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Seismic analysis of CFST frames considering the effect of the floor slab

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Abstract. This paper describes the refined 3-D finite element (FE) modeling of composite frames composed of concrete-filled steel tubular (CFST) columns and steel-concrete composite beams based on the test to get a better understanding of the seismic behavior of the steel-concrete composite frames. A number of material nonlinearities and contact nonlinearities, as well as geometry nonlinearities, were taken into account. The elastoplastic behavior, as well as fracture and post-fracture behavior, of the FE models were in good agreement with those of the specimens. Besides, the beam and panel zone deformation of the analysis models fitted well with the corresponding deformation of the specimens. Parametric studies were conducted based on the refined finite elememt (FE) model. The analyzed parameters include slab width, slab thickness, shear connection degree and axial force ratio. The influences of these parameters, together with the presence of transverse beam, on the seismic behavior of the composite frame were studied. And some advices for the corresponding seismic design provisions of composite structures were proposed.

Key words: CFST; frame; seismic; floor slab; finite element.

1. Introduction

In the early 1990s, performance-based seismic design has attracted worldwide attention since it leads to more reasonable and economical designs in earthquake engineering and provides accurate evaluation of the seismic behavior of the structural system. There are two commonly used methods, namely the experimental method and finite element (FE) method, to assess the seismic behavior of the composite frame. The experimental method is close to reality, but it cost much more labor and resources. On the contrary, the FE method acts as a cost-saving choice to study the seismic behavior of the composite frame. There are a number of reports on numerical simulation of the composite structure. According to the level of modeling details, these models were categorized into three groups:

(1) The plastic hinge model that concentrates the inelastic behavior by employing a moment-rotation hysteretic rule at the end of the composite component. duPlessis and Daniels (1973) carried out a number of tests on composite beam-column joints to study the positive moment capacity of the composite beam and proposed a bilinear model for the positive flexural behavior of the composite beam. Lee (1987) expanded the de Plessis's model to include the negative flexural behavior and

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incorporated a hysteretic rule that included the behavior of the opening and closure of slab concrete crack. Kim and Engelhardt (2005) modified the Lee's model by replacing the straight reloading line with curve and introducing strength degradation to more accurately simulate the behavior of the composite beams. The plastic hinge model is simple and easy to be adopted in the practical application. However, the determination of the parameters for this model is not straightforward. The interface slip between the two materials was considered approximately by multiplying a reduction factor for strength or stiffness. This model also had difficulty in dealing with problems of axial-moment (P-M) interaction.

(2) The fiber section model that distributes the inelasticity into some sections along the member and discretizes the section into a number of fibers for which the material constitutive laws are based on uniaxial stress-strain relationships. The fiber model was capable of modeling the bond slip between the concrete and the steel, which is a major issue of interest for the research of the composite structure. From a formulation point of view, this model was subdivided into three types. They were the displacement-based method (Hajjar *et al.* 1998), force-based method (Salari *et al.* 1998) and mixed method (Ayoub and Filippou 2000). The fiber section model is more rational than the plastic hinge model since it is materially-based and is capable of analyzing structures comprised of different materials loaded with axial force and biaxial moment. Although the fiber section model provided an elegant solution to the problem of bond slip, it is suffered from the problem of shear-lag for composite beam since it assumes a certain equivalent section based on the concept of effective width of concrete slab.

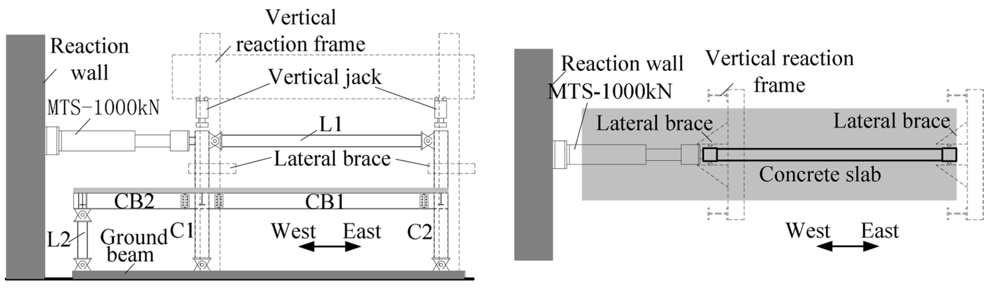
In general, the shear and the torsion are not considered in the plastic hinge model and the fiber section model. And both models have trouble in tracing the local effects such as the bulking or the fracture in the composite structure.

(3) The refined FE model that incorporates different element types such as the beam element, the shell element, the solid element etc. to trace the overall and local behavior of the composite systems. Sebastian *et al.* (2000) analyzed the steel-concrete composite beam based on a refined nonlinear FE model. The shear lag in the concrete slab was modeled by the shell element. Zhou *et al.* (2007) used DIANA to study the seismic behavior of steel-concrete composite frames. The steel beams and columns were modeled by beam elements and the concrete slab was modeled by shell elements. The headed shear studs that connected the steel beam with the concrete slab were modeled by spring elements. Based on the Zhou's model, Zhao *et al.* (2010) introduced a deformable panel zone to the FE model of the composite frame to model the flexible beam-column joint. They concluded that their model was feasible for the analysis of elastoplastic behavior of composite frames.

However, the existing models seldom considered the bearing effect of the column flange to the slab concrete near the beam-column joint. The effect of slab width, slab thickness, axial load etc, on the seismic behavior of the composite frame is still need to be studied. This paper provides a refined nonlinear FE modeling of composite frames to study the seismic behavior of composite frames using MSC. Marc (2005). The validity of the analysis models is confirmed by comparing of both global and local behavior of the composite frame with those of the test results. Parametric analyses were conducted to study the effects of slab width, slab thickness, shear connection degree and axial load ratio on the seismic behavior of composite frames.

2. Outline of the experiment

Nie *et al.* (2011) tested a full-scale CFST composite frame with floor slab subjected to cyclic load. The test specimen was designed according to Chinese code GB50011 (2003) which is similar to AISC



（a）elevation （b）plan

Fig. 1 Setup of the test specimen

(2005a). The setup of test specimen is shown in Fig. 1. The dimension of the specimen is 9 m in the horizontal direction and 3.1 m in the vertical direction. For details of the tested composite frame, refer to (Nie *et al.* 2011).

3. Finite element modeling

The steel plate of the steel beam column was modeled by the shell element to represent its real dimension. The reinforced concrete slab is modeled by the layered shell element so that both the concrete and the rebar in the slab were considered., In the FE model, the concrete slab below the top of the rib is omitted because the rib of the profiled steel sheet is perpendicular to the axis of the beam. The brick element was used to model the concrete in the steel tube.

The perfect bond hypothesis is used between the interface of the steel tube and the core concrete because the interface slip hardly affected the mechanic behavior of the CFST column (Hajjar *et al.* 1998). Point to point springs that model the shear stud and the bolt were used. The force-slip relationship of the shear stud was proposed by Ollgaard *et al.* (1971), as shown in Fig. 2. The force-slip relationship of the bolt was proposed by Chen *et al.* (2004), as shown in Fig. 3.

For steel model, the bilinear constitutive laws were used to model the steel plates in the FE model. Fracture property was introduced to model the heat affected zone (HAZ) of the beam bottom flange.

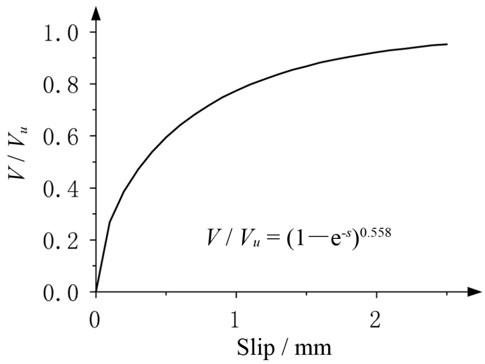


Fig. 2 Shear-slip curve for studs

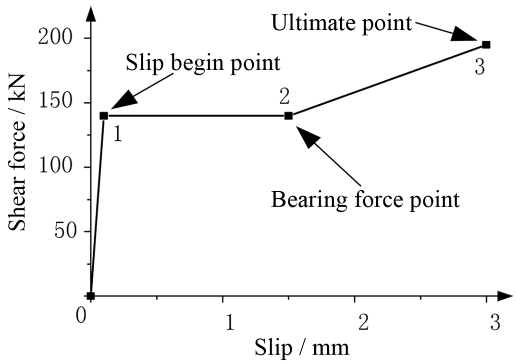


Fig. 3 Force-slip curve for bolts

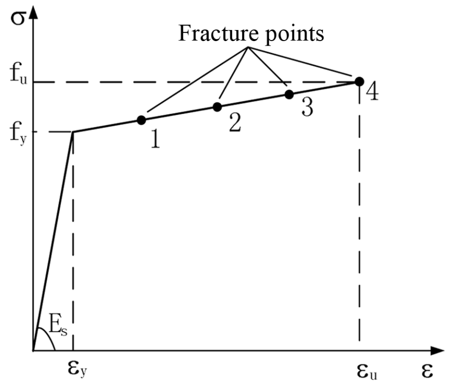


Fig. 4 Bilinear stress-strain curve for the steel plate

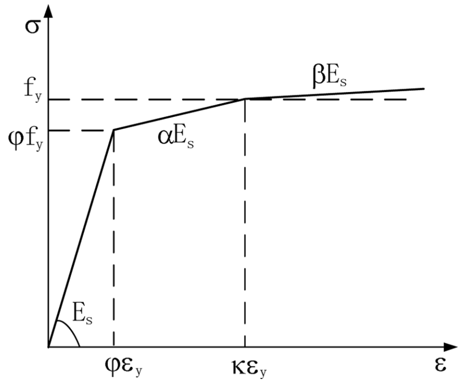


Fig. 5 Tilinear stress-strain curve for the rebar

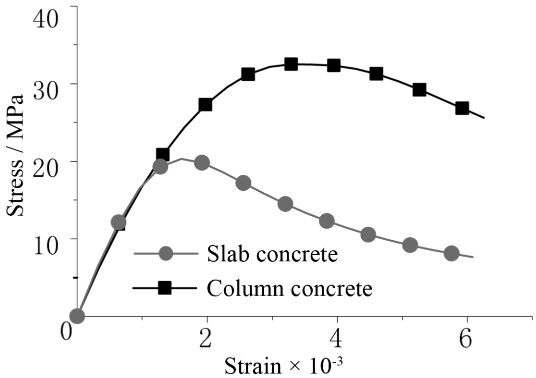


Fig. 6 Stress–strain model used for the concrete

The values of the fracture stress were assumed as the 4 quarter points in the hardening line, as shown in Fig. 4. For rebars in the slab, the trilinear stress–strain relation (Wang and Teng 2008) was assumed, as shown in Fig. 5. For the slab concrete, The Sargin's model (1971) was assumed. For the concrete in the steel tube, Han's model (2010) was assumed to consider the confinement of the steel tube, as shown in Fig. 6.

The FE model together with its constraints is shown in Fig. 7. For the horizontal load, both the monotonic loading and the cyclic loading were applied. The object displacement of the monotonic loading was 150 mm. The object displacements of the reverse cyclic loading are similar with those in the test, except for only one cycle at each amplitude. Geometric nonlinearity was considered based on a total Lagrange description. The Newton-Raphson method was used to deal with the nonlinear equilibrium iteration problems.

4. Validation of the FE model

Fig. 8(a) shows the comparison of the FE load–displacement results of the monotonic loading with the test results. Both the elastic stiffness and the yield strength of the FE models were in good

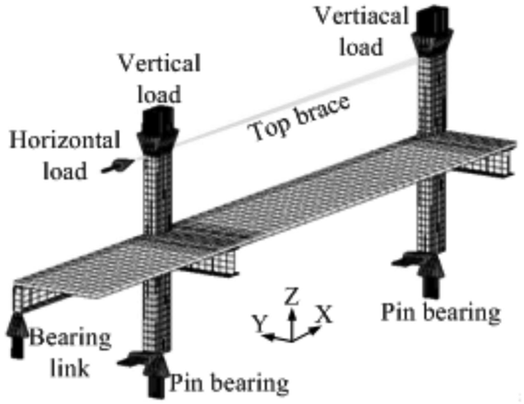
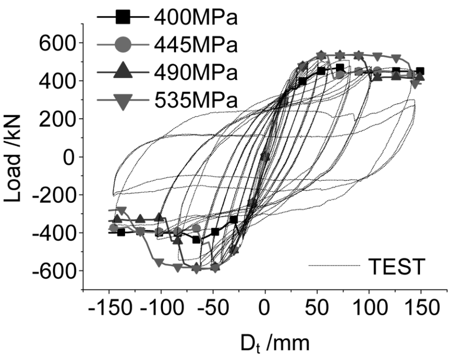


Fig. 7 FE model and boundary conditions

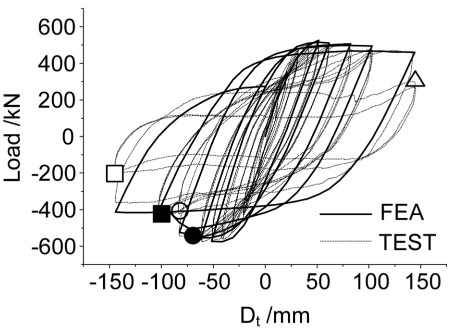
agreement with those of the test. The upward triangle marked line with the fracture stress of 490 MPa fitted best with the test results. Therefore, the fracture stress was set as 490 MPa in the cyclic analysis for simplicity. This assumption is based on the fact that with the same welding process and material properties of the base metal and weld metal, the status of the residual stress and initial imperfections was almost identical in the fractured bottom flanges. Thus it is reasonable to assume that the fracture strength of all bottom flanges is equal.

Fig. 8(b) shows the comparison of the FE load–displacement results of the cyclic loading with the test results. The triangle, square and round symbols represent the failure of the beam bottom flange at the west side of interior joint, the east side of interior joint and the exterior joint respectively. The hollow and solid symbols stand for the fracture in the test specimen and the FE model respectively. The strength and stiffness of the FE model fitted well with those of the test. And the sequence of the fracture point of the FE model agreed reasonably with that of the test.

Fig. 9 shows the comparison of the local deformations of the FE model with those of the test. R_D is the displacement contributed by the component divided by the total displacement. The subscript p and b represent the panel zone and the beam. respectively. The round and square symbols are the local deformations of the beam and the panel zone, respectively. FE results agreed reasonably with test



(a) Monotonic loading



(b) Cyclic loading

Fig. 8 Comparison of FE and test load-displacement curves

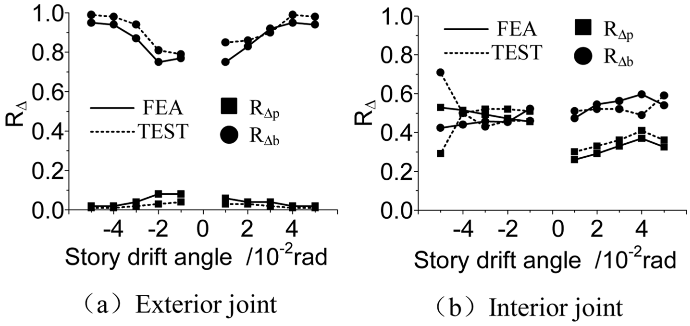


Fig. 9 comparison of the local deformation

results at both exterior joint and interior joint. These phenomenons conformed to the observation and measurement in the experiment, which provided additional evidence for the validity of the FE model.

5. Parametric analysis

Based on the FE model, parametric analyses were conducted to get more information about the seismic behavior of the CFST composite frame. Parameters analyzed were listed in Table 1. Besides, the effect of transverse beam on the frame performance is also studied. The FE model of the test specimen was taken as the standard model. As the value of one parameter varied, the other parameters remained consistent with the standard model to make the influence clear.

5.1 Slab width

Because of the effect of shear lag, the whole flange cannot be taken in computing the flexure capacity of the beam. Effective width of the concrete slab is an interesting issue for the analysis of composite frame. Research on strength of composite beam (du Plessis and Daniels 1973, Lee 1987) suggested that the effective slab width under positive bending be assumed as the width of the bearing column flange while that under negative bending be assumed as the effective width defined in AISC (2005a) regardless of the concrete in tension. That is, when subjected to negative moment, only the longitudinal rebars in the effective width of the slab are considered in the composite beam.

As defined in AISC (2005a), the effective slab width b_{eff} for each side of the beam centerline is assumed as the minimum of $L/8$, b_i, in which L is the beam span center to center of supports, b_i is the geometric width which is taken as the half distance from the beam centerline to the centerline of the adjacent beam, except that at a free edge, is the distance from the beam centerline to the edge of the

Table 1 Variables for parametric study of composite frame

Variable	Value			
Slab width (mm)	330	600	2330	4000
Slab thickness (mm)	70	90	110	130
Shear connection degree	0.5	1	1.5	2
Axial ratio	0	.0.2	0.4	0.6

slab. Similar definition of effective width for composite beam could be found in Eurocode 4 (2004) where the distance between the centers of the outstand shear studs is added.

The above definitions of effective width are based on the tests of the simply supported composite beam subjected to positive bending. However, for horizontally loaded composite frame, the boundary condition of the composite beam is different. For the composite beam in the laterally loaded frame, the definition of effective width of concrete slab is proposed in Eurocode 8 (2003) where for elastic analysis of the structure, the value of effective width is assumed as 0.075 L for positive bending and 0.1 L for negative bending. And for evaluation of plastic moment resistance, the value is taken as high as twice those suggested for elastic analysis.

In the present study, the column flange width is 330 mm. 0.1 L is 600 mm. The width of the slab in the test is 2330 mm. The value of 4000 mm is an enough wide value more than the distance from the beam centerline to the centerline of the adjacent beam. The focus is on how broad the concrete slab width should be taken in the test and the FE model to accurately consider its influence on the performance of laterally loaded composite frame.

Fig. 10 shows a comparison of FE results for different slab width. The dotted line is the analysis results of composite frame without floor slab. The stiffness, yield strength and post-fracture strength of the composite frame grew with the increase of the slab width. The composite frame with slab width of 330 mm had an obvious increase of stiffness compared to bare steel beam frame while the increase of strength was not so apparent. The distinction in terms of stiffness and strength between the composite frame with slab width of 2330 mm and 4000 mm is quite small. Therefore, the concrete slab width in the test is broad enough to investigate its effect to the behavior of the plan composite frame. Further analyses of slab width showed that the concrete slab width be taken as at least $L/4$ in the 3D analysis model or in the test specimen of composite frame to precisely calculate the impact of composite action of RC slab. Otherwise the stiffness and strength of the model would be underestimated and the interaction between beam, column and panel zone is different from that of the realistic composite frame. Note that the effective slab width is not a constant along the beam span by considering shear lag effect (Bursi *et al.* 2005). The value of effective width for accurate 1D FE analysis of composite frame depends on many parameters, such as frame layout, loading conditions, slab geometry *et al.* and is beyond the discussion of this study.

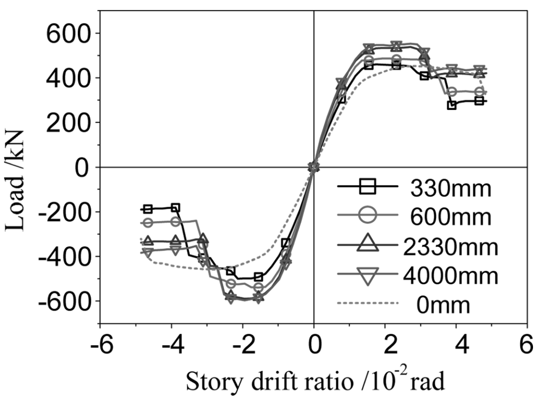


Fig. 10 FE results for different slab width

5.2 Slab thickness

AISC (2005b) required that the distance from the maximum concrete compression fiber to the plastic neutral axis should not exceed

$$Y_{PNA} = \frac{d}{1+\left(\frac{1700f_y}{E_s}\right)}$$

Where d is the depth of the composite beam, as shown in Fig. 11. f_y is the yield stress of the steel beam. E_s is the elastic modulus of the steel beam. The equation is derived assuming that the strain in the steel at the extreme fiber ε_a is at least five times the tensile yield strain ε_y prior to concrete crushing at strain ε_{cu} equal to 0.003.

Eurocode 8 (2003) suggest a upper limit for the ratio x/d, where x is the distance from the top fiber of concrete in compression to the plastic neutral axis. The purpose is to achieve ductility requirement in theplastic hinge. It is assumed that ε_a equals 6.8 ε_y for the structures designed with high ductility (DCH).

Fig. 12 shows the comparison of FE results for different slab width. These values of slab thickness are commonly used in practice. The purpose is to verify the slab effect on the ductility of composite frame. In general, the stiffness and strength increased slightly with the growth of slab thickness. The story drift ratio at flange fracture changed little with the increase of slab thickness. It is concluded that slab thickness had little effect on the ductility of the composite frame.

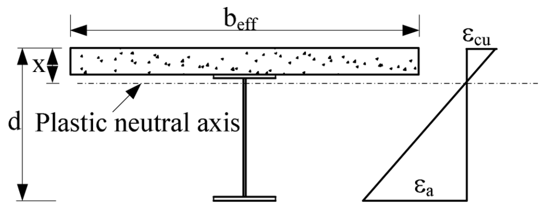


Fig. 11 ductility analysis of composite beam section under positive moment

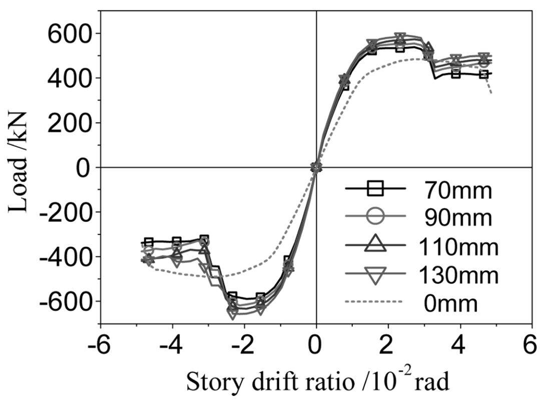


Fig. 12 FE results for different slab thickness

5.3 Shear connection

The partial shear connection is widely used for composite beams. The amount of shear connectors has a critical impact on the structural and economical behavior of composite constructions. During the past decades, a number of studies were conducted on the seismic behavior of shear connection and its influence on the seismic behavior of composite system. Most of them were based on the push-out specimen or vertically loaded composite beams. The effect of partial shear connection on the seismic performance of the composite frame was rarely studied. Salari and Spacone (2001) found that the behavior of composite frames remained for degrees of connection greater than 0.2. Bursi *et al.* (2005) got similar conclusion that the composite frames with shear connection degree as low as 0.4 do as well as the frames with full shear connection degree under rare earthquake loading. Fig. 13 shows the comparison of FE results for different degree of shear connection. Compared with the companion frames without concrete slab, the composite frames with concrete slab have substantial increase of initial stiffness even at a low shear connection degree of 0.25. However, the strength of composite frame with shear connection degree of 0.25 reduced greatly compared with that with shear connection degree of 0.5. The composite frame with shear connection degree of 0.5 performs as well as that with full shear connection.

5.4 Axial force ratio

Fig. 14 shows the comparison of FE results for different axial force ratio. The ultimate resistance of the composite frame declined with the growth of the axial force ratio because of the P-Δ effect. However, the story drift ratio when the beam bottom flange fractured grew with the increase of axial load. The main reason for the delay of the fracture is that an increased deformation in panel zone caused by the increase of column axial load had reduced the rotation demand at beam plastic hinge, which resulted in a relieved strain and delayed the fracture at beam flange. It was also noted that the post-peak behavior shifted from hardening line to softening line with the increase of axial force ratio from zero to 0.6.

5.5 Effect of transverse beam

Eurocode 8 (2003) proposes the possible force transfer mechanisms between the transverse beam and the concrete slab, as shown in Fig. 15. F_{tr} is the force transfer by the transverse beam. The rebars within

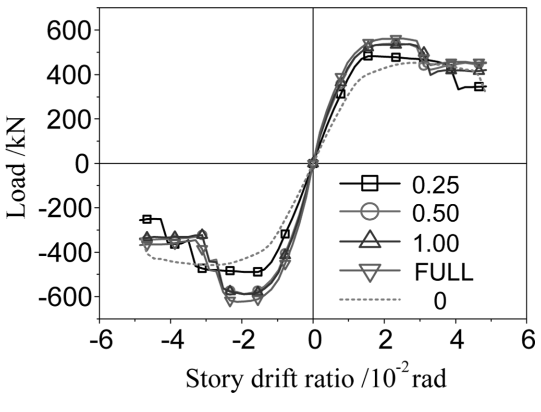


Fig. 13 FE results with for shear connection ratio

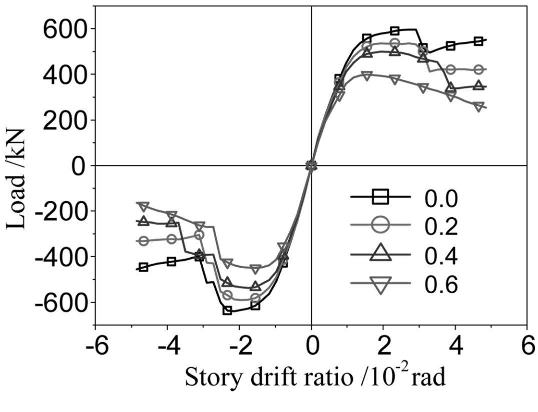


Fig. 14 FE results for different axial force ratio

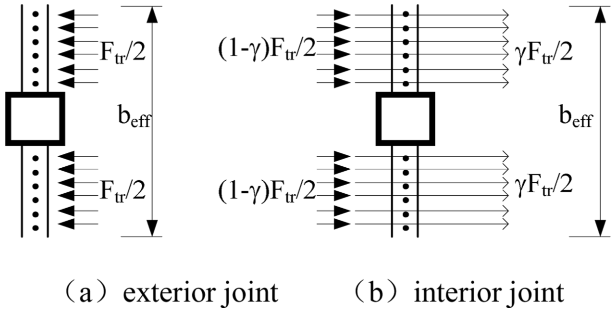


Fig. 15 force transfer mechanism at the transverse beam

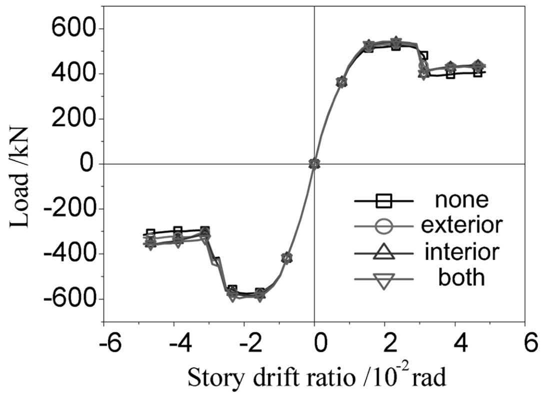


Fig. 16 FE results for different types of transverse beams

the effective width b_{eff} in tension are anchored to the concrete slab on the transverse beam for the negative bending. The concrete slab within the effective width in compression is braced with the transverse beam for positive bending.

Fig. 16 shows the comparison of FE results for different types of transverse beams The square, round, upward triangle and downward triangle marked line represent the results of composite frame without transverse beam, with only exterior transverse beam, with only interior transverse beam and with both interior and exterior transverse beams. It is apparent that these results are almost identical. Therefore, the presence of transverse beam has little effect on the seismic behavior of the composite frame.

6. Conclusions

Based on three-dimensional nonlinear FE model of CFST composite frame verified by the test result, parametric analysis was conducted to investigate the effects of slab width, slab thickness, shear connection degree, axial force ratio, panel web thickness and transverse beam on the seismic behavior of the composite frame. The main conclusions can be summarized as follows:

1. The FE results agreed well with that of the test results globally and locally, especially for the progressive fracture of the beam flange occurred in the test, as this phenomenon was frequently reported in the post-earthquake research.

2. The increase of the slab width has comparatively large effect on the frame behavior when the slab width is less than one fourth of the beam span. However, when the slab width is greater than one fourth of the beam span, the increase of slab width has negligible effect on the seismic behavior of the composite frame because of the shear lag effect. It is suggested that the concrete slab width should be taken at least as $L/4$ in the 3D FE model to accurately evaluate the impact of RC slab composite action on the seismic behavior of the composite frame.

3. The ductility of the composite frame is not sensitive to the slab thickness in the commonly used range.

4. The lower limit of the shear connection degree is suggested to be taken as 0.5 because the composite frames with that value of shear connection degree perform as well as those with full shear connection.

5. The axial force has a significant effect on the frame performance, especially at large story drift due to P-Δ effect. The post-peak behavior changed from hardening line to softening line with the increase of axial force ratio from zero to 0.6.

6. The presence of transverse beam has negligible effect on the seismic behavior of the composite frame in terms of stiffness and strength.

There are few reports on the seismic analysis of composite frames with CFST column and composite beams. This paper showed that FE analysis a helpful tool to expand the information on seismic behavior of composite frame. Additional numerical studies are needed to model the deteriorated bond slip behavior of the shear stud under reversed cyclic loading.

Acknowledgments

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