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3D finite element modelling of composite connection of RCS frame subjected to cyclic loading

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Abstract. Composite special moment frame is one of the systems that are utilized in areas with low to high seismicity to deal with earthquake forces. Composite moment frames are composed of reinforced concrete columns (RC) and steel beams (S); therefore, the connection region is a combination of steel and concrete materials. In current study, a three dimensional finite element model of composite connections is developed. These connections are used in special composite moment frame, between reinforced concrete columns and steel beams (RCS). Finite element model is discussed as a most reliable and low cost method versus experimental procedures. Based on a tested connection model by Cheng and Chen (2005), the finite element model has been developed under cyclic loading and is verified with experimental results. A good agreement between finite element model and experimental results was observed. The connection configuration contains Face Bearing Plates (FBPs), Steel Band Plates (SBPs) enveloping around the RC column just above and below the steel beam. Longitudinal column bars pass through the connection with square ties around them. The finite element model represented a stable response up to the first cycles equal to 4.0% drift, with moderately pinched hysteresis loops and then showed a significant buckling in upper flange of beam, as the in test model.

Keywords: composite structure; RCS connection; panel zone; cyclic loading; plastic work

1. Introduction

One of the several types of hybrid systems that accepted as a cost-effective alternative to traditional steel or reinforced concrete frames for seismic design, are the composite special moment frames consisting of steel beams and reinforced concrete columns, so called RCS moment frames. This system can be used in areas with high seismicity to deal with earthquake forces. In this system, the interaction of two types of steel and concrete materials should be considered in the connection region. In 1986, Griffis represented that using the concrete columns reduces construction costs and increases lateral stiffness of frame in comparison with steel columns. Also

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using composite connections eliminates the use of welding at joint region (or panel zone) and ultimately the connection reliability increases.

Various types of connections are shown in Fig. 1. These beam-column connections are generally divided into two categories: beam-through-type and column-through-type. Beams that continuously pass through column panel zones (beam-through type)behave in a ductile manner under seismic loading.Useofcontinuouscolumnsinbuildingswillenhanceconstruction stage, but seismic capacity and damping of energy in these joints must be investigated, and ultimately proper executive details should be presented (Cheng and Chen 2005).

In 1989, Deierlein *et al*. (1989) and Sheikh *et al*. (1989) at Texas University tested fifteen beam-through-type connections without slabs. According to their research, two significant failure modes in RCS frames were detected: panel shear failure and bearing failure, as shown in Fig. 2. Kanno (1993) tested several RCS connections without slab. In this research, main parameters including tie details in the panel zone, column axial load, and bearing strength of the concrete were considered. Test results represented that the seismic capacity of RCS systems was not less than

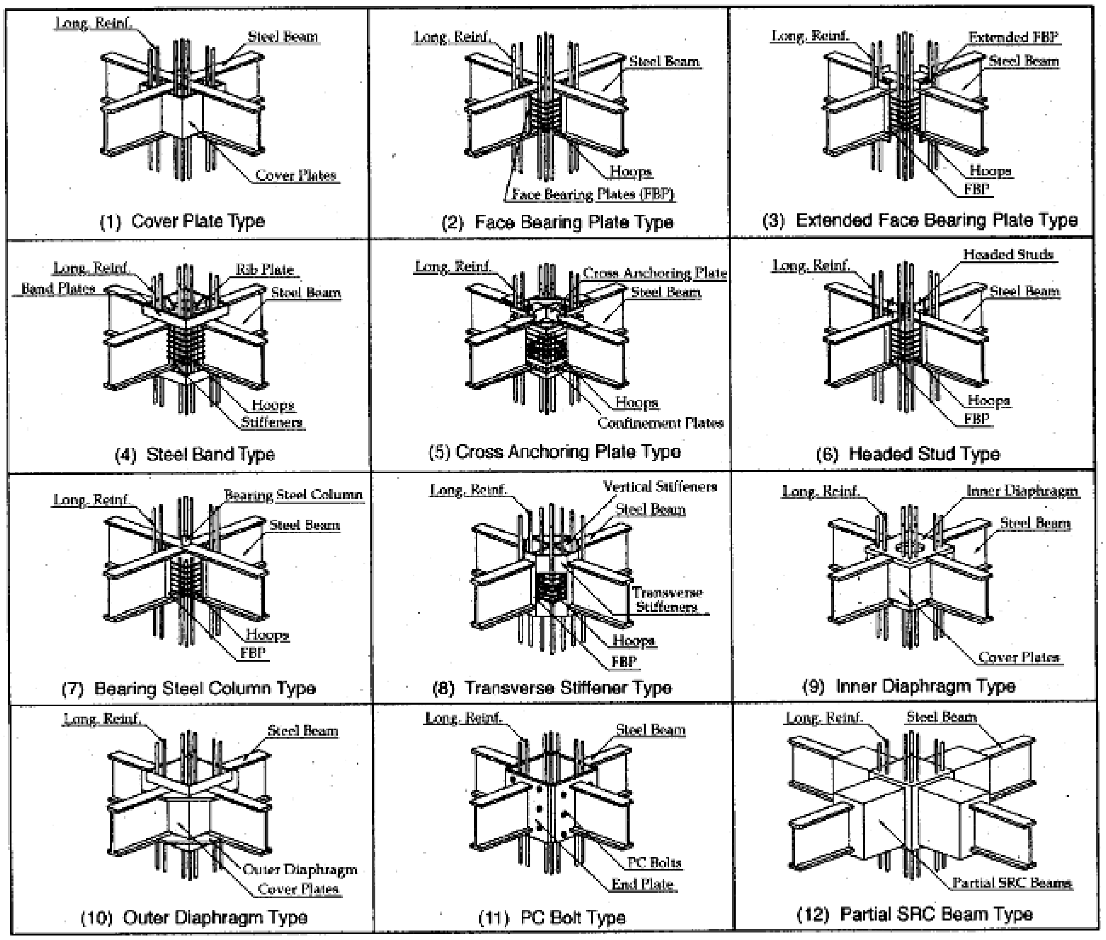


Fig. 1 The various types of composite connections

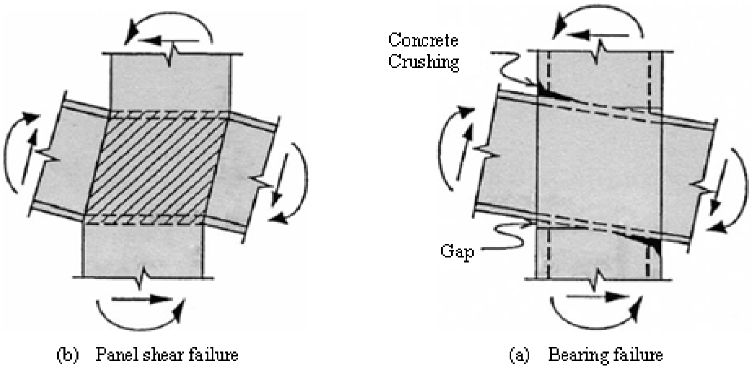


Fig. 2 The failure mechanism of an inner panel

that of ordinary reinforced concrete or steel structures. Since 1997, some researchers have investigated the behavior of composite moment frames and composite connections in the US and Japan, such as research of Parra-Montesinos and Wight (2000), Bugeja *et al*. (2000) and others.

Yu *et al*. (2000) and Liu and Astaneh-Asl (2000) have tested several composite connections of steel beam and RC column for investigating composite effects of the steel beam and floor slab. They demonstrated that composite effects might vary with the types of connection, distribution of shear studs, slab thickness, and amount of reinforcing bars in the slab.

In 2002, a Taiwan–US research cooperation group proposed a test of full-scale three story–three-bay in-plane RCS frame. Before the frame test, the seismic behavior of the beam–column connections should be clarified. According to the literature results, beam through-type connections may have improved seismic performance when compared with the column-through-type ones. Six full-scale beam-through-type composite beam–column sub-structures were designed and tested to act as a component test for the design of a three-story three-bay in-plane RCS frame. Parameters considered were composite effects of the slab, tie details in the panel zone, effects of the transverse beam, and the loading procedure.

Based on the full-scale three bays and three stories tested model by Cordova (2005) (and also Cordova and Deierlein 2005), Cheng and Chen (2005) tested six specimens of RCS connections with and without slab along cross-beam in the orthogonal direction, in Taiwan. Test results demonstrated that all specimens have a ductile behavior and plastic hinges formed in the beam flanges and web near the column face. In addition, the test performance showed that cross-beams and the configuration of ties in the panel zone had only a marginal effect on the shear transfer in the panel zone due to the strong column and weak beam design for all specimens.

Noguchi and Uchida (2004) investigated finite element model of composite moment frames with reinforced concrete columns and steel beams by three-dimensional nonlinear FEM program that has been developed by Uchida (1994, 1998). Axial loads and monotonic lateral loads were applied to the model in the analysis.

Braconi *et al*. (2007) have been modeled the exterior and interior beam-to-column joints for partial-strength composite steel–concrete moment-resisting frames to predict the inelastic monotonic response of these joints. The model was found to reproduce very accurately the observed response and experimental measurements for the joints configurations (Braconi *et al*. 2007).

A three-dimensional fiber-based beam finite-element model was developed by Tort and Hajjar (2010) to investigate the nonlinear response of composite frames consisting of rectangular concrete-filled steel tube (RCFT) beam-columns and steel framing subjected to static and dynamic loads. The results of this study show that the mixed finite-element formulation provides an efficient procedure for tracking detailed local constitutive response in the context of analyzing complete 3D composite structures, as may be done for assessing high-level performance objectives or seismic demand within a reliability based performance-based design framework (Tort and Hajjar 2010).

In 2010, Wang *et al*. (2010) have developed a nonlinear finite element model of the steel-concrete composite beam to concrete-filled steel tubular column joints. According to the literature results, the finite element model can accuratcely predict the overall seismic behavior and the inelastic performance of composite joints, and can be used to conduct the nonlinear parameter analysis of the joints hysteretic behavior (Wang *et al*. 2010).

An accurate finite element model using ABAQUS have been developed by Mirza and Uy (2010) to study the behaviour of shear connectors in push tests incorporating the time-dependent behaviour of concrete. The results of investigation show the finite element modeling is useful to predict the overall composite beam behaviour (Mirza and Uy 2010).

In addition, Shen and Qiang (2010) modeled the nonlinear finite element model of exterior RC-column to Steel-beam connection.

Various two-dimensional fiber-based RCS composite moment frames finite-element models were developed by Azar *et al*. (2013) to investigate the seismic performance of RCS composite moment frames subjected to seismic loads.

As mentioned above, a great deal of research has conducted on RCS and composite joint behavior with highly costs of testing. However, finite element modeling needs to be further investigated and implemented by software for predicting the behavior of RCS connections. This method is able to predict the behavior of RCS connections with a little cost. Therefore, results of simulation of a three dimensional finite element model of RCS connection for predicting the behavior of connection due to cyclic loading could be used in the investigation of seismic behavior of RCS connections and modeling of connections in the whole RCS frames in the analysis programs such as Opensees. The model, based on laboratory experiments has been developed in the National University of Kaohsiung in Taiwan by Cheng and Chen (2005), and have been compared the intended results with experimental results. The main objective of this paper was to develop a reliable nonlinear three-dimensional finite element model to investigate the behavior of the composite concrete column and steel beam connection under cyclic loading. For this purpose, the finite element code LUSAS was employed. Consequently, the aims of the present study were as follows:

- Finite element modeling of hysteretic behavior of RCS connections under cyclic loading,
- Evaluating the reliability of finite element method in studying seismic behavior of RCS connections,
- Comparing finite element and experimental model results,
- Investigation of plastic work at joint panel of RCS connection

2. Description of tested model by Cheng and Chen (2005)

The latest research of Cheng and Chen (2005) has been considered for verifying the finite element results. This model was considered as interior connection of a RCS moment frame with the start and end of beams and columns as inflection point of base frame (Cordova and Deierlein 2005). In the sub-structural test, as shown in Fig. 3, the specimen ICSC/INUC, as described by Cheng and Chen 2005, has the same dimensions, with the steel beam H596 × 199 × 10 × 15 mm and 650 × 650 mm columns reinforced with 12 #11 longitudinal bars, representing beam–column connection of the first floor of the in-plane frame. In labeling this specimen, the first character, I, represents the interior column connection. The second character, C/N, represents whether the connection is with or without a cross-beam in the orthogonal direction. The third character represents the shape of the ties reinforced in the panel zone. U or Square-shaped ties were used in the panel zone of the models. The fourth character, C, distinguishes cyclic loading protocol.

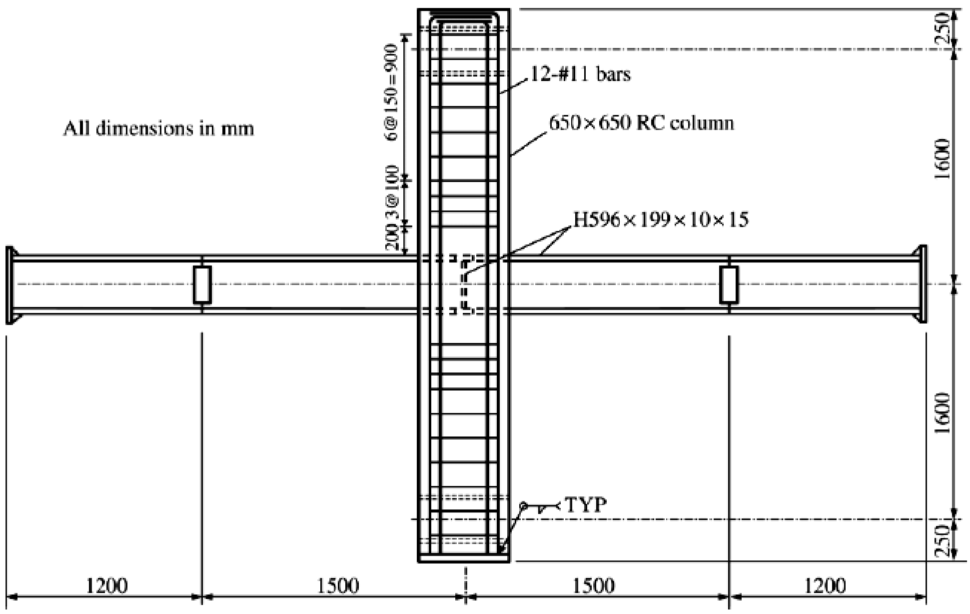


Fig. 3 Detail of the beam-column joint in tested model (Cheng and Chen 2005)

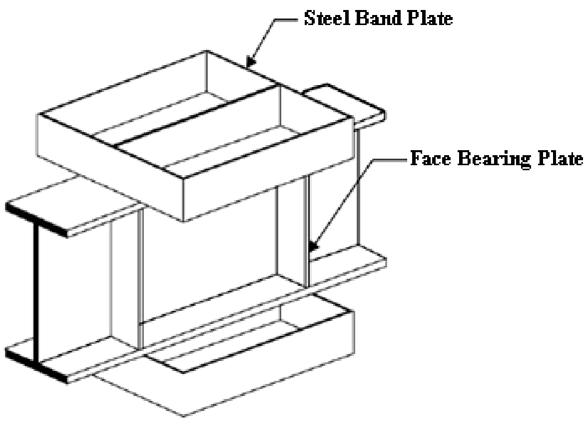


Fig. 4 Two reinforcing techniques in the panel zone of the beam-through type connections

According to the research conducted by Kanno and Deierlein (2000), the panel zone of beam-through-type connections can be divided into two elements: inner and outer elements. Failure modes in the inner element can be panel shear yielding or bearing failure of the column concrete, while failure modes in the outer element may be bond failure of the longitudinal reinforcement or panel shear yielding. To prevent these premature failures of connection, two retrofit techniques were applied, as p shown in Fig. 4. To prevent bearing failure of the column concrete near steel beam surface, band plates (BP) were embedded around the column. To enhance the shear transfer in the panel zone, face-bearing plates (FBP) were fillet welded to the beams at the column face. Also square shape of ties was used in the column and anel zone, as shown in Fig. 5.

3. Three dimensional finite element model

Development of computer processors has made advances in data analysis and more complex problem solving in engineering. This has created software that is more powerful in static and dynamic nonlinear analysis in the engineering field and has reduced the analysis time.

Based on the combination of two materials (steel and concrete), used in composite connections, the problem of convergence occurs in these connections. In order to overcome this problem, researchers often apply simplifications to the FE model. The simplifications can reduce the difficulty of creating the FE model and help to overcome the convergence problem. However, this reduces the reliability of the analysis result. Since the failure modes of the connection vary, a FE analysis, describing the actual damage and failure of the connection, is necessary.

As noted earlier, the main objective of this paper is to develop a reliable nonlinear three-dimensional finite element model in order to investigate the behavior of the composite RCS connections, concrete column and steel beam, due to cyclic or seismic loading.

3.1 General

In the FE model, the whole of the specimen INUC has been modelled based on experimental reports. Six components including steel beams, longitudinal rebars, ties, concrete columns, and region of connection with inner and outer panel zone have been modeled. Since constraints and interactions between the components also had a great influence on the analysis results, they were applied. Both geometric and material nonlinearity have been included in the finite element analysis.

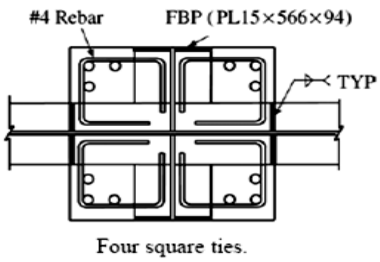


Fig. 5 Shapes of ties in the panel zone

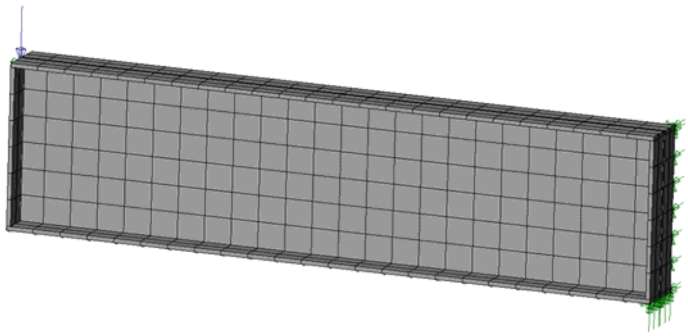


Fig. 6 Cantilever beam with shell and solid elements and monotonically increasing load

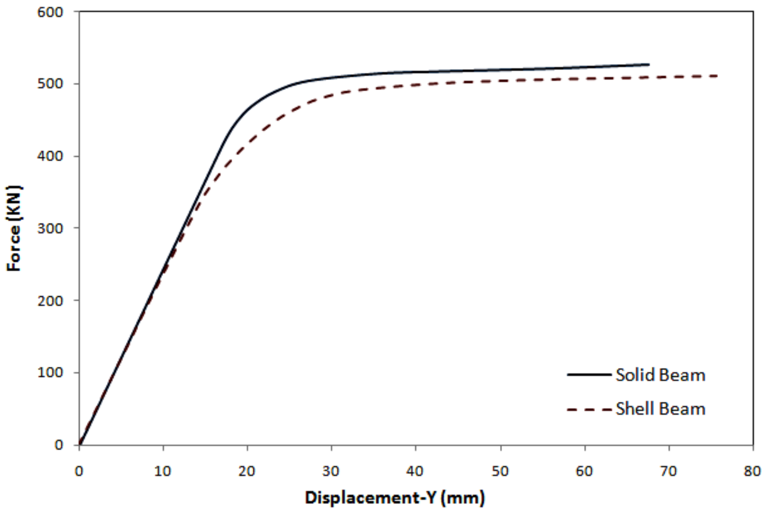


Fig. 7 Comparison of modeling results of shell and solid cantilever beam

3.2 Finite element type and mesh

3.2.1 Type of mesh

For selecting shell or solid elements in the modeling of steel part of finite element model, two types of cantilever beam model with shell and solid elements and monotonic loading were used, as shown in Fig. 6. In addition, the rigid support at one of beam-ends was used. Comparison of modelling results, as shown in Fig. 7, has demonstrated that both elements for the main modeling could be used. However, given that the element type for the concrete part was solid, so to avoid problems related to compatible degrees of freedom at boundary elements and possibly the lack of convergence, the solid elements were used for the steel parts of model such as flange and web of beams, FBP and bond plates of connection.

3.2.2 Mesh size

To investigate mesh size effects in the modeling, a cantilever beam was modeled, as shown in Fig. 6. A nonlinear static analysis using a monotonic loading at the beam end has been carried out.

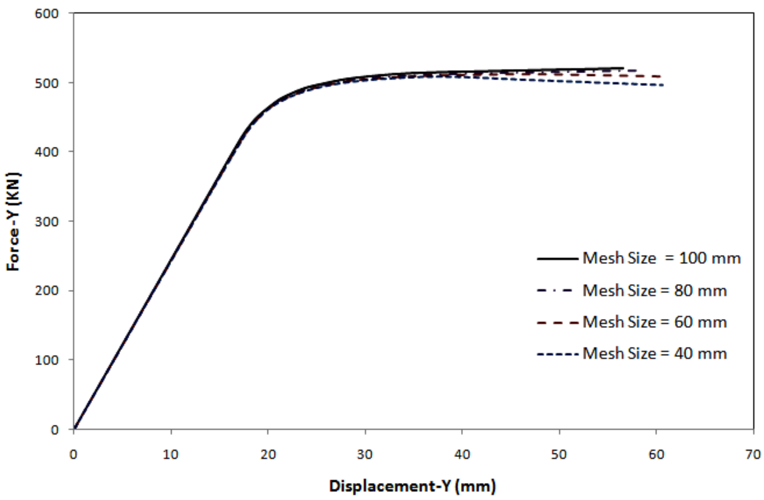


Fig. 8 Comparison of modeling results of mesh size sensitivity of cantilever beam

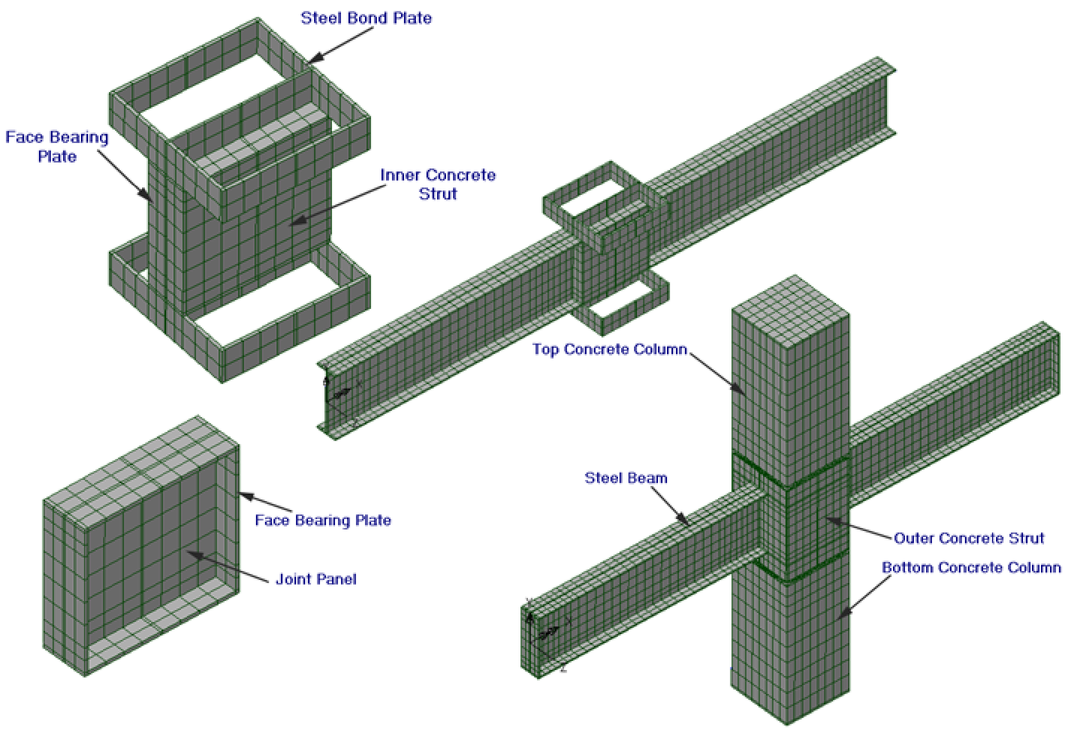


Fig. 9 Three-dimensional finite element model

Four mesh sizes 40, 60, 80 and 100 mm and aspect ratio equal or less than 3 was considered. Fig. 8 shows the results of analyses. The results show that the model has slight sensitivity to mesh size. To avoid increased costs and reduced analysis time, the mesh size of 60 mm and smaller was used at regions with concentrated stress distribution or a high stress.

3.2.3 Mesh type and size for modelling

The steel beams and concrete columns were modeled using solid element HX8M. It is 8-node brick element and was used for nonlinear analysis including large displacements, large rotations (Update Lagrangian geometric nonlinearity) and plasticity, as shown in Fig. 9. The ties and rebars were modeled using bar element, BRS2. The usual mesh size was 100 mm at web, along the beam, while the smallest mesh size was about 10 mm at thickness of web. The aspect ratio range of the used meshes was from one to three.

3.3 Material models

3.3.1 Steel beam, FBP plates and rebars

The Multi-linear elastic-plastic model named as Stress Potential with Von Mises type was used for the web and flanges of steel beam, and the bi-linear elastic-plastic model was used for rebar and ties (Fig. 10). The mechanical behavior for both tension and compression was assumed identical. The yield and ultimate tensile strength of the steel beam was obtained initially from the test information's of Cheng and Chen (2005), and then was approximated into the true stress and plastic strain with appropriate input format as shown in Fig. 10. For the elastic part of the stress–strain curve, the value of the Young's modulus and the Poisson's ratio of the steel were considered 2.06×10^5 N/mm^2 and 0.3 respectively.

3.3.2 Concrete column

In order to overcome the convergence problem after cracking, a Drucker-Prager elasto-plastic model was adopted to simulate the concrete material. The Drucker-Prager elasto-plastic model may be used to represent the ductile behaviour of materials exhibiting volumetric plastic strain (for example, granular materials such as concrete, rock and soils). The model incorporates isotropic hardening.

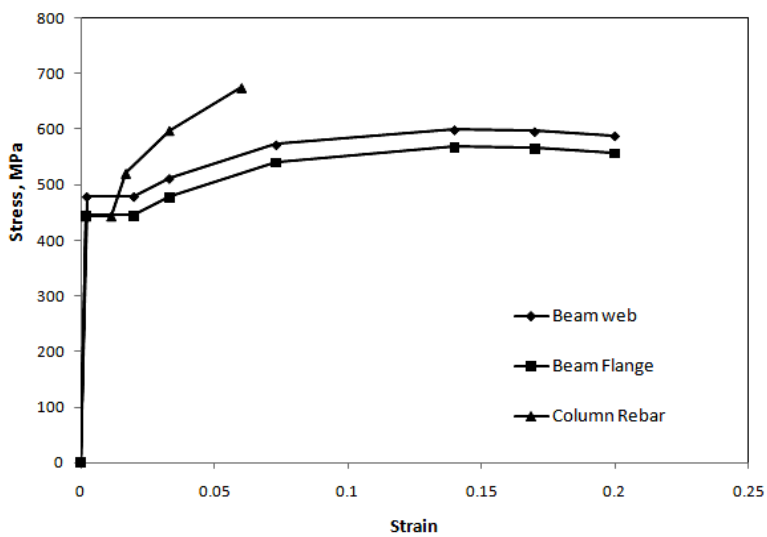


Fig. 10 Multi-linear elastic-plastic material models for steel parts of 3D finite element model

Based on test information's of Cheng and Chen (2005),in this model the concrete strain and Young's modulus corresponding to f'_c, were obtained using the following equations

$$\varepsilon_0 = 0.001648 + 0.0000165\, f'_c \tag{1}$$

$$E = 5000\sqrt{f'_c} \tag{2}$$

where f'_c and ε_0 are the concrete compressive strength (Mpa) and the concrete strain corresponding to f'_c, respectively. In the elastic part of the stress–strain curve, by using Eq. (2) and experimental measured concrete compressive strength equal to 42 Mpa, the value of the Young's modulus and the Poisson's ratio of the concrete are obtained 3.24×10^4 Mpa and 0.2, respectively. For the plastic part, it is required to identify the yield Surface. In the Drucker-Prager material model, it is needed to identify the initial cohesion, initial friction angle, slope of yield stress and plastic strain based on the compressive and tension strength of concrete.

Eq. (3) shows the yielding surface of Drucker-Prager elasto-plastic model. By using Eqs. (4)-(6), the initial cohesion (C) and initial friction angle (ϕ), can be obtained as shown in Table 1.

$$f = \alpha I_1 + \sqrt{J_2} - k \tag{3}$$

$$\alpha = \frac{2\sin\phi}{\sqrt{3}(3-\sin\phi)} \quad , \quad k = \frac{6c\cos\phi}{\sqrt{3}(3-\sin\phi)} \tag{4}$$

$$\alpha = \frac{1}{\sqrt{3}}\left(\frac{\mathrm{f_c} - f_t}{f_c + f_t}\right) \quad , \quad k = \frac{2}{\sqrt{3}}\left(\frac{f_c f_t}{f_c + f_t}\right) \tag{5}$$

$$\sin\phi = \frac{3\alpha\sqrt{3}}{2+\alpha\sqrt{3}} \quad , \quad c = 6\frac{k\sqrt{3}(3-\sin\phi)}{\cos\phi} \tag{6}$$

3.4 Boundary conditions

In the analysis, according to the boundary conditions of tested model, the following conditions have been considered, as shown in Fig. 11:

a) Constraint in X, Y directions at top of column by using of hinged support.
b) Constraint in X, Y, Z directions at bottom of column by using of hinged support.
c) Constraint in X, Z directions at each beam end by using of hinged support.
d) Constraint in Z direction at along beam's flanges for considering of lack of lateral moves by using of hinged support.

Table 1 The Drucker-Prager model parameters (N-mm), (Mpa)

f_c	f_t	α	k	ϕ°	c
42.00	4.08	0.48	4.29	60.99	5.43

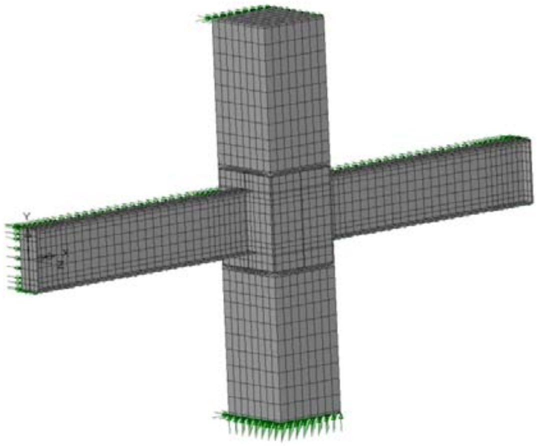


Fig. 11 Boundary conditions of 3D finite element model

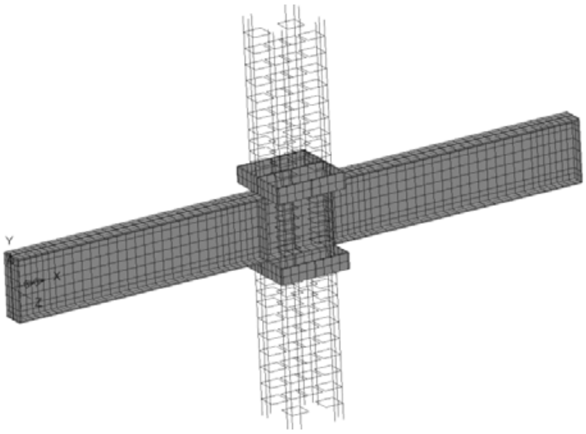


Fig. 12 Rebar and ties of concrete parts of 3D finite element model

3.5 Interaction and constraint conditions between components

Experimental results show that relative slip between concrete and steel plates did not occur. However, the interface element has been used for the concrete near the steel plate's surfaces. The embedded constraint was applied to the rebars and concrete of column as shown in Fig. 12. In this constraint, the translational DOF of nodes on the rebar elements were constrained to the interpolated values of the corresponding DOF of the concrete elements. It should be noted that during the conducted experiment the slip of the rebars was ignored.

3.6 Loading procedure

One of the important points in the hysteretic analysis is the pattern of cyclic loading history applied to the models. Since the loading history has significant effects in consistent evaluation of seismic resistance, it has been of particular interest for both reinforced concrete structures (e.g.,

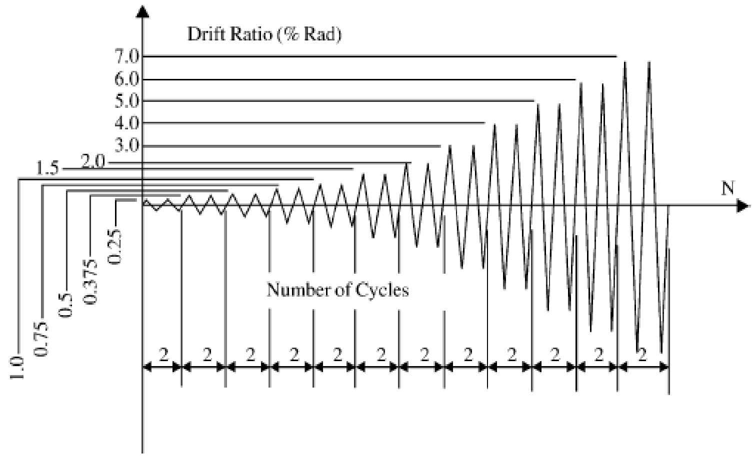


Fig. 13 The cyclic loading procedure

Hwang and Scribner 1984) and steel structures (e.g., Krawinkler 1990).

Based on the experimental research (Cheng and Chen 2005), a monotonically increasing cyclic loading for finite element modeling purposes has been used. As shown in Fig. 13, was applied the cyclic loading with displacement control at each beam end. This loading protocol is based on the Interim Protocol I, Fig. 2-1 of FEMA 461 (2007) that presents a conceptual diagram of the recommended loading history. In addition, a 1000 KN constant axial load at the top of the column has been applied to represent the gravity load, was obtained from the frame analysis.

4. Model results

The 3D finite element model has been analyzed under cyclic loading with nonlinear static analysis method. It is worth noting that the analysis was accomplished in about 2700 loadingsteps and corresponding displacement of beam end at the end was about 150 mm. The analysis results obtained at each step have been recorded. In general, there are good agreement between the finite element and experimental model, as shown in Fig. 14. The test and finite element model results show that both models are in a ductile manner with plastic hinges formed at the beam near the column face, where local buckling took place successively at the beam flange and web, based on strong column/weak beam concept (SCWB). In the finite element modeling, concrete cracking was not considered for concrete materials. Therefore, there is a small difference between finite element model and experimental results.

Fig. 15 shows the comparison of hysteretic curve of beam shear – displacement at beam end of finite element model and experimental results. In addition, the comparison of the 3D finite element modeling and experimental results (INUC and ICSC specimens) is summarized in Table 2. According to the results, the moments in the left and right beam of finite element model are about 7%, as average value, smaller than experimental results. In addition, the positive and negative stiffness of finite element model are about 12%, as average value, larger than experimental results. The panel shear strength of connection at damaged step of finite element model is about 13% larger than experimental result. This table also summarizes the analytical shear strength in the

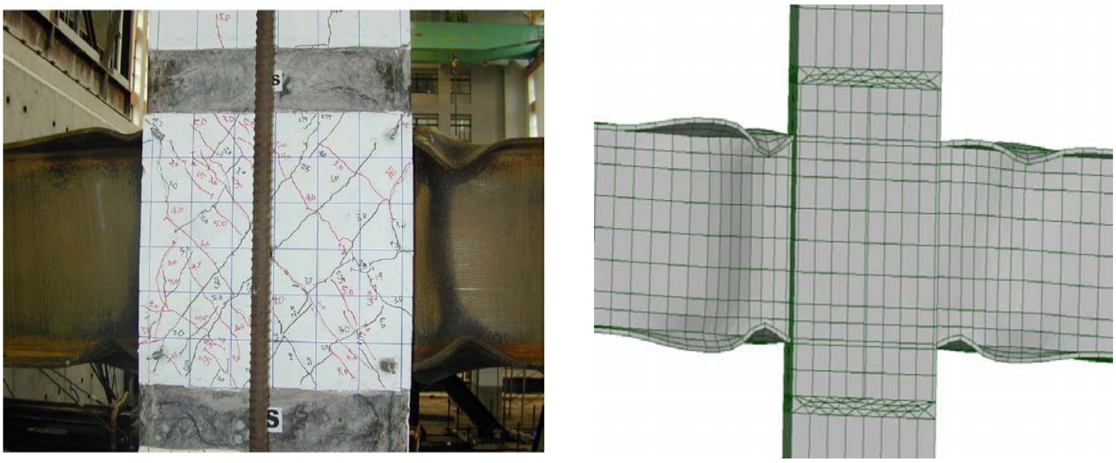


Fig. 14 Comparison of damage of finite element and experimental models

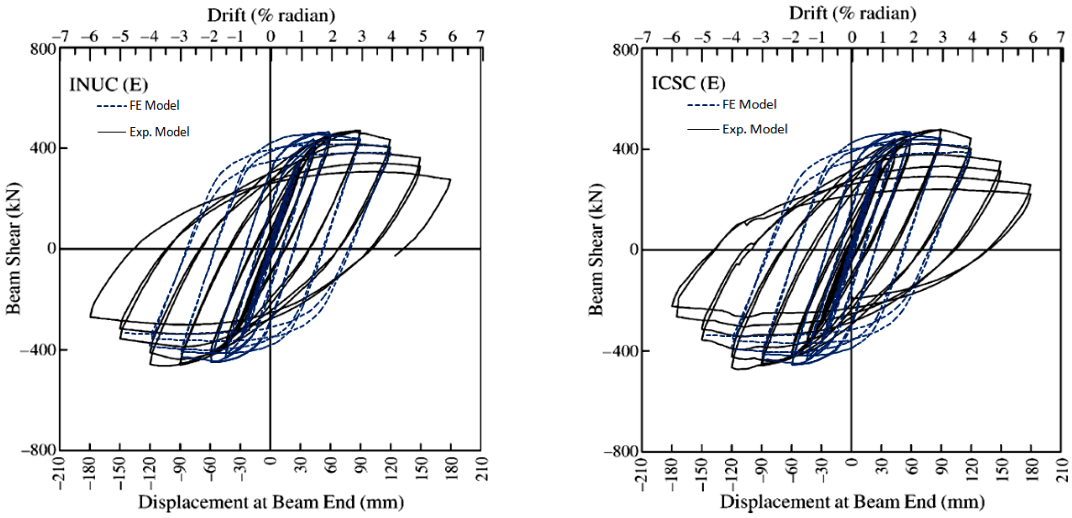


Fig. 15 Comparison of hysteretic curve of beam shear – displacement at beam end

Table 2 The strength and stiffness of FE and experimental model

Models	Results of comparison							Analytical panel shear (KN)		
	Moment in east beam (KN.m)		Moment in west beam (KN.m)		Stiffness (KN/m)		Panel shear (KN)			
	Positive	Negative	Positive	Negative	Positive	Negative		K&D	P&W	AIJ
INUC*	1293	1276	1229	1229	12321	11515	3477	3570	5760	5546
ICSC*	1272	1253	1244	1204	12418	10776	3408	3229	5083	5788
FE Model	1150	1179	1188	1166	13018	12850	3945			

*Experimental model

panel zone, based on the recommended relations by Kanno and Deierlein (2000), Para-Montesinos and Wight (2000), and AIJ-1994.

Fig. 15 indicates that the unloading stiffness of finite element model is approximately parallel with initial stiffness. This can be attributed to approximately modeling of perfect material properties of the experimental model in the finite element model, such as concrete or steel properties. However, there is a good agreement between the results of finite element and experiments, especially on the cyclic loops and maximum values in the curves and reduction amount of maximum value of beam shear at any stage of loading and unloading.

To evaluate the results in comparison manner, the enveloped curves, or push curves, of cyclic results for finite element and experimental models are created. Fig. 16 presents the push curves of finite element and experimental models at beam. It shows that there is a good agreement between them. However, initial stiffness of finite element model is slightly larger than that of experimental model. The maximum beam shear, top value of curves for FE and experimental models, has occurred at displacements equal to 55 and 85 mm, respectively. This difference can be attributed to difference between the initial stiffness of push curves. In addition, the top value of curves for FE

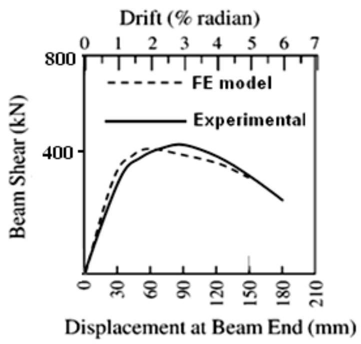


Fig. 16 Comparison of the push curve for FE and experimental models

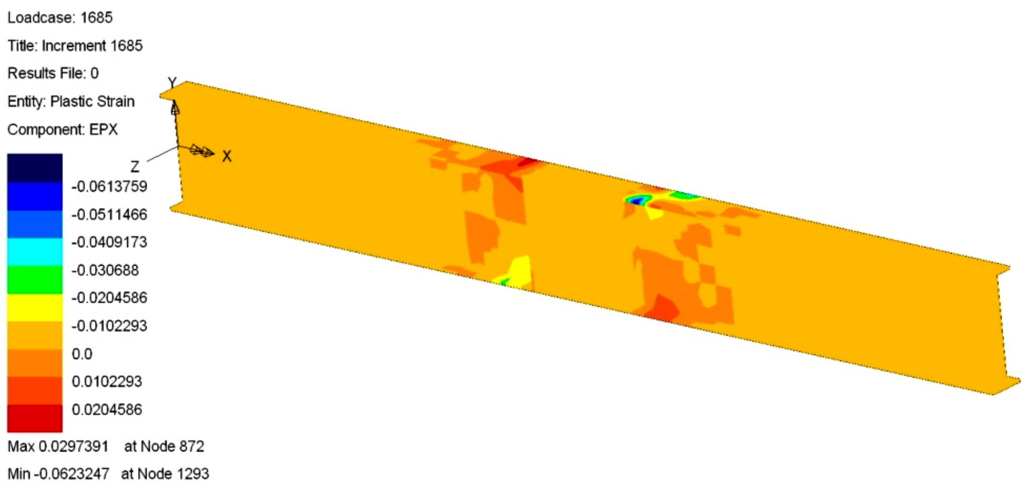
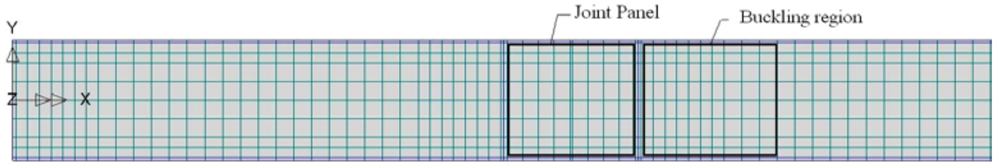
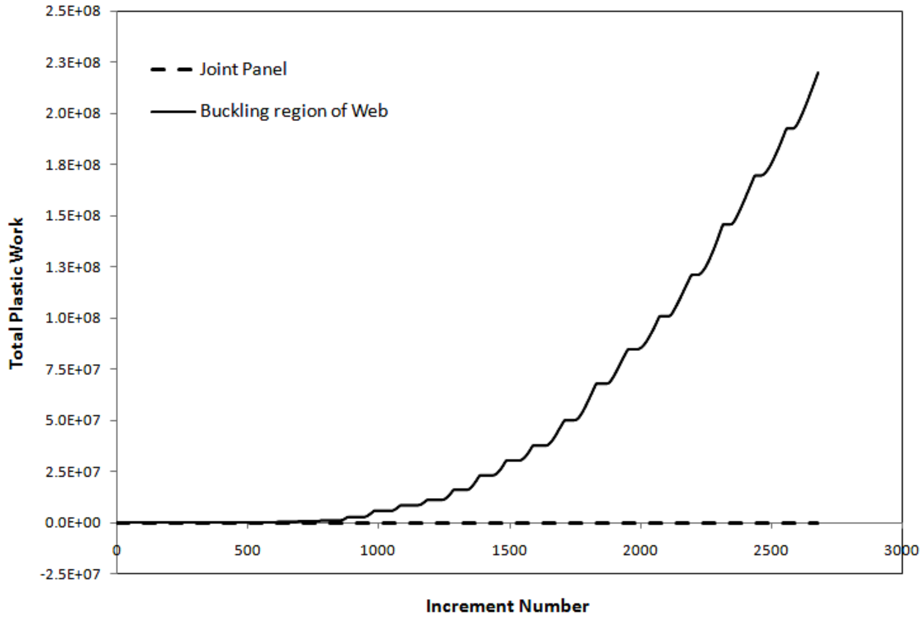


Fig. 17 Axial plastic strain contour at the joint panel and along of beams at FE model



(a) Joint panel and buckling region of web



(b) Total plastic work diagram

Fig. 18 Total plastic work of Joint panel and buckling region of web

and experimental models is equal to 453 and 470 KN, respectively, with the difference of less than 4%.

Fig. 17 shows the axial plastic strain contour, ε_{px}, at the joint panel and beams at the beginning of beam flange buckling. The joint panel plastic strains are about zero and this region often has an elastic behavior, the same as laboratory observations. Based on test results, Cheng and Chen (2005), all specimens performed in a ductile manner with plastic hinges formed at the beam-ends near the column face, and only minor damage such as cracks were observed in the column and the panel zone.

Total plastic work at two regions of beam web, as shown in Fig. 18(a), has been evaluated for the finite element model. The total plastic work at joint panel, as shown in Fig. 18(b), is equal to zero at all increments. In addition, at the buckling region of web near the joint panel, plastic work has been increased by increasing of increment number or cyclic loading. However, joint panel has an elastic behavior at all increments while the beam web, near region of joint panel, has a plastic behavior after increment number equal to 528 (when the beginning of web buckling has occurred at this region). However, in the experimental results has not been considered the plastic work value.

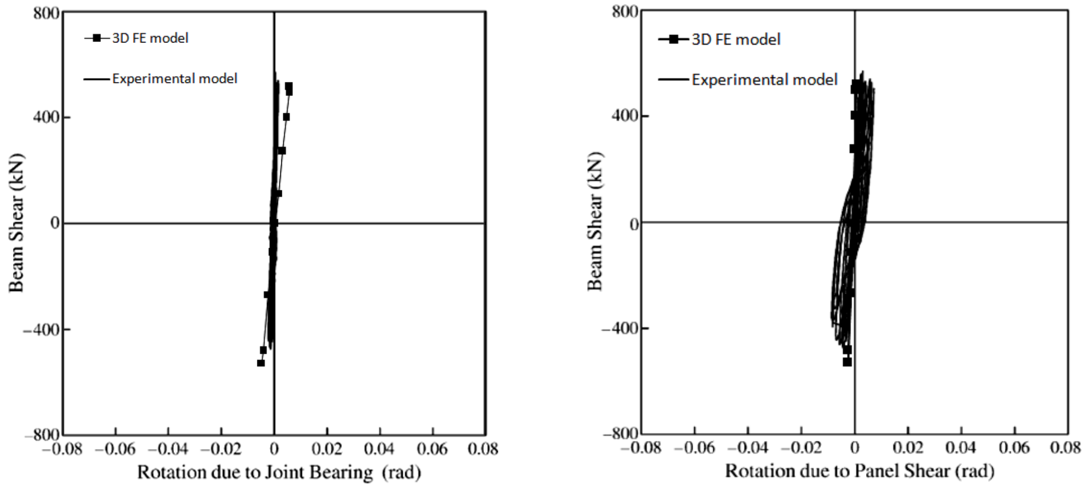


Fig. 19 Joint bearing and panel shear rotations

Fig. 19 shows the rotations due to joint bearing and panel shear in the finite element and considered experimental models due to cyclic loading. The behavior of joint panel of finite element model is elastic and does not sensible plastic or damaged behavior, as observed on the experimental model.

5. Conclusions

It can be concluded that the proposed 3D FE modeling reveals good correlation at all the main features of the behavior of composite RCS connections. It offers a reliable and very cost-effective alternative to laboratory testing. The conclusions are summarized as follow:

- Plastic phase in the beam web of finite element model begins at the drift equal to 1%, which is also been observed by experiment.
- The beginning of plastic phase in the beam flange of finite element model occurred at the drift equal to about 0.07% earlier than laboratory observations. This probably is related to the finite element approximation of modeling at geometry and material.
- The beginning of buckling has been occurred at drift equal to 3%, but the beginning of sensible buckling occurred at the displacement equal to about 108 mm and the drift equal to 4% of beam end, as shown in the experimental results.
- Rotation due to joint bearing and panel shear is remaining in the elastic behavior that has been shown in the experimental results. This indicates that the failure mechanism does not occur in the inner panel.
- It is obtained that the initial stiffness of FE model is slightly higher than that of experimental model. That is possibly related to size of elements and material properties at FE model. In addition, in the finite element solution the displacements are (on the "whole") underestimated and hence the stiffness of the mathematical model is (on the "whole") overestimated (Bathe 1996).

References

American Institute of Steel Construction (2001), *Manual of Steel Construction Load and Resistance Factor Design* (*LRFD*), 3rd Edition, Chicago, IL, AISC.

Azar, B.F., Ghaffarzadeh, H. and Talebian, N. (2013), "Seismic performance of composite RCS special moment frames", *KSCE J. Civil Eng*., **17**(2), 450-457.

Bathe, K.J. (1996), *Finite Element Procedures*, Simon &Schuster, A Viacom Company, Upper Saddle River, New Jersey, USA.

BIBLIOGRAPHY AIJ composite committee (1994), *Design Guidelines for Composite RCS Joints*, Architectural Institute of Japan, Tokyo, Japan.

Braconi, A., Salvatore, W., Tremblay, R. and Bursi, O.S. (2007), "Behaviour and modeling of partial-strength beam-to-column composite joints for seismic applications", *J. Earthq. Eng. Struct. Dyn*., **36**(1), 142-161.

Bugeja, M.N., Bracci, J.M. and Moore, W.P. (2000), "Seismic behavior of composite RCS frame systems", *J. Struct. Eng. – ASCE*, **126**(4), 429-436.

Cheng, C.T. and Chen, C.C. (2005), "Seismic behavior of steel beam and reinforced concrete column connections", *J. Construct. Steel Res*., **61**(5), 587-606.

Cordova, P.P. and Deierlein, G.G. (2005), *Validation of the Seismic Performance of Composite RCS Frames: Full-Scale Testing, Analytical Modelling, and Seismic Design*, Report No. 155 the John A. Blume Earthquake Engineering Center, Stanford University.

Cordova, P.P. (2005), *Validation of the Seismic Performance of Composite RCS Frames*, Ph.D. Dissertation, Stanford University.

Deierlein, G.G., Sheikh, T.M., Yura, J.A. and Jirsa, J.O. (1989), "Beam–column moment connections for composite frames: part 2", *J. Struct. Eng. – ASCE*, **115**(11), 2877-2896.

FEMA 461 (2007), "*Interim Protocols for Determining Seismic Performance Characteristics of Structural and Nonstructural Components through Laboratory Testing*", Federal Emergency Management Agency.

Griffis, L.G. (1986), "Some design considerations for composite-frame structures", *AISC/Engineering Journal*, *Second Quarter*, 59-64.

Hwang, T.H. and Scribner, C.F. (1984), "R/C member cyclic response during various loadings", *J. Struct. Eng. - ASCE*, **110**(3), 477-489.

Kanno, R. (1993), *Strength Deformation, and Seismic Resistance of Joints between Steel Beams and Reinforced Concrete Columns*, Ph.D. Dissertation, Cornell University, Ithaca, NY.

Kanno, R. and Deierlein, G.G. (2000), "Design model of joints for RCS frames In: Composite construction in steel and concrete" *IV*, *ASCE*.

Krawinkler, H. and Nassar, A. (1990), "Strength and ductility demands for SDOF and MDOF systems subjected whittier narrows earthquake ground", *SMIP 1990 Seminar on Seismological and Engineering Implications of Recent Strong-Motion Data*, 3-10.

Krawinkler, H.A., Gupta, R. and Luco, N. (2000), *Development of Loading Histories for Testing of Steel Beam–Column Assemblies*, Report for the SAC steel project, FEMA.

Liu, J. and Astaneh-Asl, A. (2000), "Cyclic behavior of shear connections with floor slab", *Proceedings of 6th ASCCS Conference*, Los Angeles, March.

Lusas (2006), Software package, Version 14.03.

Mirza, O. and Uy, B. (2010), "Finite element model for the long-term behaviour of composite steel-concrete push tests", *J. Steel Compos. Struct*., **10**(4), 45-67.

Noguchi, H. and Uchida, K. (2004), "FEM Analysis of hybrid structural frames with RC columns and steel beams", *J. Struct. Eng. – ASCE*, 328-335.

Parra-Montesinos, G. and Wight, J.K. (2000), "Seismic response of exterior RC column-to-steel beam connections", *J. Struct. Eng., ASCE*, **126**(10), 1113-1121.

Sheikh, T.M., Deierlein, G.G., Yura, J.A. and Jirsa, J.O. (1989), "Beam–column moment connections for composite frames: part 1", *J. Struct. Eng., ASCE*, **115**(11), 2858-2876.

Shen, H.X. and Qiang, G. (2010), "Nonlinear finite element analysis of exterior RC column to steel beam

connection", *Proceedings of ICCASM 2010*, Taiyuan-China, October.
Tort, C. and Hajjar, J. (2010), "Mixed finite-element modelling of rectangular concrete-filled steel tube members and frames under static and dynamic loads", *J. Struct. Eng. – ASCE*, **136**(6), 654-664.
Wang, Y., Xuan, Y. and Mao, P. (2010), "Nonlinear finite element analysis of the steel-concrete composite beam to concrete-filled steel tubular column joints", *Int'l J. Nonlinear Sci.*, **9**(3), 341-348.
Yu, Q.S., Noel, S. and Uang, C.M. (2000), "Composite slab effects on strength and stability of moment connections with RBS or welded haunch modification", *Proceedings of 6th ASCCS Conference*, Los Angeles, March.

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