composite beam-concrete filled square tubular column), SL-CFST (steel beam-concrete filled square tubular column), CL-ETRC (composite beam-equivalent stiffness RC column), SL-ETRC (steel beam-equivalent stiffness RC column) and RC frame structures are separately created. Then the mode analysis, the response spectrum and the inelastic analysis under rare earthquakes are carried out. The analysis results illustrate that, compared with the SL-CFST frame, the integral stiffness of the CL-CFST frame is enhanced; natural periods are shortened; the top deflection of and the storey-drift-angle are decreased, after considering the composite effect of RC floor slabs. But because of the increase of the stiffness of composite beams, the integral stiffness of the structure and the linear stiffness ratio of beam and column are also changed, which probably makes not only the column damaged heavily but also the weak section transferred. And compared with the CL-RC frame, under rare earthquakes, RC columns can not satisfy the demand of “no collapsing with strong earthquake”. And after increasing dimensions and reinforcement ratios of columns, the plastic hinge is also found at the end of columns. According to the above comparisons, as a whole, the CFST frame structure has qualified aseismic performance. But the effect which composite frame beams have on structural seismic behavior should be considered comprehensively.

KEYWORDS: concrete filled square tubular(CFST), composite beam(CL), frame , aseismic performance

1. INTRODUCTION

With the development of design methods and experimental researches for CFST columns and composite beams, the steel-concrete composite frame has been widely used in multi-story and high-rise buildings. The composite structure has the advantages of fabrication in factories, assemble construction, short construction period, high bearing force and good ductility etc[1-3]. But up to now, researches on the total structural behavior especially in dynamic properties are lesser. In literature [3], the inter-storey hysteretic model based on results of the quasi-static test and the dynamic test is suggested to perform inelastic dynamic analysis of structures. And results of the test and computing show that steel-concrete composite structure has well aseismic behavior. Based on results of experiments, a 3 poly-line restoring force model of stiffness degradation for the composite structure is developed to carry out the inelastic time history analysis of a 3-storeyed structure, in literature [4].

Based on the research of CFST columns[5-6] and composite beams[6-8], a 4 poly-line moment-curvature (M-φ) model is suggested for the inelastic analysis of composite beams and the plastic hinge model for the CFST column is created. Then the inelastic model of the composite frame structure is constructed. By using SAP2000 software, the mode analysis, the response spectrum and the inelastic analysis under server earthquake for the 15-storeyed CL-CFST, SL-CFST, CL-ETRC, SL-ETRC and RC frame structures are performed. By comparing the analysis results, the aseismic performance of the steel-concrete composite frame is studied.

2. STRUCTUAL CONFIGURATION AND MATERIAL

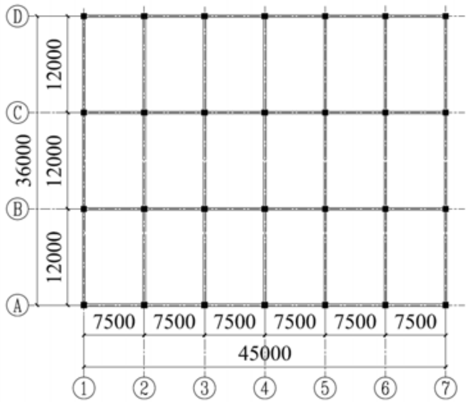


Figure 1 Structural plane layout (unit: mm)

For the five buildings, the storey height is 3.6m, and the total height is 54m. The layout of the plane is illustrated in Fig. 1. Frame elements are used to simulate columns and beams, types and dimensions of which are shown in Table 2.1. The concrete of CFST columns is C40, and the steel tube uses Q345. The concrete of RC columns and beams is C40, and the reinforcement is HRB400. The steel beam uses Q235. The floor and the roof use 140mm thick RC slab simulated by shell elements, the concrete of which is C30 and the reinforcement in which is HRB335. The nominal values of dead load and live load on slabs are respectively 4.5kN/m^2 and 2.0kN/m^2.

Table 2.1 Sections of columns and beams (unit: mm)

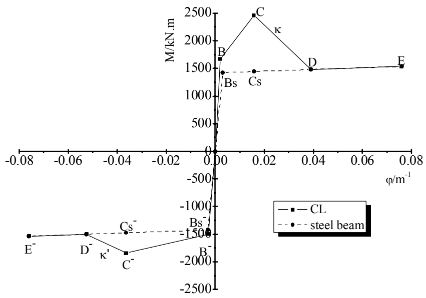
frame type		RC	CL-CFST	SL-CFST	SL-ETRC	CL-ETRC
column type		RC	CFST	CFST	RC	RC
column section	1F-5F	850×850	600×20	600×20	681×681	681×681
	6F-15F	750×750	600×15	600×15	640×640	640×640
beam type		RC	CL	H steel	H steel	CL
beam section	transverse	350×900	750×300×13×24	750×300×13×24	750×300×13×24	750×300×13×24
	longitudinal	350×800	700×300×13×24	700×300×13×24	700×300×13×24	700×300×13×24
slab thickness(mm)		140	140	140	140	140

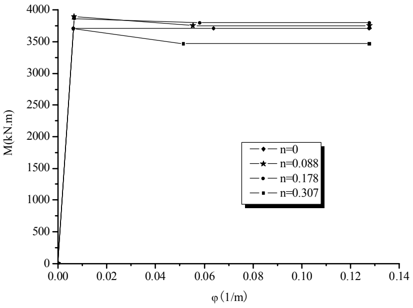
3. PLASTIC HINGE MODEL OF COLUMN AND BEAM

Yielding and post-yielding behavior can be modeled by using hinges model in SAP2000. Moment (M3) hinge is used to simulate the plastic hinge of the CL. Based on researches of restoring force models of composite beams and behavior of CFST column connections in Ref. [8-10], a 4 poly-line moment-curvature (M-φ) model is suggested for the inelastic analysis of composite beams. The M-φ curves for crossbeams are illustrated in Fig. 2, and the bilinear model is used to simulate the corresponding steel beam. In Fig. 2, point Bs, Bs$^-$ and point E, E$^-$ respectively represent the yielding and ultimate capacity of the steel beam in positive and negative moment region, point B, B$^-$ and point C, C$^-$ dividedly denote the yielding and the ultimate capacity of the composite beam in positive and negative moment region, letter κ and κ′ denote the negative slope of the segment C-D and C$^-$-D$^-$, point D and D$^-$ are intersection between negative portion of the CL and the strengthen portion of the steel beam, and point Cs and Cs$^-$ in the M-φ curve of the steel beam are defined of the same curvature as point C and C$^-$. The method of getting values of point B, B$^-$, C, and C$^-$ can refer to Ref. [8, 10]. As shown in Fig. 2, stiffness and bearing capacity of the composite beam are different in positive and negative moment region.

Coupled P-M2-M3 hinge in SAP2000 which yields based on the interaction of axial force and bi-axial bending moments at hinge location is used to model CFST columns. And the method of gaining M-φ and P-M2-M3 interaction curves is available in the Ref. [5]. M-φ curves considering different axial compressive forces and P-M2-M3 interaction curves for the 1F-5F CFST column section are attained, illustrated in Fig. 3 and Fig. 4.

The parameter n is axial compression ratio.





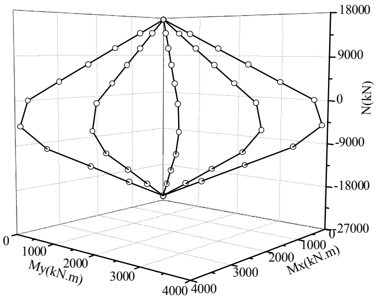


Figure 2 4 poly-line *M-φ* curve of CL Figure 3 *M-φ* curves of CFST column Figure 4 P-M_2-M_3 curves

4. ANALYSIS OF ASEISMIC PERFORMANCE

To study and compare the dynamic behavior and aseismic performance of the five structures, mode analysis, response spectrum analysis and time history analysis are performed.

4.1. Mode Analysis

Table 4.1 gives the first five periods of the five structures. And the first three modes of structures are all translation in Y direction, translation in X direction and rotation.

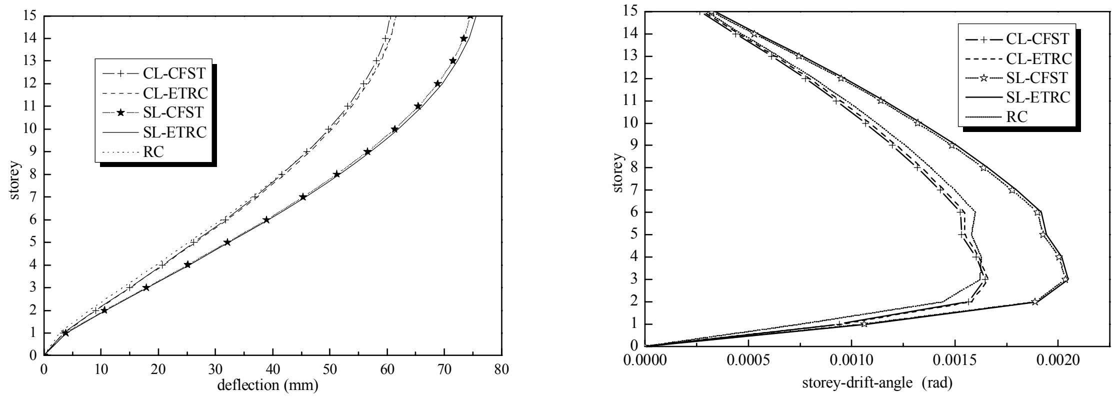
Table 4.1 Comparisons of first ten periods of frames (unit: s)

mode	RC frame	CL-CFST frame	SL-CFST frame	SL-ETRC frame	CL-ETRC frame
1	2.204	2.193	2.466	2.480	2.210
2	1.969	1.969	2.098	2.114	1.987
3	1.876	1.966	2.041	2.051	1.977
4	0.726	0.726	0.813	0.818	0.732
5	0.652	0.655	0.696	0.702	0.661

From Table 4.1, the first period of the SL-CFST frame with no consideration of the effect of floor slabs is larger about 12% than that of the CL-CFST frame considering the effect of floor slabs. The RC frame designed is used to be compared its aseismic ability with the CL-CFST frame. So in the course of design, the dynamic properties of the two frames are similar, as shown in the table.

4.2. Seismic Response Spectrum Analysis

The mode analysis method is applied to calculate seismic responses of structures under the horizontal earthquake. And to guarantee the computing accuracy, the first thirty modes are chosen. Structures are all in 8 regions of earthquake intensity category and belong to the first seismic group. And the categorization of building sites is the second kind. According to GB 50011－2001[11], the earthquake affecting coefficient is 0.16 and the design characteristic periods is 0.35s. To compare conveniently, the damping ratio of the other four structures is 0.035, except the damping ratio of the RC frame is 0.05. The direction of the inputting design earthquake motion is along the Y axis (weak direction). Fig. 5 gives the comparisons of structural deflections and storey-drift-angles in every floor for five structures. From Fig. 5, under the same earthquake load, deflections and storey-drift-angles of the CL-CFST frame are the smallest; those of the CL-ETRC frame are smaller; and those of the SL-ETRC frame are the largest among the five structures and a little larger than those of the SL-CFST frame. For the CL-CFST frame, the top storey deflection is 60.715mm, and the maximum storey-drift-angle in the 3rd floor is 1/607. For the SL-CFST frame, the top storey deflection is 74.733mm, and the maximum storey-drift-angle in the 3rd floor is 1/491. Deformations of the two frames both satisfy the demand of the storey-drift-angle limit value 1/300 in the CECS 154:2004[12]. From the above analysis, considering the composite effect of floor slabs, the structural integral stiffness is enhanced, while structural deformations are reduced. For frames with equivalent stiffness RC columns, the maximum storey-drift-angles are both in the 3rd floor; and deformations under the frequent earthquake are all a little larger than those of the frames with CFST columns, which is because the mass of RC columns is heavier than that of CFST columns.



(a) Deflection in every floor (b) Storey-drift-angle

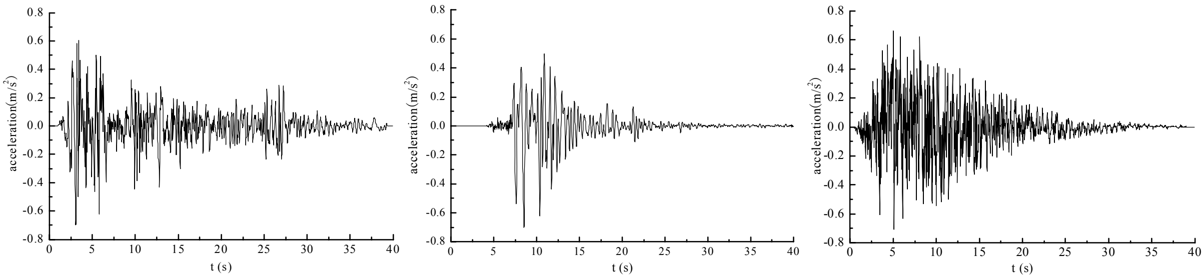
Figure 5 Deflections and storey-drift-angles for frames under earthquake loads in Y direction

From Fig. 5, under the same load, the RC frame has the similar deformation as the CL-CFST frame. But the member sections of the RC frame are much larger than those of the CL-CFST frame, which is easy to lead to "stout beam corpulent column".

The bearing capacity of structures and members is checked under the frequency earthquake load. Results show that some sections of RC columns with the equivalent stiffness to CFST columns can not satisfy the demand of bearing capacity. So in the time history analysis, for the CL-ETRC frame and the SL-ETRC frame, the dimension of columns in 1F-5F is enlarged to 800mm×800mm, and in 6F-15F is enlarged to 700mm×700mm.

4.3. Inelastic Time History Response Analysis

El Centro wave, Kobe wave and Beijing artificial wave are used to perform elastic and inelastic time history analysis. The PGA of each wave is respectively scaled to 0.7 m/s^2 and 4m/s^2. The acceleration time history records for the elastic time history analysis are given in Fig. 6. And the time step is 0.02s, the mode damping ratio is 0.035. During the inelastic analysis, Rayleigh damping is used, and the mode damping ratio is 5%.The acceleration time history is input along Y direction (weak direction) for the elastic and inelastic time history analysis.



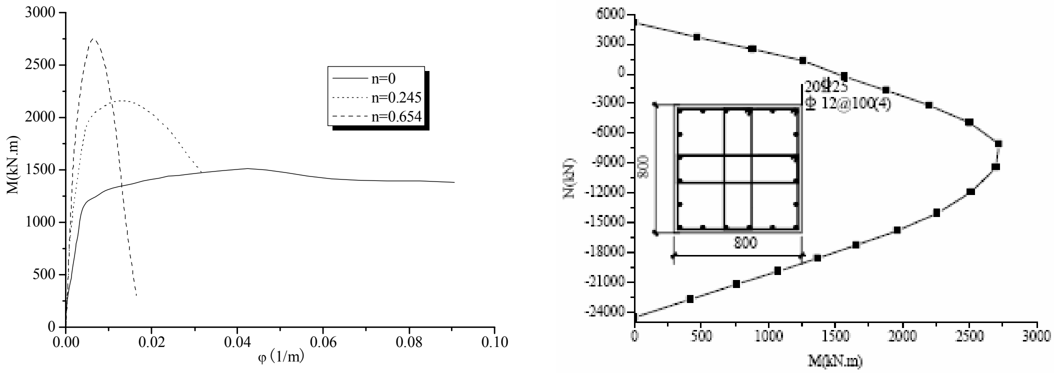
(a) El Centro wave (b) Kobe wave (c) Beijing artificial wave

Figure 6 Seismic acceleration time history for elastic time history analysis

According to results of response spectrum analysis, for the SL-ETRC frame and CL-ETRC frame, to satisfy the demand of the section bearing capacity under the frequent earthquake, the dimension of columns in 1F-5F is enlarged to 800mm×800mm, and in 6F-15F floor is enlarged to 700mm×700mm. To be compared with the

CFST frames under the rare earthquake, the longitudinal reinforcement ratio of RC columns in the ground storey firstly uses the ratio of the frequent earthquake 1.53％ to check the inelastic deformation. And then the reinforcement ratio is augmented to 3.58％ to carry out the inelastic analysis. Response-2000 is used to analyze the column section. Fig. 7 only gives the RC column section with reinforcement ratio 1.53％ from the first storey of the building and the corresponding plastic hinge model with different axial compression ratio *n*.

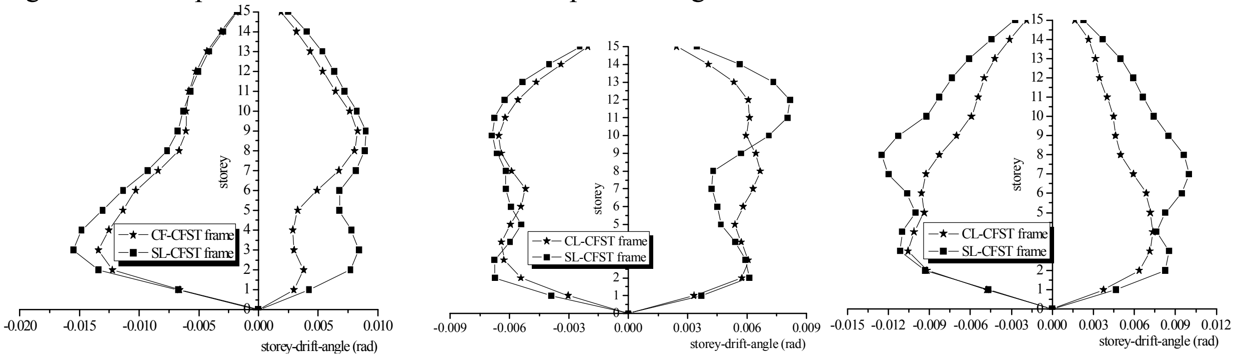
The plastic hinge models of composite beams and steel beams are respectively the 4 poly-line model and the bilinear model, as shown in Fig. 2. And the plastic hinge model of CFST column is shown in Fig. 3 and Fig. 4. The bilinear hysteretic model is used for columns and beams. Using the plastic hinge models, the inelastic models for the CL-CFST frame, SL-CFST frame, CL-RC frame and SL-RC frame are respectively constructed. Then inelastic response-history analyses for the four frames are performed under the action of El Centro wave, Kobe wave, Beijing artificial wave. Before the inelastic time history analysis, the representative seismic gravity loads are put on the structures to determine the initial internal force of the structures.



(a) M-φ curves of RC column with different n (b) M-N interaction curves

Figure 7 Plastic hinge model and section of RC column

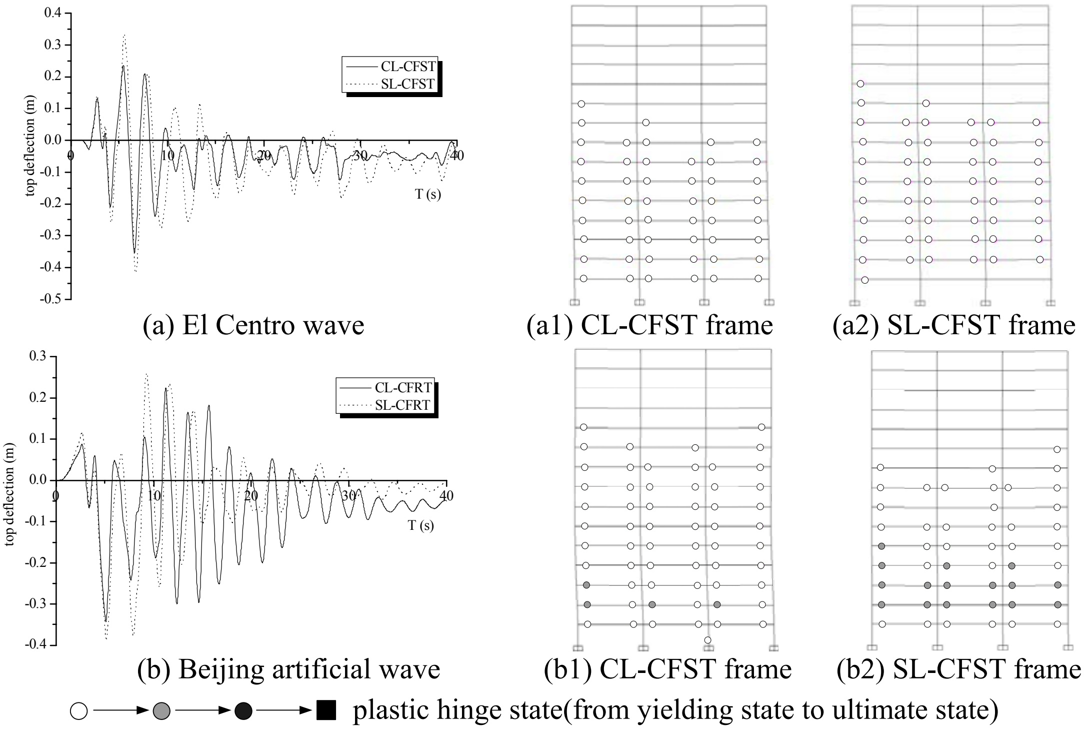
The results of the inelastic response-history analysis give not only the enveloping values of storey-drift-angles but also the distributions of the plastic hinge. Firstly, inelastic responses considering the composite effect of floor slabs are compared. The enveloping values of the storey-drift-angle under three earthquake waves for the CL-CFST frame and the SL-CFST frame are illustrated in Fig. 8. As a whole, the enveloping values of the CL-CFST frame considering the composite effect of floor slabs are greater than those of the SL-CFST frame, and change more uniformly along the height of the structure. Moreover values of the maximum storey-drift-angle under the action of the three earthquake waves all can satisfy the inelastic limit value 1/50. Fig. 9 gives the top deflection time history and plastic hinge distributions of frames in No. 4 axis under El Centro wave and Beijing artificial wave. In Fig. 9, different circle and rectangular marks denote the developing states of the plastic hinge, hollow circular marks represent the yielding state of the plastic hinge and the solid rectangular marks represent the ultimate state of the plastic hinge.



(a) El Centro wave (b) Kobe wave (c) Beijing artificial wave

Figure 8 Enveloping values of the storey-drift-angle under the rare earthquake

There are no plastic hinges in the two ends of CFST columns under El Centro wave, in Fig. 9. The quantity of the plastic hinge for the two frames is about equivalent, and the plastic hinge position of the SL-CFST frame is higher than that of the CL-CFST frame. Furthermore top deflections and residual deformations at most time points are greater than those of the CL-CFST frame. Under Beijing artificial wave, the plastic hinge appears in the bottom end of the middle column for the CL-CFST frame. And the plastic hinge states of steel beams develop more than those of composite beams, while the plastic hinge quantity of steel beams is less than that of composite beams. From Fig. 9 (b), in the first 10s, the displacement of the SL-CFST frame is greater than that of the CL-CFST frame, while the residual deformation is smaller than that of the CL-CFST frame.



(a) El Centro wave (a1) CL-CFST frame (a2) SL-CFST frame

(b) Beijing artificial wave (b1) CL-CFST frame (b2) SL-CFST frame

Figure 9 Inelastic displacement time history in Y direction and distribution of plastic hinge

To compare aseismic performance between CFST column frames and RC column frames, plastic hinge distributions of the CL-RC frame under the rare earthquake are illustrated in Fig. 10. Fig. 10(a) and 10(b) give the plastic hinge distribution of the CL-RC frame at original reinforcement ratios. Fig. 10(c) and 10(d) give the plastic hinge distribution with increased reinforcement ratios at which the ultimate axial compressive force and the moment bearing capacity under certain axial compressive loads of RC columns are higher than those of CFST columns. Under El Centro wave, with original reinforcement ratios, the plastic hinge firstly appears in the bottom of the column, and with the development of the plastic hinge, calculation isn't convergent at 6.5s, when plastic hinges are found in both ends of the right fringe column from 1F to 3F and plastic hinges in the bottom end of both sides columns both get to the ultimate state (in Fig. 10(a)). After increasing longitude reinforcement ratios, the plastic hinge firstly appears in the right end of the 2 floor beam, and the final plastic hinge distribution at 40s is shown in Fig. 10(c). Under Beijing artificial wave, at the smaller longitude reinforcement ratio, the plastic hinge firstly appears in the bottom end of the right column and then appears in beams, the structure reaches the failure state at 6.2s, when plastic hinges in the bottom end of the first floor columns all get to the complete failure states. When the reinforcement ratio is increased, the plastic hinge firstly appears in the end of beams. And at 4.7s, plastic hinges appear in the bottom end of middle columns. At 14.46s, the calculation isn't convergent, and the distribution of the plastic hinge at this moment is shown in Fig. 10(d).

From comparisons between the CFST column frame and the RC column frame, the failure model of the CFST column frame is beam hinge model; while the failure model of the RC column frame is column hinge model.

Moreover, the plastic hinge distribution of the CFST column frame is more extensive and uniform, and the failure model is global mechanism and has the better energy-absorbing capability.

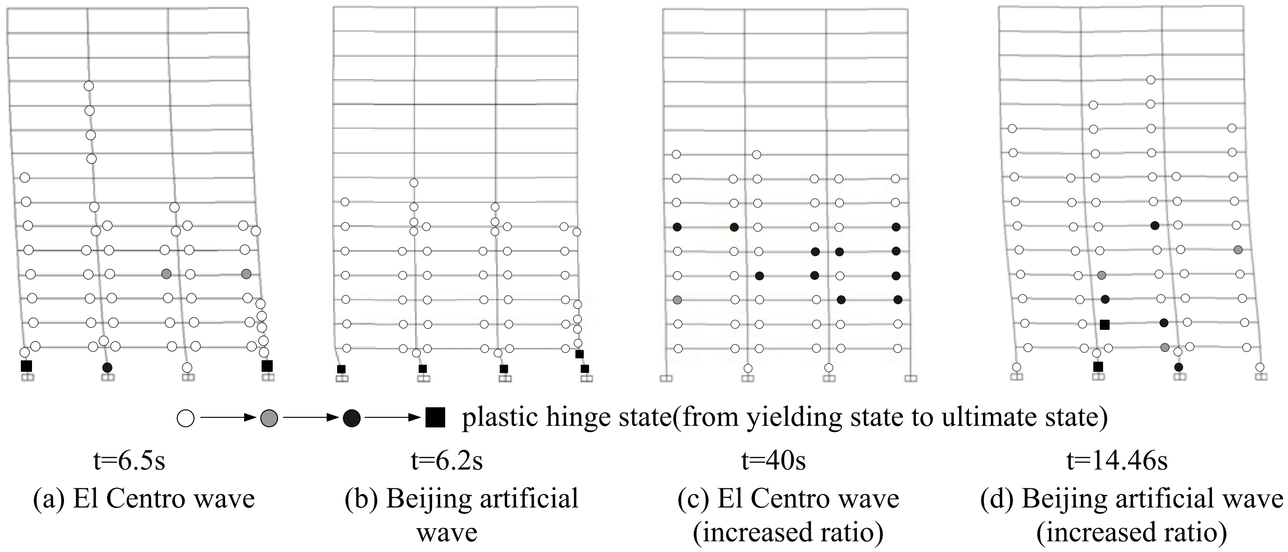


Figure 10 Plastic hinge distribution of the CL-RC column frame

5. CONCLUSIONS

From the above contrastive analyses of the aseismic performance for the steel-concrete composite frames, conclusions can be gained as follows:

1. Results of mode analysis and spectrum analysis show that, for CL frames with considering the composite effect of floor slabs, the structural stiffness is enhanced, the periods are shorten and the structural top deflection and the maximum storey-drift-angle are reduced by about 18%. For the RC column which has the equivalent stiffness to the CFST column frames, the top deflection and the maximum storey-drift-angle are both greater than those of the corresponding CFST frames, and the ultimate bearing capacity of RC columns is smaller than that of CFST columns. Therefore the axial compression ratio and moment bearing capacity of RC columns can not satisfy the demand of the seismic check. So sections of RC columns are enlarged to meet the demand of the bearing capacity.
2. Member dimensions of frames are determined by deformations. And compared with the RC frame, under the same elastic deformations, member dimensions of the CL-CFST frame are much smaller than those of the RC frame. Furthermore with the same member dimension, the CL-CFST frame can achieve a higher height and a greater span.
3. The inelastic time history analysis shows that as a whole, the enveloping value of the storey-drift-angles is smaller than that of the SL-CFST frame. And furthermore it changes more uniformly along the height of the building. But the inelastic response depends much on the selected earthquake waves. And because of the increase of the stiffness for composite beams, the linear stiffness ratio of beam to column and the global structural stiffness both change, which probably makes not only the column damaged heavily but also the weak section transferred. So the effect which the increase stiffness of composite beams has on the structural aseismic performance should be considered comprehensively. Compared with the CL-ETRC frame, the frame with the original RC column can not resist the rare earthquake. Although the structural stiffness and the ultimate bearing capacity can be enhanced by increasing the section dimension and the reinforcement ratio of columns, the self-weight and the earthquake action are accordingly increased. And furthermore the ductility of RC columns is inferior to that of CFST columns, so RC columns are apt to be injured under the rare earthquake, which doesn't make for resisting earthquake loads.

ACKNOWLEDGEMENTS

This paper is supported by the National Natural Science Foundation of China (No. 50438020).This support is gratefully acknowledged. The authors have benefited from discussions with several schoolfellows, especially Guo Bing (Master of Tsinghua University).

REFERENCES

[1] Nie Jianguo; Fan Jiansheng. (2006). The development and prospect of generalized composite structures. *Journal of Building Structures*.**27:6**, 1-8.
[2] Li Zhu, Guo Xiuhua, Zhang Wenfang. (2004). Simplified analysis of concrete filled columns analysis of concrete filled tubular columns in the structural analysis of national grand theatre of China. *Engineering Mechanics*.**21:4**, 34-38.
[3] Zong Zhouhong, Lin Dongxin, Fang Zhenzheng etc. (2002). Experimental research on seismic behavior of a two-story concrete filled steel tubular composite frame. *Journal of Building Structures*.**23:2**, 27-35.
[4] Jiang Lizhong, Cao Hua, Yu Zhiwu. (2005). Elastic-plastic earthquake response of steel-concrete composite frame structure. *Journal of Railwy Science and Engineering*.**2:3**, 1-8.
[5] Han Linhai, Yang Youfu. (2004). Modern Technology of Concrete Filled Steel Tubular Structures. *China Building and Industry Press*, Beijing, China.
[6] Nie Jingo, Qin Kai, Xiao yan. (2006). Push-over analysis of the seismic behavior of a concrete-filled rectangular tubular frame structure. *Tsinghua Science and Technology*.**11:1**, 124-130.
[7] Nie Jianguo, Liu Ming, Ye Lieping. (2004). Steel-concrete composite structure. *China Building and Industry Press*, Beijing, China.
[8] Chen Ge. (2005). Experiments and research on steel-concrete composite frames [Dissertation]. Tsinghua University, Beijing, China.
[9] Nie Jianguo, Qin Kai, Xiao Yan. (2006). Experimental investigation and nonlinear finite element analysis on the behavior of concrete-filled square steel tubular column connections. *Engineering Mechanics*.**23:11,** 99-115.
[10] Nie Jianguo, Yu zhouliang, Yuan Yansheng, et al. (1999). Research on restoring force model of composite steel concrete beams. *Journal of Tsinghua University* (*Science and Technology*).**39:6**, 121-123.
[11] Ministry of the People's Republic of China. (2002). Code for seismic design of buildings (GB 50011-2001). *China Building and Industry Press*, Beijing, China.
[12] CECS. (2004). Technical specification for structures with concrete-filled rectangular steel tube members (CECS 154:2004). *China Planning Press*, Beijing, China.