Journal of Mathematics and System Science 7 (2017) 198-211
doi: 10.17265/2159-5291/2017.07.003

Numerical Analysis of the Composite Connection of Steel Joist Embedded in Concrete Girder

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Abstract: So far, numerous numerical studies have been conducted on the behavior of Composite Reinforced Concrete-Steel (RCS) beam-to-column connections. However, the lack of studies regarding the steel joist-concrete girder connection has yet to be addressed through comprehensive finite element methods to get an understanding of influential parameters. Hence, in this paper, composite connection of embedded steel joist in concrete girder is investigated with an appropriate finite element software, namely, ABAQUS. The validity of the proposed model is examined by the comparison made with the test data in literature. Results indicate that maximum bending capacity of the connection is achieved when embedment ratio is 1.78. Moreover, double web angles in the embedment region significantly reduce the embedment length required to achieve the maximum bending capacity. Finally, damage analyses show that bending capacity of concrete girder is slightly reduced in the connection zone.
Keywords: Composite beam-to-column connection, embedment length, steel coupled beam, bending capacity

1. Introduction

In recent years, composite connections have gained popularity among researchers due to the optimal usage of concrete and steel in resisting the forces applied to the structures. Few specific guidelines are available for the connection of steel secondary beams embedded in Reinforced Concrete (RC) girder. Hence, investigating the behavior of composite connections is of paramount importance. Their applications include column base connections in steel structures, embedded steel coupling beams in RC shear walls and RCS frames. Furthermore, roof systems with steel joists incorporated in concrete frames, reduce the overall weight of the structure, and therefore the seismic loads applied to it. Ease of concreting, elimination of framework, capability to cover long spans in powerhouses (attributed to the high moment of inertia of steel profiles) and reduction in cost and construction time are some advantages of these roof systems.

Moment-resisting frame structures of high ductility class were studied (Salvatore and Bursi, 2005). Some research was done on the behavior of confined concrete using Drucker-Prugertype plasticity model in ABAQUS (Yuet al., 2010). A finite element model of composite frames was developed using shell elements (Bursi et al., 2005). An experimental model was used to evaluate the strength deterioration and damage propagation of RCS connections (Chou and Chen, 2010). Sustained damage to RCS connection in high seismic risk zoneswas investigated (Montesinos et al., 2003). The seismic behavior of steel beam- to-RC column connection with and without floor slabwas studied (Cheng and Chen, 2004). Composite frame structures having high-strength concrete columns, confined by continuous compound spiral ties and steel beams were

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studied (Li et al., 2012). Seismic behavior of RCS frames based on FEMA-356 and allowable rotation criterion were assessed (Farahmand Azar et al., 2013). It is desirable to design the coupling beams as shear-yielding members since a shear-critical coupling beam exhibits a better energy dissipation mode than a flexure-critical coupling beam (Park and Yun, 2005). Some research has been carried out on the interaction of shear force-bending moment in steel joist-concrete girder connections and several equations have been proposed (Yu et al., 2012). In this study, a specific length of steel joist was embedded in concrete with an angle shear connector. Hence, embedment length and its calculation is crucial.

2. Finite Element Model

2.1. General descriptions

In order to simulate the real behavior of the connection, four components need to be modeled:
* Contact between steel joist and concrete girder in the embedded region.
* Contact between steel joist and concrete slab.
* Interaction between reinforcing bars and concrete.
* Contact between anchor bars and concrete girder.

2.2. Material model

The mechanical behavior of concrete was simulated using a Concrete Damaged Plasticity (CDP) model for which the pertinent parameters were estimated by uniaxial stress.Fig.1 shows the default properties of concrete.

2.3. Material modeling of reinforcing bars

Regardless of the reinforcement service stage and Bauschinger effect in stress-strain relationships, ties and longitudinal reinforcements are assumed ideally elasto-plastic for simplification (Li et al., 2012).

2.4. Contact model between concrete-reinforcing bars, concrete and steel

Interaction model between concrete and bars is of embedded type and frictional behavior has been adopted for the contact between steel joist and concrete girder with friction coefficient of 0.7. Moreover, no slip between steel joist and concrete slab is assumed.

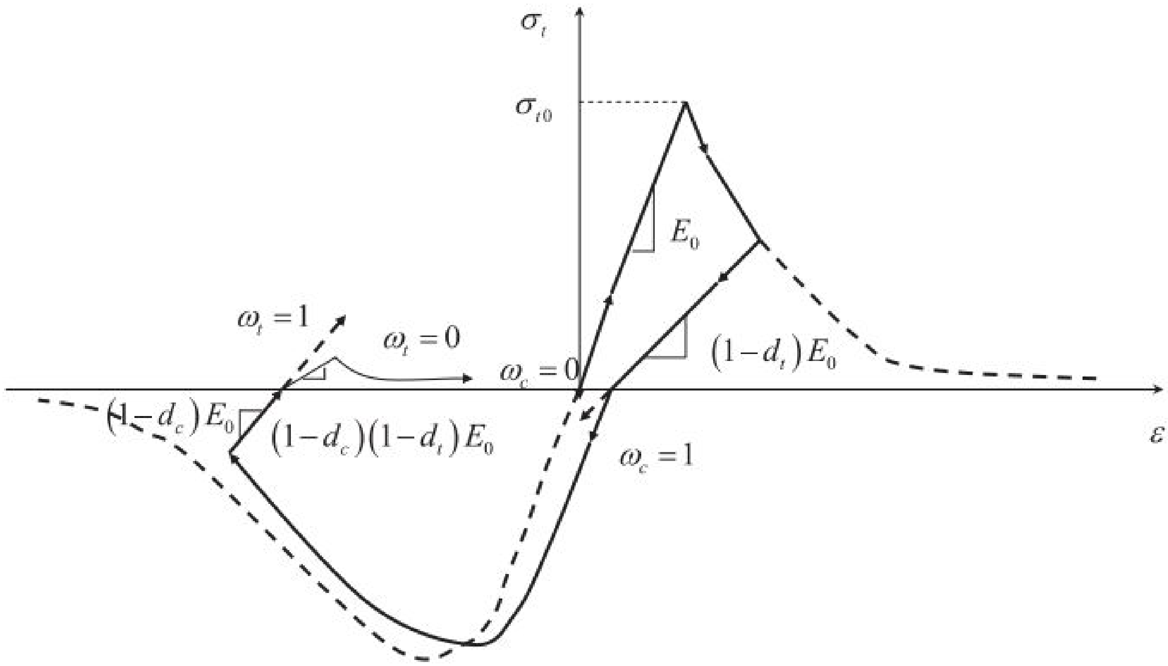

Fig. 1. Uniaxial load cycle (tension–compression–tension). Default values for the stiffness recovery factors are: wt=0and wc=1 (Li et al., 2012).

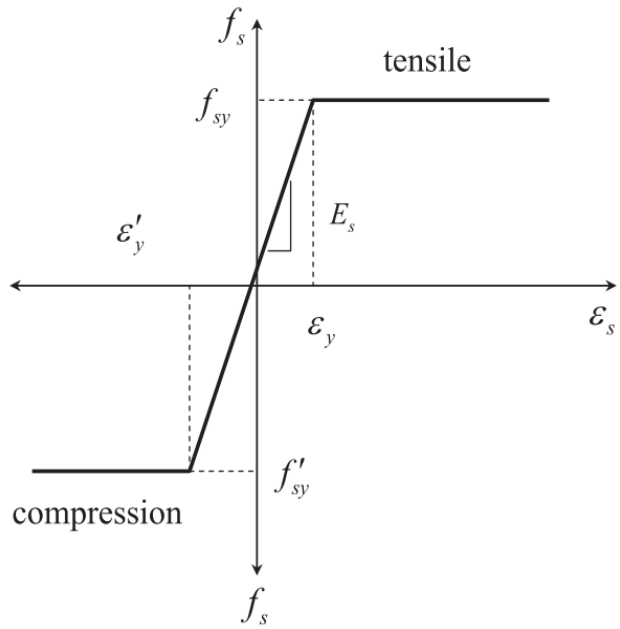

Fig. 2. Stress–strain relationship of reinforcement (Li et al., 2012).

Loading and boundary conditions of the verified model are shown in Fig. 3 in which two ends of the concrete girder are completely fixed and the load is transmitted to the concrete slab via four plates.

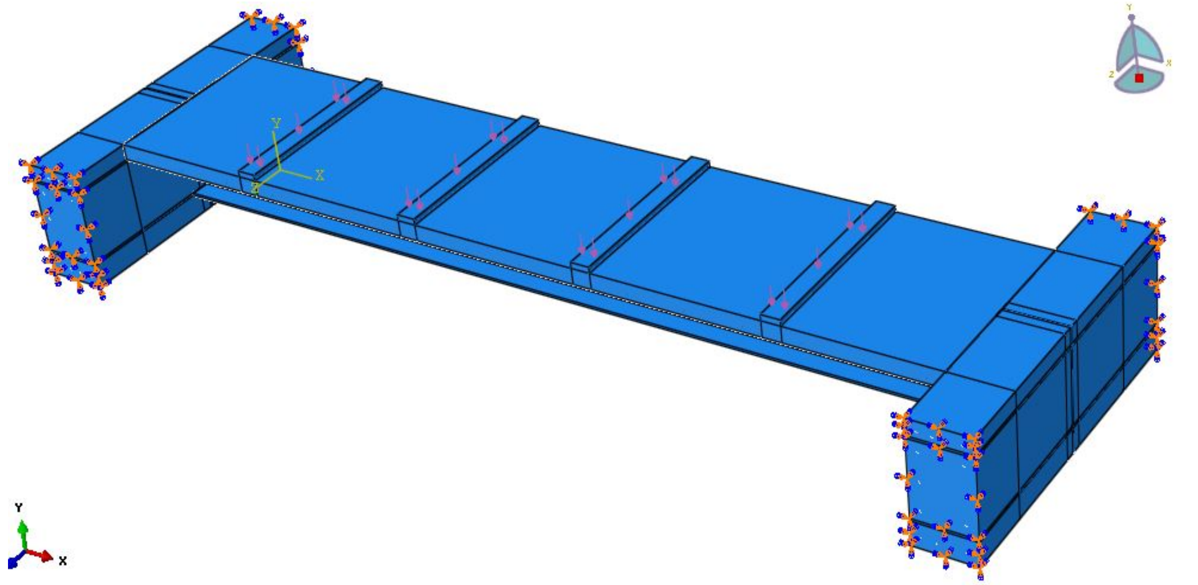

Fig. 3. Loaded Model

3. Validation of the Finite Element Model

In order to corroborate the proposed finite element model, load-displacement diagram of the simulated model was compared with the experimental model (Yu et al., 2011) in Fig. 4. Also, crack pattern of the aforementioned models is shown in Fig. 5. 8-node linear brick, reduced integration (C3DR8) solid elements plus 2-node linear 3-D (T3D2) truss elements were used to model concrete and reinforcements respectively. Details of reinforcements and their respective properties are shown in Tables 1 and 2.

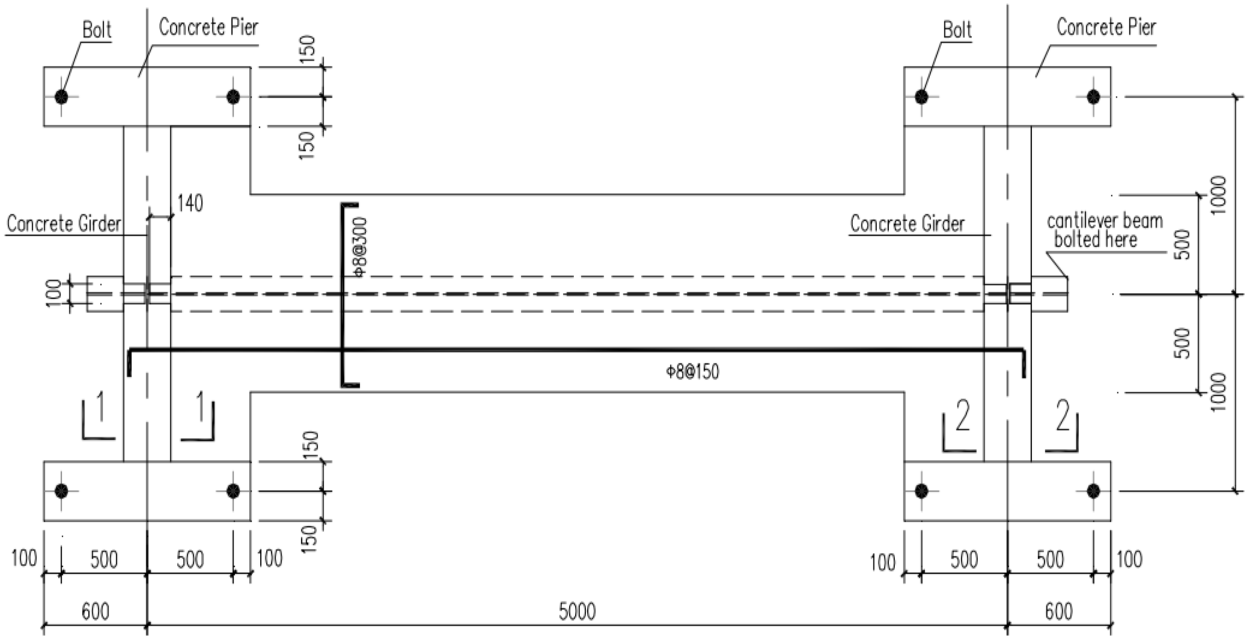

Fig. 4. Specimen model (Yu et al., 2011).

3.1. Mid-span force-displacement relationship of the steel joist

As it can be seen in Fig. 6, model behaves linearly till the 25mm deflection (corresponding force, 390kN). Afterwards, when the load reaches 550kN, steel joist slips inside the concrete girder causing failure and damage.

Table 1-Details and size of specimen

Specimen		
Girder	Section	b×h=320×900 mm
	Longitudinal reinforcement	12-D20 & 8-D10
	Tie	D12@200
	Length	2000 mm
Slab	Longitudinal reinforcement	23D8
Steel beam	Section	HN400×220×10×12 mm
	Length	5000 mm

.**Table 2**- Properties of materials.

Material	Compression strength (N/mm^2)	Tensile strength (N/mm^2)	Modulus of elasticity (10^4 N/mm^2)
Concrete	20.1	1.84	2.11
	Yield strength (N/mm^2)	Modulus of elasticity (10^5 N/mm^2)	
Steel (reinforcement)	369.7	2.05	
Steel (beam)	360.8	2.05	

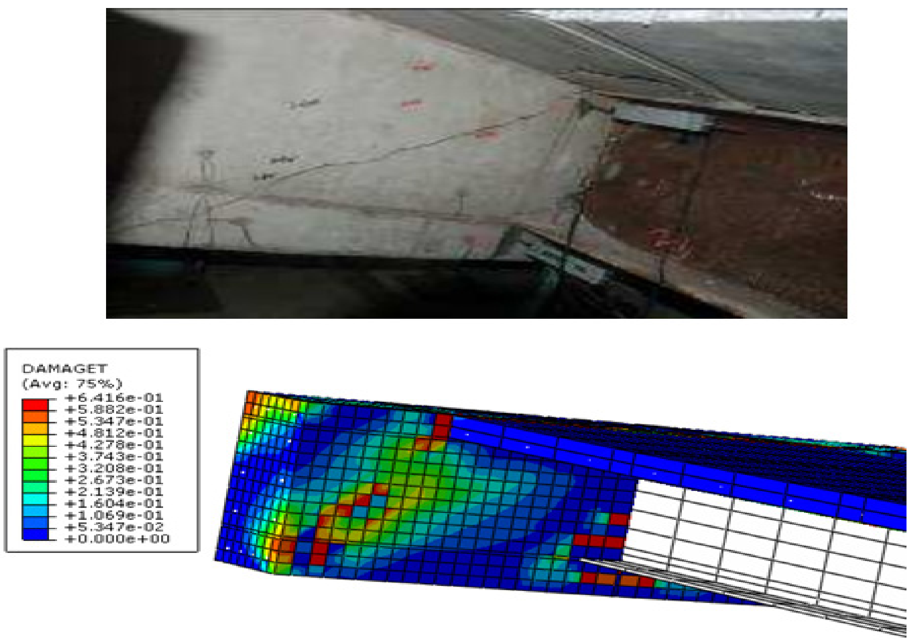

Fig. 5. Crack pattern of the numerical and experimental models(Yu et al., 2011).

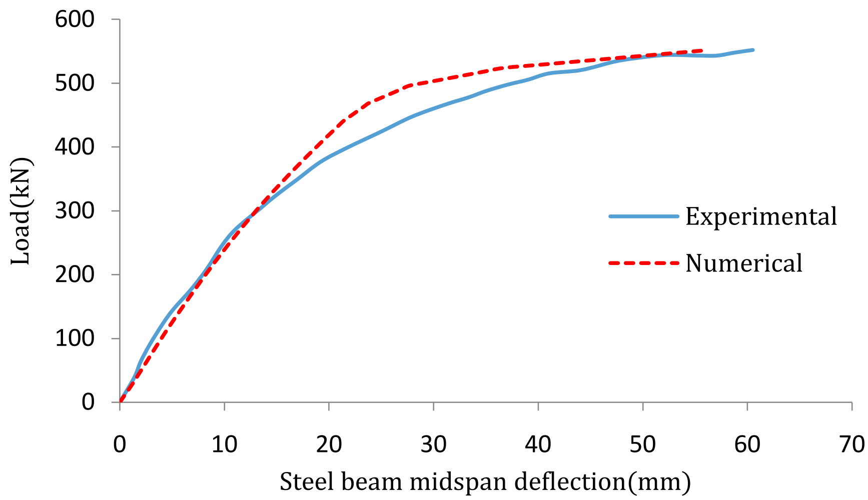

Fig. 6. Load vs. displacement relationship of numerical model and experimental model

4. Finite Element Investigation of the Steel Joist-Concrete Girder Connection

4.1. Overview

As mentioned in previous sections, ABAQUS software was used for evaluating the performance of composite connection of steel joist embedded in concrete girder.The influence of embedment length of steel joist in concrete girder on the bending capacity of the connection as well as the performance of double web angle shear connectors embedded in concrete were investigated(based on the specifications outlined in Table 3). $L/_h$ is the embedment ratio where L is the embedment length and h is the height if the steel joist. Besides, angle shear connectors of $(a \times a \times b)$ are of leg length a and thickness of b.

4.2. Sensitivity analysis

In order to investigate the sensitivity of the response to parameters, steel joist was modeled like a cantilever beam as shown in Fig. 7. A 200mm displacement was applied to the free end of the cantilever. Furthermore, the analysis was carried out under several embedment ratios of the steel joist (Fig. 8) and for each, a comparison was made with the cantilever type. Based on the results (Fig. 9), it is seen that $L/_h = 1.78$ provides the maximum bending capacity and acts like a rigid connection. Table 4 lists the increase in stiffness in relation to the given embedment ratios.

Table 3- Details and size of simulated cantilever model

Specifications		
Girder	Section	b×h=300×400 mm
	Longitudinal reinforcement	7-D20
	Tie	D10@200
	Length	5000 mm
Steel beam	Section	IPE140
	Length	1500 mm

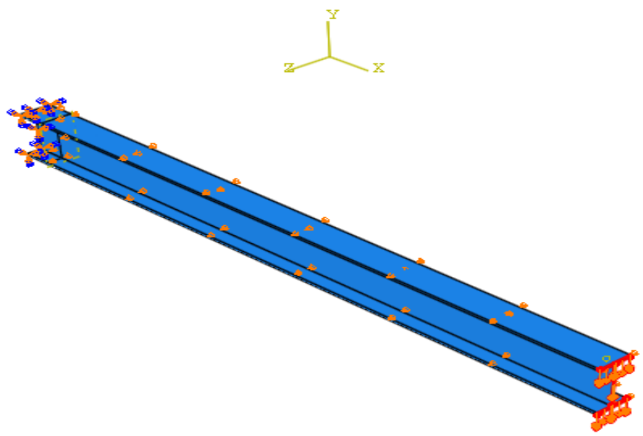

Fig. 7. Cantilever- Steel beam

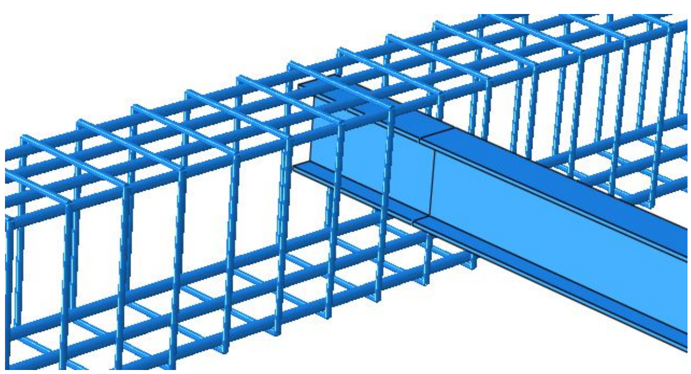

Fig. 8. Embedded steel beam in concrete girder

L/h	Increase of capacity (%)	Failure force
0.57	-	7.64
1	31%	13.24
1.78	24%	16.95

Table4- Increase of bending capacity and failure force in relation to $(L/_h)$

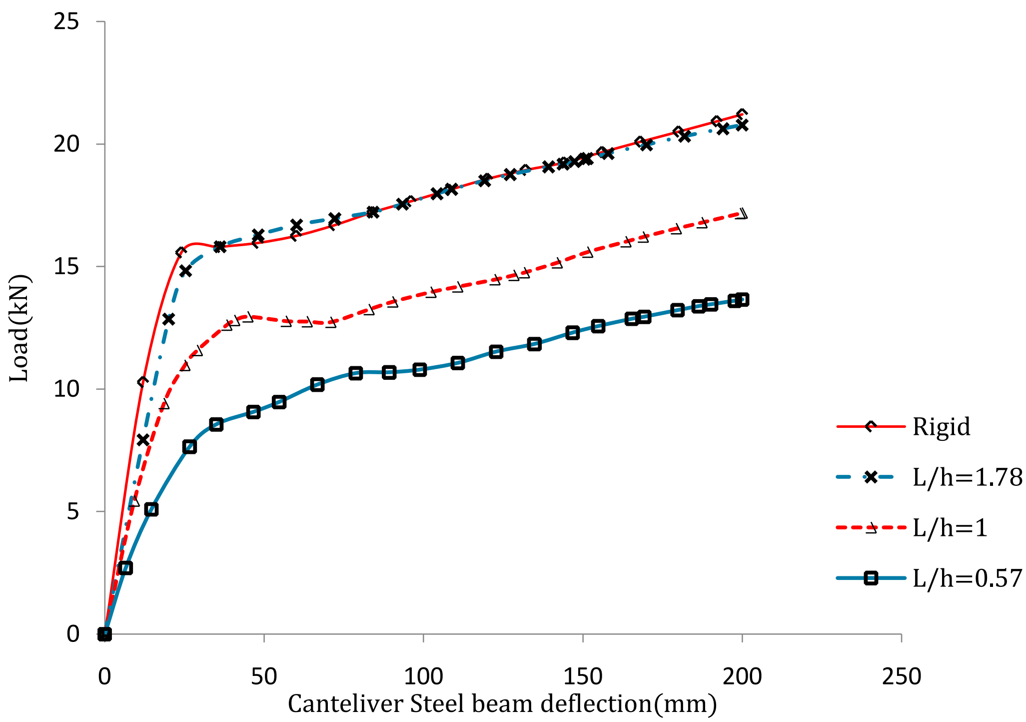

Fig. 9. Influence of embedment ratio on the bending capacity of the connection

4.3. Investigation of the double web angle connection in the embedment region

In order to investigate the influence of web shear connector in the bending capacity of the connection, double web angle shear connectors were employed in the embedment region $(L/_h = 1)$. As shown in Fig.10, a nonlinear static analysis was carried out with the same boundary conditions as before. Moreover, a comparison was made between the load-displacement curves of the analysis with that of the cantilever beam (Fig. 11).

Results indicate that the connection with $L/_h = 1$ and double web angle connections ($L40 \times 40 \times 4$) yields the maximum bending capacity. Therefore it is deduced that this connection provides economical detailing with a decrease of 40% in embedment length when compared to the $L/_h = 1.78$ case. In addition according to Fig.11, for a given $L/_h$ ratio, shear connector increases the bending capacity by 20%.

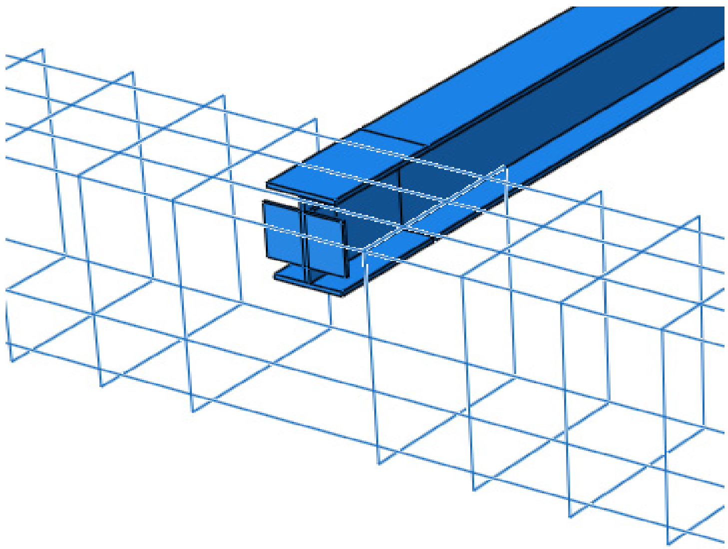

Fig. 10. Double web angle ($40 \times 40 \times 4$)

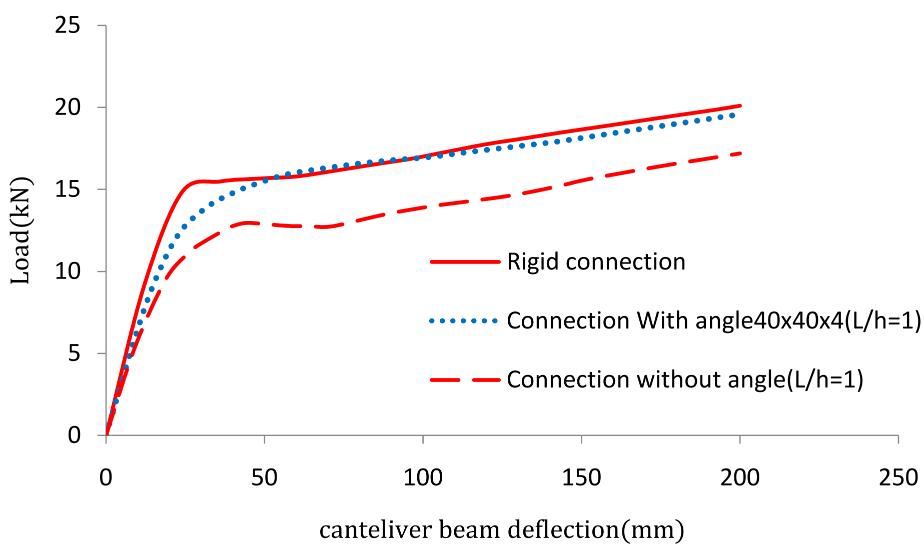

Fig. 11. Bending capacity of the embedded connection with web shear connector

5. Influence of Damageof Connection on the Bending Capacity of the Concrete Girder

In order to study this case, bending capacity of the concrete girder with and without steel joists (Fig.12) was analyzed considering Fig.13 and Fig.14. Results are presented in table5.

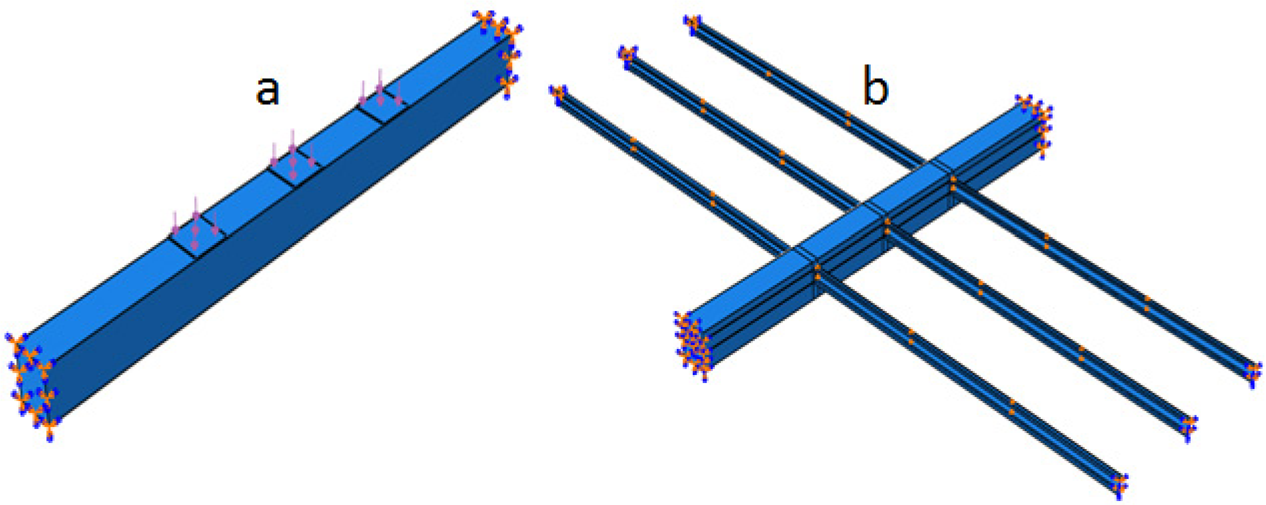

Fig. 12. Concrete girder with and without steel joists

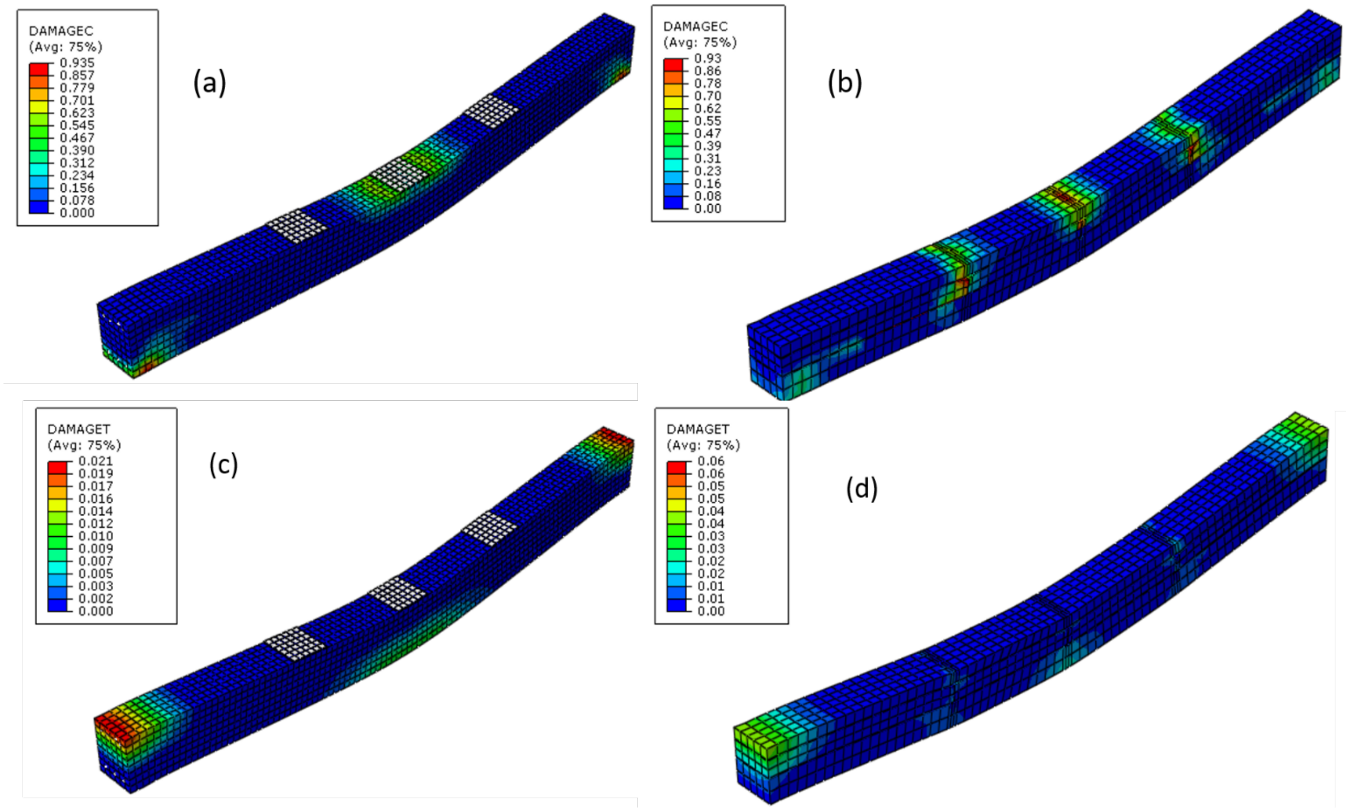

Fig. 13. Damage contours of concrete girder in ultimate load:(a) compressive damage to a girder without steel joist (b) compressive damage to a girder with steel joist(c) tensile damage to a girder without steel joist (d) tensile damage to a girder with steel joist.

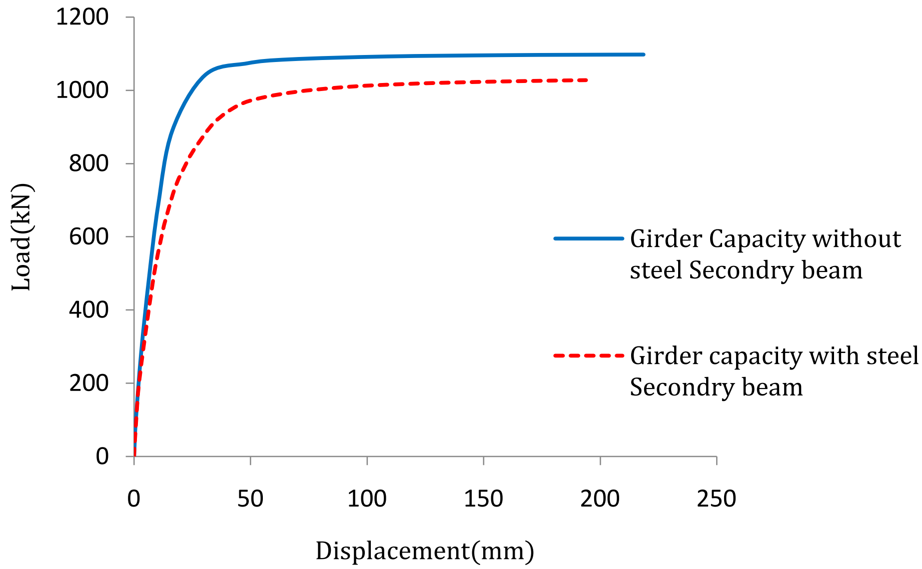

Fig. 14. Comparison of the bending capacity of the concrete beam with and without double web angles

Member	Bending capacity of concrete beam (kN)	Decrease in strength (%)
Concrete girder without steel joist	1092.39	_
Concrete Girder with double web angle shear connectors embedded in concrete	996.692	10%

Table 5-Reduction of bending capacity of the concrete girder with double web angle connection

6. Conclusion

Taking all above-mentioned discussions into consideration, the following conclusions can be drawn:

Embedment ratio of $L/_h = 1.78$ without any shear connectors provides the maximum bending capacity of the connection. Meanwhile, using double web angle shear connectors reduces this ratio to $L/_h = 1$. In other words, the interaction between the double web angles and concrete prevents the slipping of the steel joist inside the concrete girder.

Moreover, taking into account the concrete damage plasticity in the analysis, it is observed that bending capacity of the concrete girder is reduced by 10% in presence of steel joist.

Acknowledgement

The author gratefully acknowledges the support by Yu, Qiong for providing the concrete damage plasticity data.

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Nomenclature

D: Diameter of reinforcement bar
L: Length of embedment
h: Height of steel beam
a: Length of angle's leg
b: Thickness of angle