

Analytical investigation of a new Through-Column-Type Joint for composite reinforced concrete and steel frames

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ABSTRACT

Composite structural systems, which consist of reinforced concrete columns and steel (RCS) beams, provide substantial savings in material cost, increase structural damping and lateral stiffness through RC columns, and expand energy dissipation capacity through steel beams. The load transfer mechanisms between RC column and steel beam may be very complicated, due to different material properties of columns and beams. In this study, a new Through-Column-Type Joint is proposed that uses different ways for developing a reliable load path. The joint is reinforced by cover plates and a vertical plate passing through the column, and beams are connected to the vertical plate named the Through Plate. This study employed finite-element analyses of the mentioned connection to investigate the structural performance and the stress transfer mechanisms. It was found that the initial stiffness and energy dissipation of the new details were increased and strength degradation was decreased under cyclic loading compared with previous details. The study further showed that the use of through plate is effective in enhancing the structural performance of the joints.

1. Introduction

Composite structures, which consist of steel and reinforced concrete members, have been used for the last 30 years in various forms. One of these systems which resist seismic moments based on the moment connection between reinforced concrete columns and steel beams is called RCS system. Using RC instead of structural steel as columns can result in substantial savings in material cost and an increase in the structural damping and lateral stiffness of the building. Energy dissipation capacity can accordingly be provided through steel beams. In fact, this system with the optimum

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combination of steel and concrete structural elements has the advantages of both concrete and steel systems.

To date, there have been recognized two main categories in RCM connections, namely the through-beam type and the through-column type. In the through-column type, beams are cut adjacent to the column, so compared with the through-beam type, there is less interruption in panel zone and longitudinal bars can be placed in appropriate arrangements in the column section. The details offered for such kind of connections that use diaphragms or cover plates to connect the steel beam and column are generally complicated and the explanation of the force transferring Mechanisms is difficult to understand. Also, additional effort in connection details to ensure a better seismic capacity in terms of strength and ductility is needed.

A primary challenge in examining the behavior of composite beam-column connections has been addressed by Sheikh (1989) and Deierlein (1989) through results of an experimental program where 15 two-third scale joint specimens were tested under monotonic and cyclic loading. They further proposed a design model for calculating the joint strength. Subsequently, the proposed design equations were incorporated in a set of ASCE guidelines for design of joints between RC columns and steel beams (ASCE 1994). Since 1997, collaborative research on RCS systems has been conducted in the US and Japan, such as the work of Baba and Nishimura (2000), Kim and Noguchi (1997), Nishiyama et al. (1997), Parra-Montesinos and Wight (2000), and Bugeja et al. (2000).

Notwithstanding the extensive research on the design and behavior of the composite RCS beam-column connections, most of the proposed details are related to the through-beam type. Given the desirable features of through-column type connections, further research is required to improve existing methods. The main objective of this study is to propose a new Through-Column-Type Joint that uses different ways for developing a reliable load path. The joint is reinforced by cover plates and a vertical plate passing through the column, and then beams are connected to the vertical plate that is called the Through Plate in this study. This study performs finite-element analyses of the mentioned connection to investigate the structural performance and the stress transfer mechanisms.

2. Proposed connection

This connection is consist of steel cover plates which encompass the column in panel zone and a vertical plate passing through the column that is called Through Plate in this study.

As shown in Fig. 2, the through plate passes through the concrete column and is connected to steel cover plates. It is supposed that all the demands for a moment connection are transferred to the column by this plate. So, as a design rule, the shear and bending moment capacity of the through plate at critical section is checked to provide the load path.

After trimming the beam web in this region, the beam is connected to the vertical plate by longitudinal connection of the flanges to either side of the through plate, so the welds are loaded parallel to axis and better performance of the welds is provided.

Two web connection plates are also required on either side of the through plate and the portion of the beam web moment is effectively transferred to the column by means of the through plate. Because of orthogonal moment frames, a cruciform through plate can be used to assemble the beams in the vertical directions.

A fundamental component of composite connections is providing effective shear connection between steel and concrete. An essential feature of such connections is that no sliding occurs between steel and concrete under cyclic loading in panel zone. Such connection with minimum interruption in the panel zone is provided by means of rigid shear connectors. As shown in Fig. 3, in the proposed connection, four rows of bar shear connectors are connected inside the cover plates to supply necessary friction between steel and concrete.

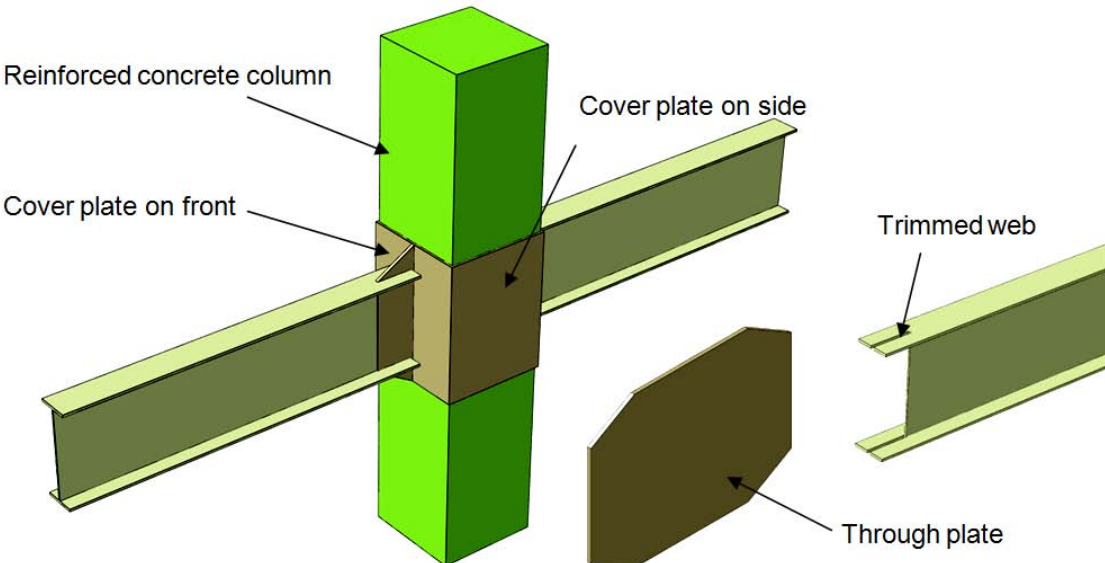


Fig. 2 Proposed composite moment connection

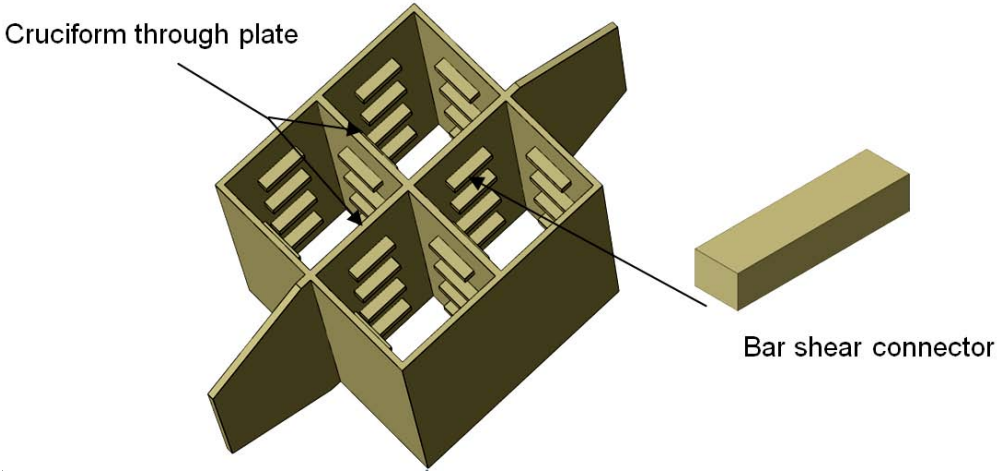


Fig. 3 Connection details in panel zone and position of shear connectors

3. Connection force transferring mechanisms

It is expected that the plastic hinges will form in the beam adjacent the column face just after the through plate. The plastic hinge moment and the plastic shear forces are transferred to the through plate by means of flange and web connections. The beam flange connection transfers the flange tension and compression forces (T_f, P_f), and the beam web connection transfers the beam shear forces (V_b) and the portion of the beam web moment (M_w) to the through plate. According to Fig. 4, all the forces are placed in the through plate plane and tend to rotate it relative to the column. Since this plate is connected to the cover plates, the induced forces are distributed between the through plate inside the column and cover plates. As depicted in Fig. 5, the distributed forces develop out-of-plane-forces in the cover plate on front and shear forces in the through plate inside the column. On the one hand, these out-of-plane-forces together with compression forces of the column, lead to the formation of the concrete diagonal strut (Fig. 6). On the other hand, the out-of-plane-forces mobilize shear forces in the side plates. Due to engagement of steel plates and concrete through shear connectors in the panel zone region, shear forces are transferred to the concrete along the panel zone. It is noticeable that the concrete is confined by cover plates in the panel zone and stirrups are not needed in this region.

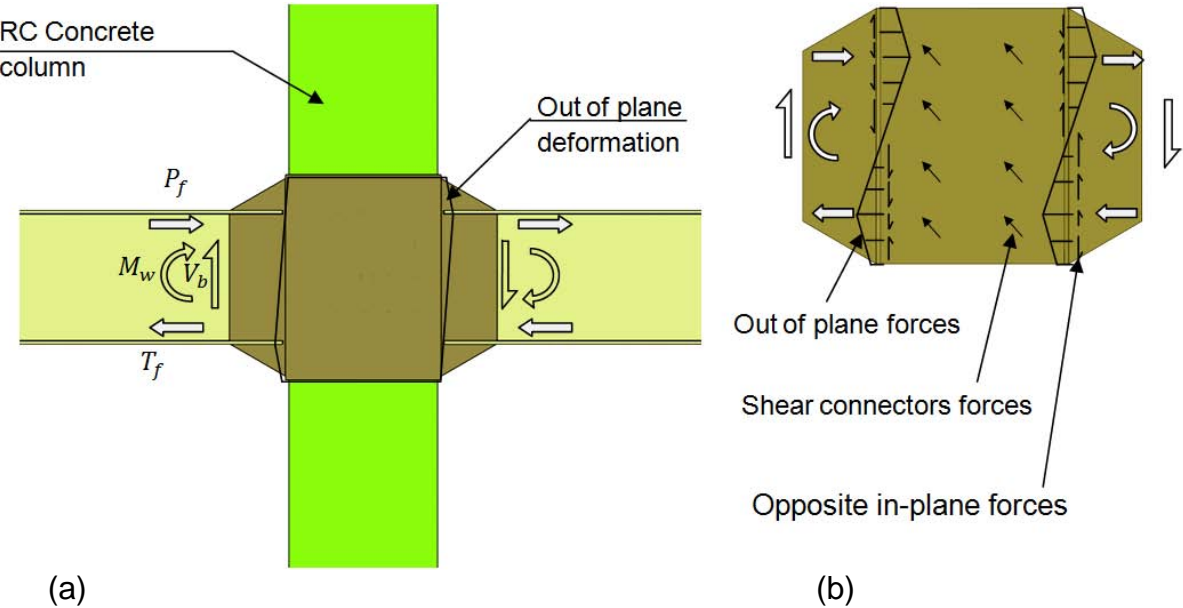
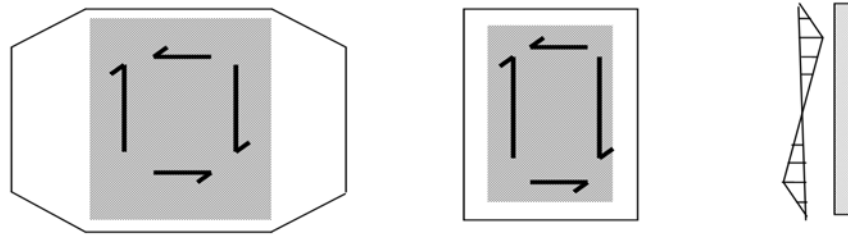


Fig. 4 Connection force transferring mechanism: (a) connection mechanism, (b) in-plane and out-of-plane resisting forces in through plate

Based on this model, the plastic hinge moment of the beams, after transferring to the through plate plane, is converted into opposite in-plane-forces on steel plates that are transferred to the concrete by shear connectors and also diagonal strut in concrete. The resultant force develops the bending moment in the column.



(a) Through plate (b) Cover plate on side (c) cover plate on front

Fig. 5 Internal forces in connection components

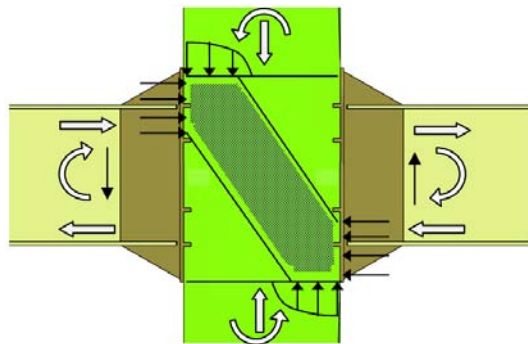
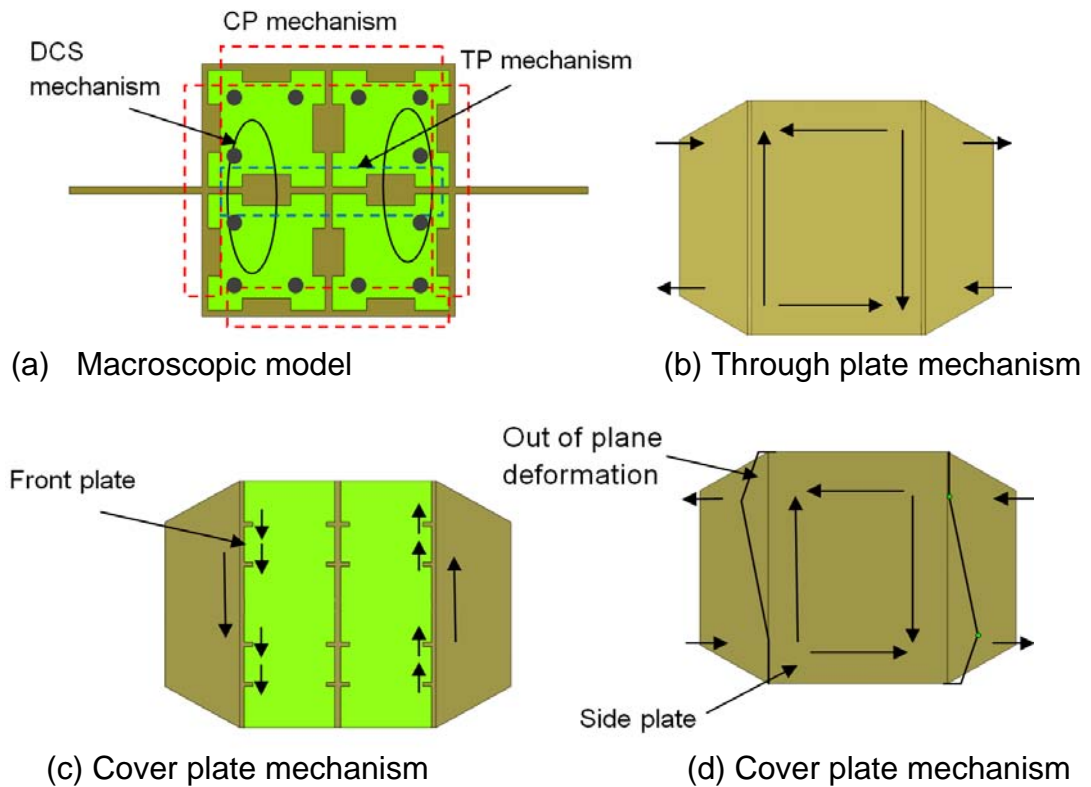


Fig. 6 Concrete diagonal strut in panel zone



(a) Macroscopic model (b) Through plate mechanism (c) Cover plate mechanism (d) Cover plate mechanism

Fig. 7 Stress transferring mechanisms for composite reinforced concrete and steel joint

Consequently, a macroscopic model for Stress transferring mechanisms in composite connection is composed of three mechanisms shown in Fig. 7, namely through panel (TP) mechanism, concrete diagonal strut (CDS) mechanism and cover plate (CP) mechanism. The TP mechanism consists of a steel plate in the joint which transfers shear stresses through resisting tensile and compressive forces from beam flanges to the concrete via shear connectors. The CDS mechanism includes concrete confined by cover plates and is caused by resisting compressive forces from beam flanges and columns as depicted in Fig. 6. The CP mechanism consists of steel cover plates which include two submechanisms. One mechanism transfers shear stresses in cover plate on side caused by out-of-plane deformation from the front plate to the concrete by means of shear connectors located inside the side plates. The other mechanism transfers beam shear forces by means of shear connectors located inside the front plates along the concrete surface.

4. Evaluation of the proposed connection behavior by finite element modeling

To evaluate the behavior of the proposed connection and also to investigate the proposed load paths, a finite element model was developed based on a verified finite element model conducted by Puladi (2011). The finite element model was constructed based on a specimen tested by Cheng (2004) to verify finite element model with experimental results. Then, the verified model was adjusted to appropriate dimensions according to the proposed connection. In this study, a finite element pre-processor ABAQUS/CAE was used to create the three dimensional model of the test.

4.1 Test specimen

In the full-scale plane frame tested by Cheng (2004), as shown in Fig. 8, the specimen has the steel beam H596x199x10x15 in size and 65x65 cm columns reinforced with 12 #11 longitudinal bars. Before the test, the hydraulic jack at the top of the column applied a 1000 KN constant axial load to represent columns in a gravity load. Then, hydraulic actuators at each beam tip applied the cyclic load with displacement control in the form of triangular waves. The material strengths of steel and the compressive strength of the concrete are summarized in Table 1.

4.2 Finite element modeling

Solid elements C3D8R were employed to model concrete column, steel beam and shear connectors. C3D8R element is an 8-node brick element with reduced-integration stiffness, and is used for nonlinear analyses including contact large deformation, plasticity, and failure. Furthermore, in order to model longitudinal bars and stirrups, truss elements T3D2 was utilized in the numerical model. The support was modeled with rigid element R3D4.

Table 1 Material strength of tested specimen (Cheng 2004)

Item	Rebar #4	Longitudinal bar #11	Beam web	Beam flange
f_y (MPa)	430.7	443.3	478.5	444.2
f_u (MPa)	680.6	674.6	598.7	568.0
f'_c (MPa)	54.3			

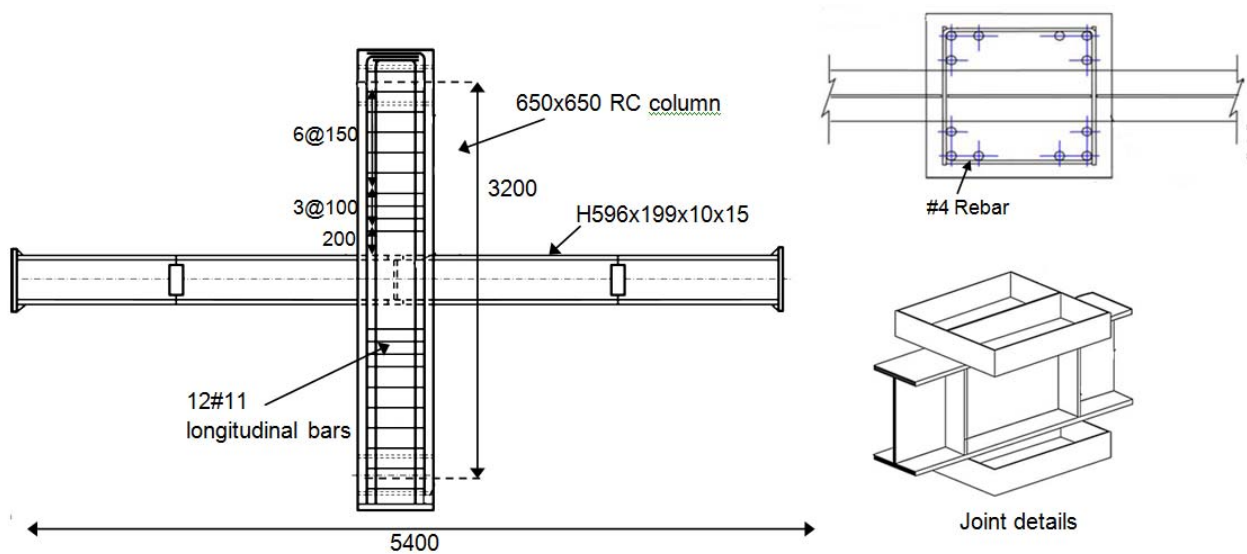


Fig. 8 Details of the specimen tested by Cheng (2004) (all dimensions are in mm)

The bi-linear elastic-plastic model, shown in Fig. 9, was used for steel. The mechanical behavior for both tension and compression was assumed to be identical. The yield stress and tensile strength of steel, as provided in Table 1 and the elongation was about 20%. Young's modulus of elasticity and The Poisson's ratio were considered to be 200 GPa and 0.3, respectively.

The concrete damaged plasticity model (CDP) was used to model the concrete of the column. When subjected to severe inelastic stress states, concrete can show a significant volume change, commonly referred to as dilation. The dilation can be represented by the appropriate plastic potential function. In this study, the dilation angle was taken equal to 40° , while other values were assumed 0.1 and 1.16 for flow potential eccentricity (ϵ) and the ratio of biaxial compressive strength to uniaxial compressive strength (f_{b0}/f_{c0}), respectively. The nonlinear behavior of the concrete material was applied to the FE model by equivalent uniaxial stress-strain curves (Fig. 10). The first part was initially assumed to be elastic to the proportional limit stress. The value of the proportional limit stress was taken as equal to $0.4f_{ck}$, as presented in

EC2 (1992), where f_{ck} is the compressive cylinder strength of concrete. The strain of the concrete peak strength (ϵ_{c1}) associated with f_{ck} is equal to 0.0022. The initial Young's modulus was calculated according to EC2 (1992). The Poisson's ratio was taken equal to 0.2. The reduction factor r was assumed to be 0.85. The ultimate strain (ϵ_{cu}) of concrete at failure associated with rf_{ck} was assumed equal to 0.01.

For concrete column and steel plates interface, a surface-based contact was used. A hard contact relationship was used to minimize the penetration of plates in the concrete. Also, to prevent the transfer of tensile stress across the interface of concrete and steel, a frictionless contact property was used. For the concrete column and the reinforcing bar interface, an embedded constrain method was used and the interfaces of steel beam, cover plat, trough plate and shear connectors were tied together.

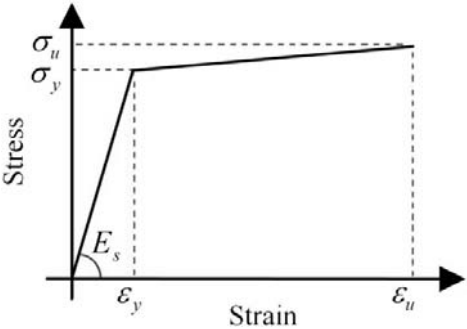


Fig. 9 Stress-strain relationship considered for steel

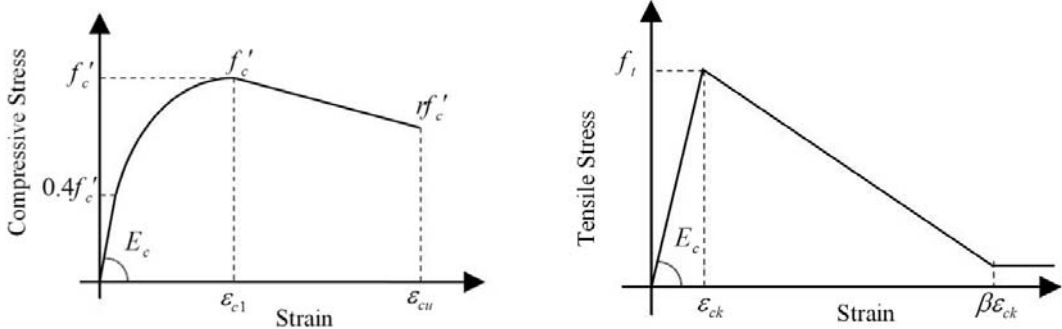


Fig. 10 Stress-Strain relationship for concrete

4.3 Validation of the finite element model

In order to validate the created model, ultimate strength, force-deformation relationship under cyclic loading and initial stiffness of the test specimen were compared with analytical results. For the test specimen, the ultimate strength was 479 KN, compared with 456 KN obtained from the finite element model which differed 4.8% from experimental result. In the test specimen and verified model, fracture of the beam flange occurred and the column and panel zone remained elastic. As presented in Fig. 11, it is obvious that the force–deformation relationship is in good agreement with test results.

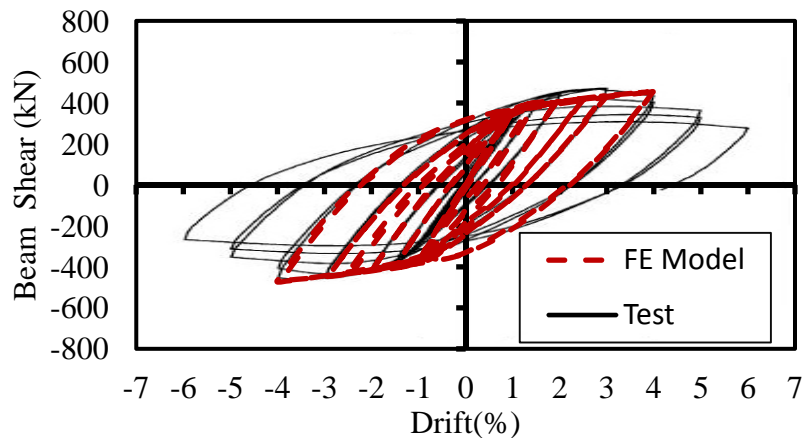


Fig. 11 Hysteretic curves for the specimen tested by Cheng (2004) and finite element analysis

5. Analytical results

To develop the proposed connection model, connection details of the verified model was changed to the mentioned configuration (

Fig. 12). Fig. 13 illustrates the deformed shape of the model at 6% story drift and maximum principal plastic strain distribution. It shows that the beam plastic hinge forms at the expected location away from the through plate. Based on the finite element results, all parts of the connection, except the beam, remained relatively elastic.

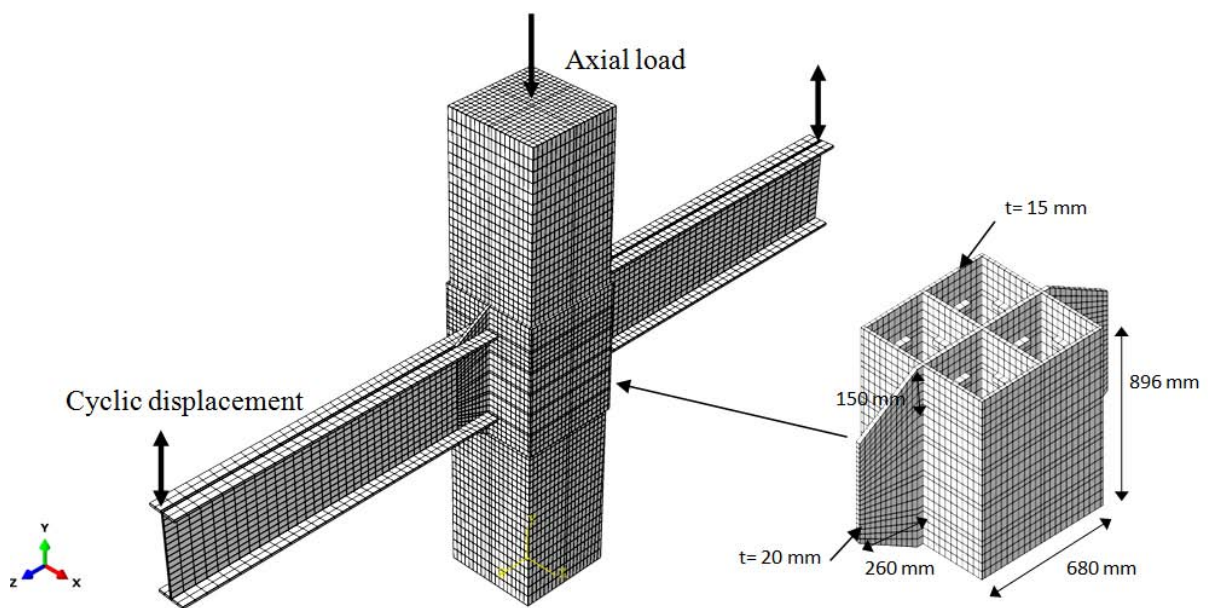


Fig. 12 3D finite element model of the proposed model

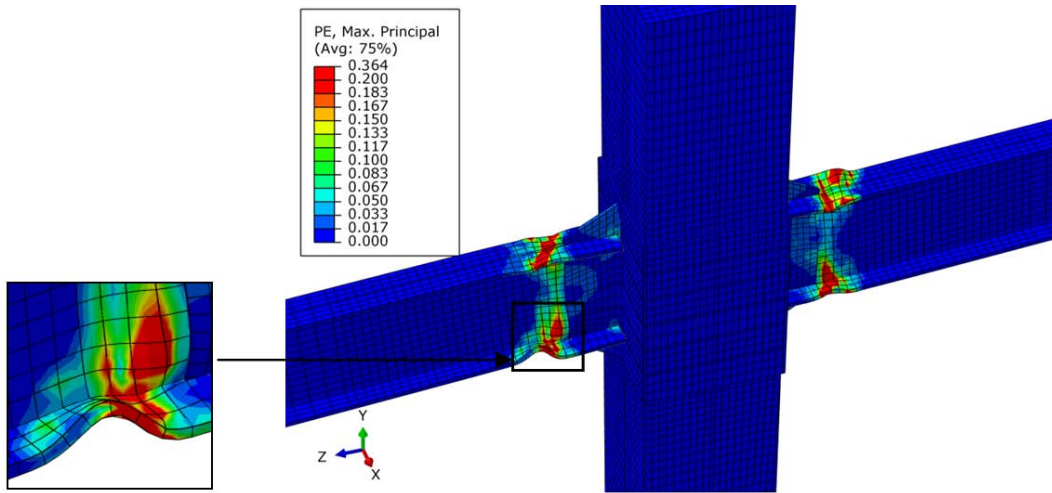


Fig. 13 Deformed shape and maximum principal plastic strain distribution at 6% story drift

The hysteretic curves of the beam tip load versus the beam tip displacement for the proposed connection and also the specimen tested by Cheng (2004) are presented in Fig. 14. On the basis of this figure, the proposed connection exhibits stable hysteretic behavior and shows more initial stiffness and better energy dissipation compared with the tested specimen. It is noticeable that the strength degradation of the specimens is resulted from ductile local and global buckles during the cyclic loading and that no fracture occurred in the specimen. The hysteretic curve shows that shear connectors acted well to accommodate steel plates and concrete in the panel zone and no noticeable pinging is observed due to slip along steel and concrete interface.

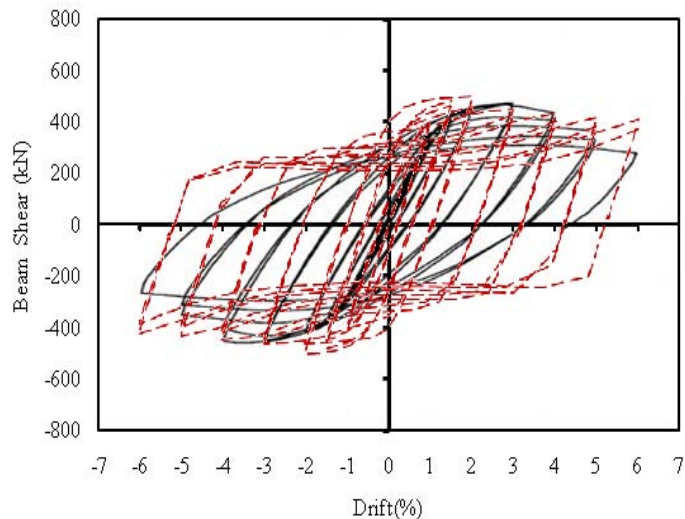


Fig. 14 Hysteretic curves for the specimen tested by Cheng (2004) and finite element analysis of proposed connection

Distribution of tension-compression and shear stresses in the through plate plane from finite element analysis at 1% drift, (Fig. 14), is in agreement with resisting forces in through plate as presented in Fig. 4(a). Tension-compression stress distribution shows that maximum stress occurs in the location of beam flanges in the through plate and the stress decreases as we move towards the edges of the through plate

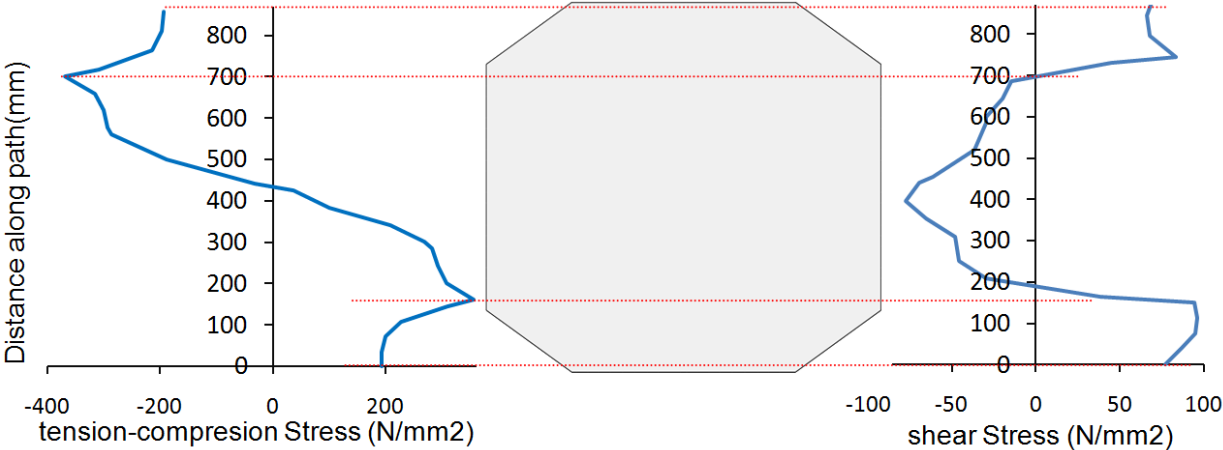


Fig. 15 Distribution of tension-compression and shear stresses in through plate plane in the specified path

Based on the finite element results, the CDS mechanism contribution to the load transfer mechanism is more than that of the other components, consisting 55% of the total shear forces transferred to the column. The portion of the TP and CP mechanisms of shear forces are 37% and 8%, respectively. Fig. 16 shows the formation of concrete diagonal strut in the panel zone and also shear stresses in the through plate at 6% drift. As it was expected, noticeable compression stress along a diagonal path in concrete confirms the formation of concrete diagonal strut in panel zone.

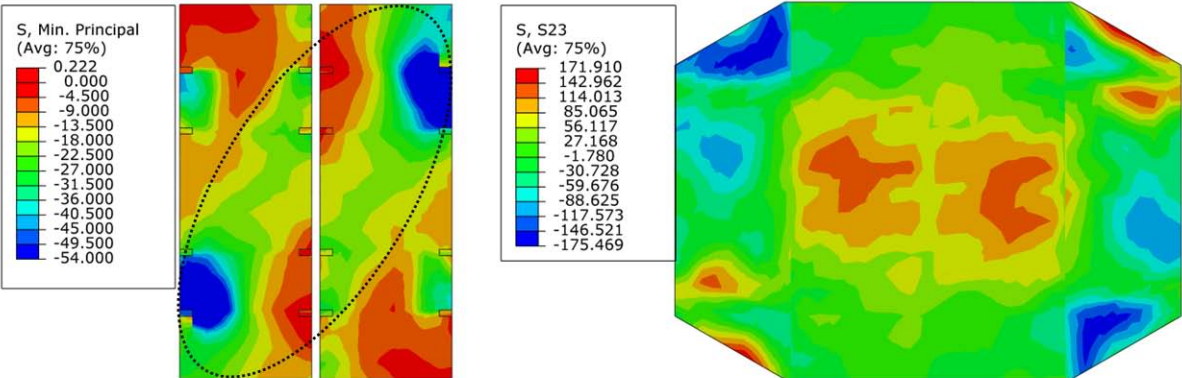


Fig. 16 (a) Minimum principal Stress contour of concrete in the panel zone; (b) Shear stress contour in the through plate

6. Conclusions

- In the proposed connection, plastic hinges formed in the beam ends near the column face.
- The induced forces of the beam plastic hinge tend to rotate the through plate relative to the column which are resisted by three mechanisms including in-plane resistance of the through plate and cover plates, and strut action of concrete in the panel zone. According to the finite element results, these mechanisms provide reliable load path for transferring the moment and forces of steel beam to concrete column.
- All the forces resulting from the beam plastic hinge are located in the through plate plane; consequently, most welds are loaded parallel-to-axis which supply more convenient performance.
- The CDS mechanism contribution to the load transfer mechanism is more than that of the other components, consisting 55% of the total shear forces transferred to the column.
- The hysteretic curve shows that shear connectors acted well to accommodate steel plates and concrete in panel zone and that no noticeable pinging is observed in the hysteretic curve due to slip along steel and concrete interface.

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