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Comprehensive Performance Evaluation of the Composite Connection of Steel Joist Embedded in Concrete Girder

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ABSTRACT

Great deal of studies has been, so far, conducted on the performance of Composite Reinforced Concrete-Steel (RCS) beam-to-column connections. This paper deals with performance of composite connection of embedded steel joist in concrete girder with appropriate numerical analysis. The proposed model is validated by experimental data presented in reference studies. performance of connection of steel beam to concrete girder are, however, assessed through different approaches including influences of embedment ratio, double web angle, size of web angle, tie distances, studs, steel beam with flanges cut in connection zone and plates in web of steel beam. As a result, an appropriate embedment ratio is offered to achieve maximum bending capacity of the connection. Using double web angles at the embedment region, however, reduce the ratio. Damage analyses show that bending capacity of the concrete girder slightly reduces in the connection zone. Better performance of steel plate installed in web at connection zone is also observed among composite connection approaches. Using low tie distances at connection zone increases capacity by 10%. Performance of double web angle is, further, compared under hysteretic and monotonic loadings. The ratio of L/h in hysteretic behavior of connection was 20% higher than that of monotonic loading.

Key words: Composite Beam-To-Column Connection, Embedment Length, Bending Capacity, Steel Joist-Concrete Girder Connection

INTRODUCTION

In recent years, composite connections have gained popularity among researchers due to the optimal usage of concrete and steel in resisting the applied forces to the structures. Few specific guidelines are available for the connection of steel secondary beams embedded in reinforced concrete girder. For this reason, investigating the behaviour 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 having 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. Currently few guidelines are available for steel joists embedded in concrete girder. Moment-resisting frame structures of high ductility class were studied (Salvatore et al., 2005). Some researches were done on the behaviour of confined concrete using Drucker-Pruger type plasticity model in ABAQUS (Yu et 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 damages to RCS connection in high seismic risk zones were investigated (Parra-Montesinos et al., 2003). The seismic behaviour of the steel beam-to-reinforced concrete column connection with and without floor slab was studied (Cheng and Chen, 2004). Composite frame structures having high-strength concrete columns confined by continuous compound spiral ties and steel beams were studied (Li et al., 2012). Seismic behaviour 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 researchers have been carried out on the interaction of shear force-bending

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moment in steel joist-concrete girder connections and proposed some equations (Yu et al., 2011). In this study, a specific length of steel joist was embedded in concrete with an angle shear connector. Hence, embedment length and its calculation are crucial.

MATERIAL AND METHODS

Finite element model

A. *General descriptions*

In order to simulate the real behaviour of the connection, four components need to be modelled:

1) Contact between steel joist and concrete girder in the embedded region.
2) Contact between steel joist and concrete slab.
3) Interaction between reinforcing bars and concrete.
4) Contact between anchor bars and concrete girder.

B. *Material model*

The mechanical behaviour of concrete was simulated using a Concrete Damaged Plasticity (CDP) model for which the pertinent parameters were estimated by uniaxial stress. Stress- strain relationship is shown in figure 1.

C. *Material Modelling of Reinforcing Bars*

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

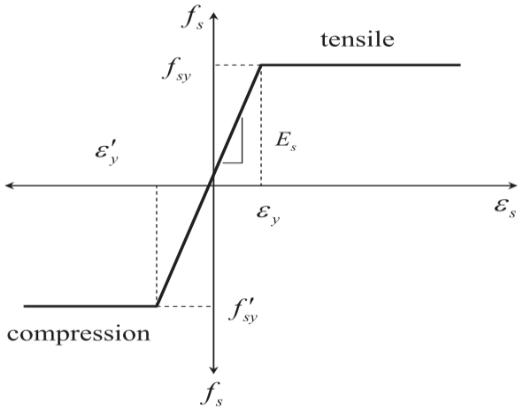

Figure 1. Stress–strain relationship of reinforcement (Li et al., 2012)

D. *Contact model between concrete-reinforcing bars, concrete and steel*

Interaction model between concrete and bars is of embedded type and frictional behaviour has been adopted for the contact between steel joist and concrete girder with friction coefficient of 0.7. Moreover, assuming no slip between steel joist and concrete slab, "tie" option is used in ABAQUS to simulate the behaviour. Loading and boundary conditions of the verified model are shown in figure 2 in which two ends of the concrete girder are completely fixed and the load is transmitted to the concrete slab via four plates.

Validation of 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 in figure 3. Also, Crack pattern of the aforementioned models is shown in figure 4. 12354 8-node linear brick, reduced integration (C3DR8) solid elements plus 397 2-node linear 3-D (T3D2) truss elements were employed to the model. Details of reinforcements and their properties are shown in Tables 1 and 2. Mid-span force-displacement relationship of the steel joist is shown in figure 5. As it can be seen model behaves linearly till the 25mm deflection (corresponding force, 390kN). Afterwards, when the load reaches 550kN, steel joist slips inside the concrete girder causes failure and damage.

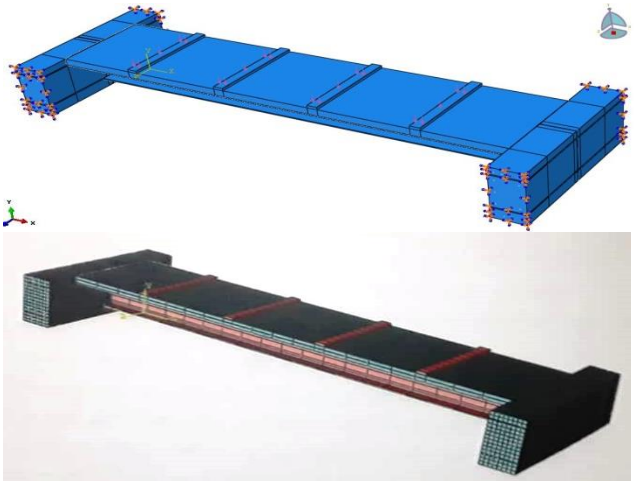

Figure 2. Loaded and meshed model

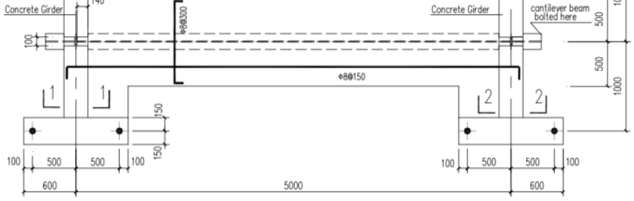

Figure 3. Specimen model (Yu et al., 2011)

RESULTS AND DISCUSSION

Finite element investigation of steel joist concrete girder connection

A. 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 new finite element method was employed to investigate 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 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×a×b) are of leg length, a, and thickness of, b.

B. Influence of embedment ratio (L/h) on the bending capacity of connection

In order to investigate the so-called parameters, steel joist was modelled like a cantilever IPE240 beam as shown in figure 6. A 200mm displacement was applied to the free end of the cantilever and the load vs. displacement diagram which is indicative of the bending capacity of the capacity was drawn. Furthermore, steel joist was analysed under several embedment ratios (Figure 7) in concrete girder and a comparison was made with the fixed type (load vs. displacement) for each ratio. Based on the results, 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. Figure 8 also shows validity of the ratio with IPE270.

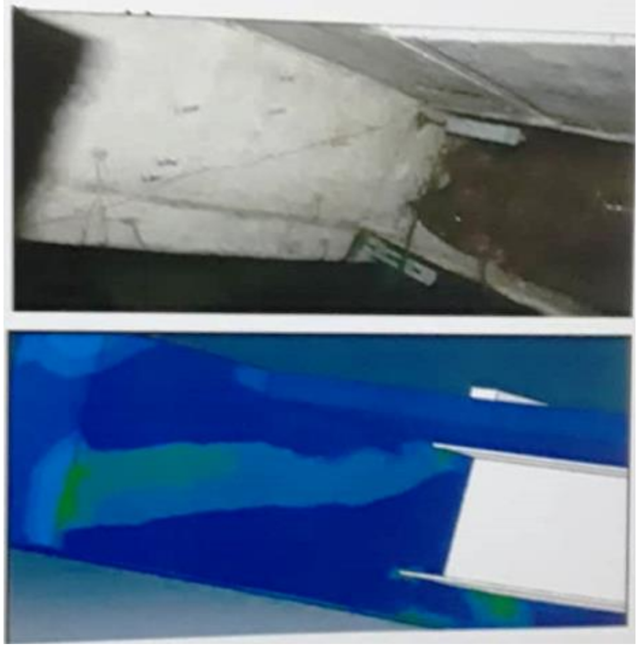

Figure 4. Crack pattern of the numerical and experimental models (Yu et al., 2011)

Table 1. Details and size of specimen

Detailed parameters of the specimen				
Member	Section (mm^2)	Longitudinal reinforcement	Tie	Length (mm)
Girder	320*900	12-D20&8-D20	D12 @200	2000
Slab	-	23-D8	-	-
Steel Beam	HN400×220×10×12	-	-	5000

Table 2. Properties of Materials

Specimen	Compression strength (N/mm^2)	Tensile strength (N/mm^2)
Concrete girder & slab	20.1	1.84
Steel reinforcement	369.7	2.05
Steel beam	360.8	2.05

Table 3. Details and size of cantilever model

Detailed parameters of the specimen				
Member	Section (mm^2)	Longitudinal reinforcement	Tie	Length (mm)
Girder	300*400	7-D20	D10 @200	5000
Slab	-	23-D8	-	-
Steel Beam	IPE150	-	-	1500

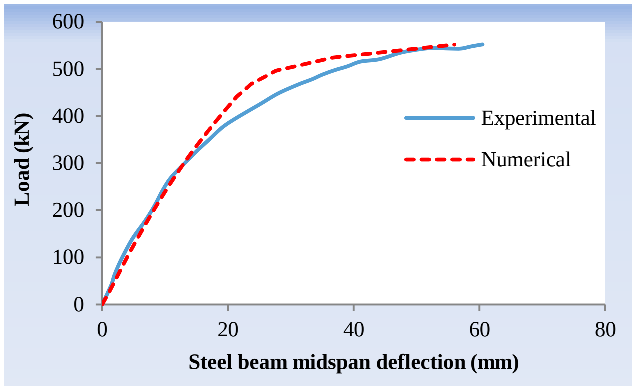

Figure 5. Comparison of load-displacement diagram obtained from numerical and experimental analyses

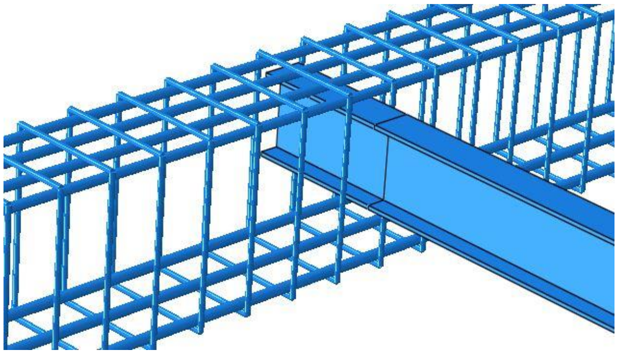

Figure 6. Embedded steel beam in concrete girder

Table 4. Increase of bending capacity and failure force in relation to (L/h)

L/h	Capacity increasing (%)	Failure force
0.57	-	7.64
1	31	13.24
1.78	24	16.95

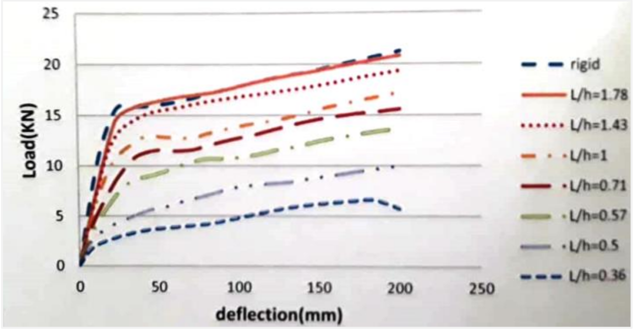

Figure 7. Influence of embedment ratio on the bending capacity of the connection

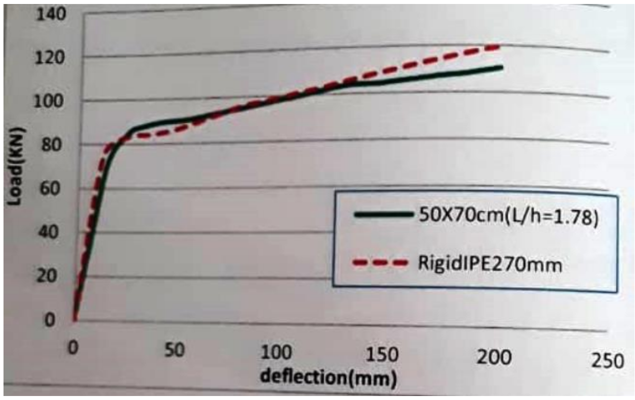

Figure 8. Load-deflection of IPE270

C. 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 figure 9, 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 (Figure 10). Results indicate that the connection with L/h=1 and double web angle connections (L 40×40×4) 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 figure 10, for a given L/h ratio, shear connector increases the bending capacity by 20%. Table 5, reports effect of size of angles on capacity of connection. As can be seen, thickness of angle has negligible effect on rigidity of connection while high efficiency of angle height is apparent.

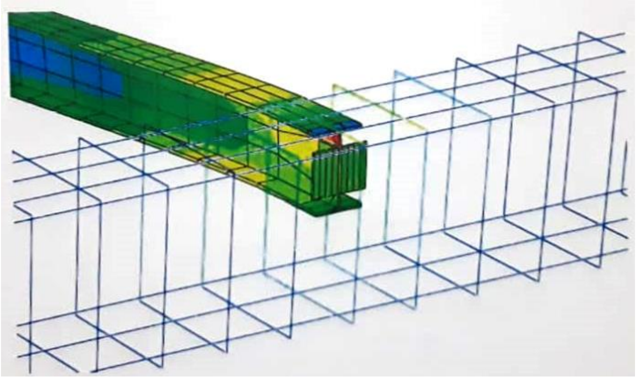

Figure 9. Connection detail of double web angle (40×40×4) in embedded region

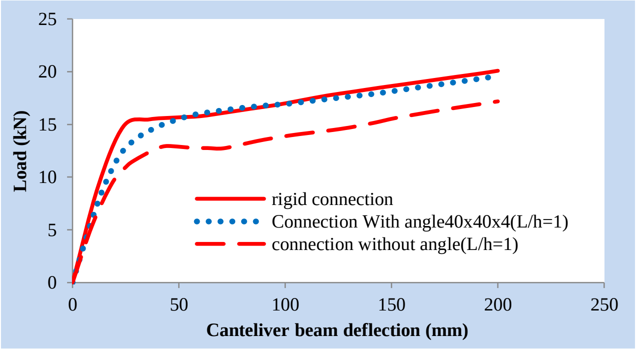

Figure 10. Bending capacity of the embedded connection with web shear connector

Table 5. Effect of angle size on bending capacity increasing

Size	Capacity increasing (%)
Angle 30×30×3	-
Angle 30×30×4	3
Angle 30×30×5	2
Angle 40×40×4	11

Influence of ties distances in connection on the bending capacity of the connection

Sensitivity of connection capacity to ties distances is assessed considering distances equal to 25, 50 and 100mm. Figure 11 demonstrates that as ties distances decrease, capacity of connection increases.

Influence of studs in connection on the bending capacity of the embedment region

Another detail of connection is evaluated based on using studs in steel beams web. Studs with 25mm diameter and length of 250mm are modeled in both sides of steel beam as shown in figure 12. Rigidity of connection increases in this detail as seen in figure 13. Further modelling is carried out with one stud in both sides of steel web which is shown in figure 14. Figure 15 shows sensitivity of this connection to diameter of stud. It is seen that effectiveness of diameter of stud on rigidity of connection is insignificant.

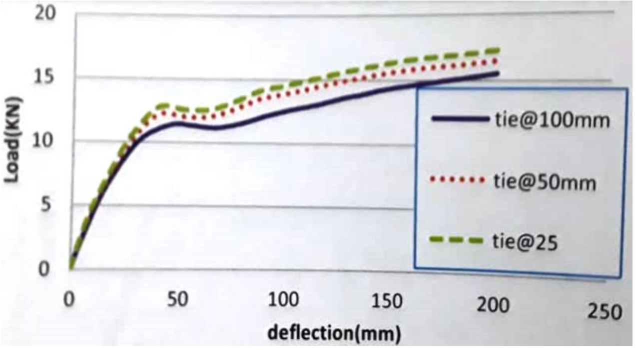

Figure 11. Effect of ties distances on connection capacity

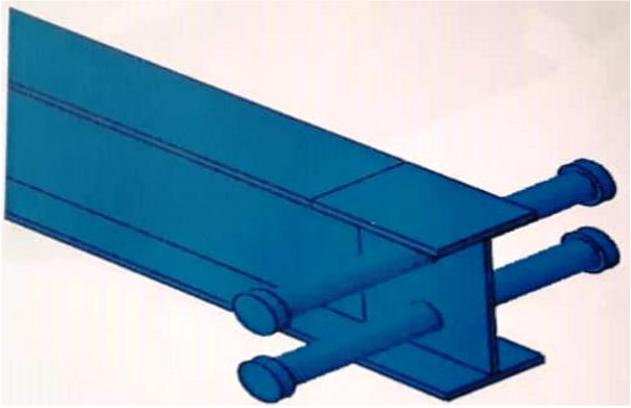

Figure 12. Detail of studs in embedded region

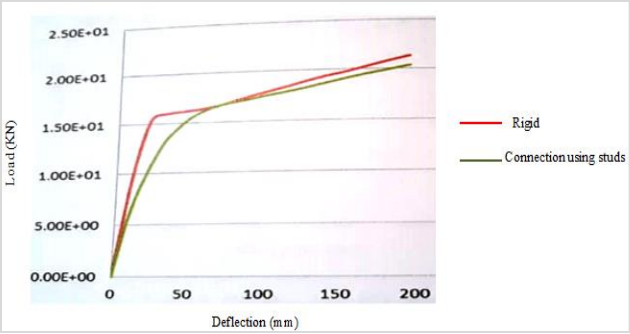

Figure 13. Effect of studs on rigidity of connection at embedded region

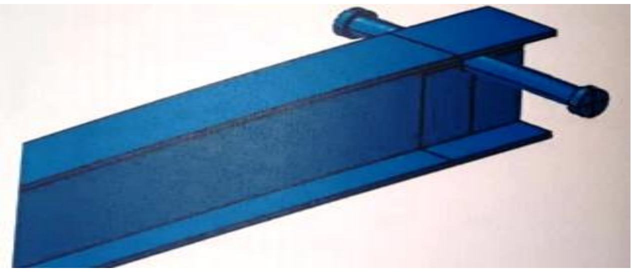

Figure 14. Detail of one stud in embedded region

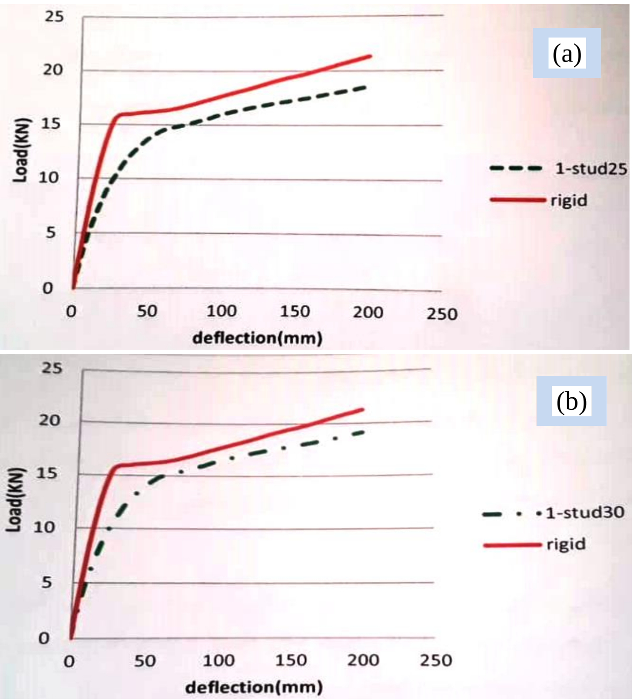

Figure 15. Effectiveness of diameter of stud on rigidity of connection; (a) 25mm; (b) 30mm

Influence of steel without flanges on the bending capacity of the embedment region

I-shape steel beams transmit bending moments through flanges while angles and stiffeners are used to make rigidity of this detail of connection. It is noted that holes made in web of beam are considered to bars of concrete beam in practical operations. Details are shown in figure 16. This kind of connection gives more damages to concrete girder in embedded region due to lack of flanges as seen in figure 17. Figure 18 shows that rigidity of this connection decreases by 17% respect to presence of concrete girder only.

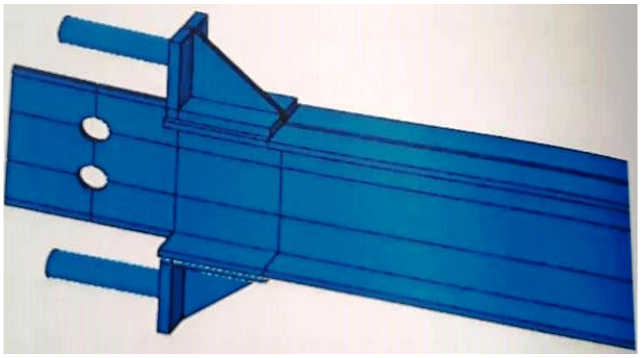

Figure 16. Detail of steel without flanges at embedded region

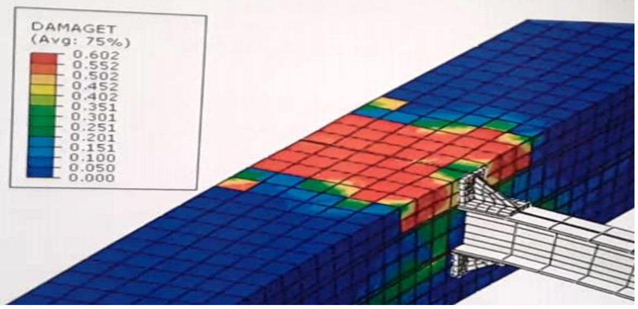

Figure 17. Tensile damages of connection at embedded region

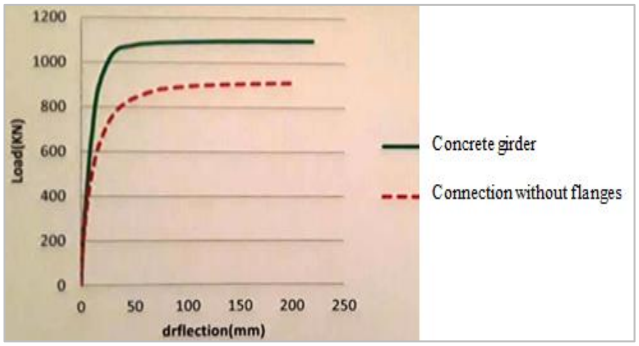

Figure 18. Reduction of connection rigidity in case of steel without flanges in embedded region

Influence of steel plate on the bending capacity of the embedment region

Steel plates are used to analyze capacity of connection rigidity as modeled in figure 19. Figure 20 shows that this detail reduces rigidity of connection up to 8%. Taking into account all kinds of abovementioned connections, it is concluded that semi rigid (or hinged) connection is approached with L/h=1 and using steel plate in web of steel joist. Capacity of both details is observed in figure 21. Concrete damage in embedded area is reduced using steel plate. Higher capacity of this detail is apparent.

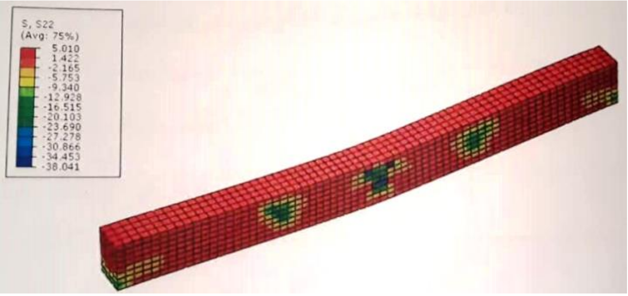

Figure 19. Modelling of connection with steel plate in embedded region

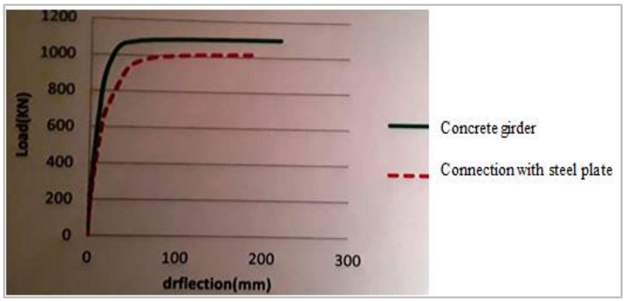

Figure 20. Effectiveness of steel plate on capacity of connection

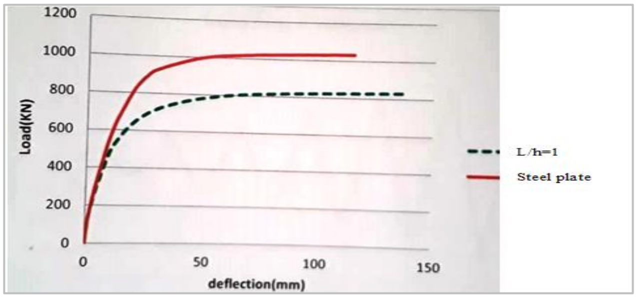

Figure 21. Comparison of semi rigid connections

Hysteretic behaviour of the double web angle connection in the embedment region

A cantilever beam with length of 1.5m using double web angles (L 40×40×4) and L/h=1 connected to a 30 by 40 cm concrete girder with 2m length is subjected to hysteretic loading. Loading protocol is shown in figure 22. Connection capacity subjected to both monotonic and hysteretic loadings at L/h=1 is compared in figure 23. Latter shows lower capacity respect to former. The capacity of connection increases as embedment length increases and reaches as high as monotonic loading capacity with L/h=1. Figure 24 shows that the hysteretic loading capacity of connection with L/h=1.2 is as high as monotonic loading capacity with L/h=1.

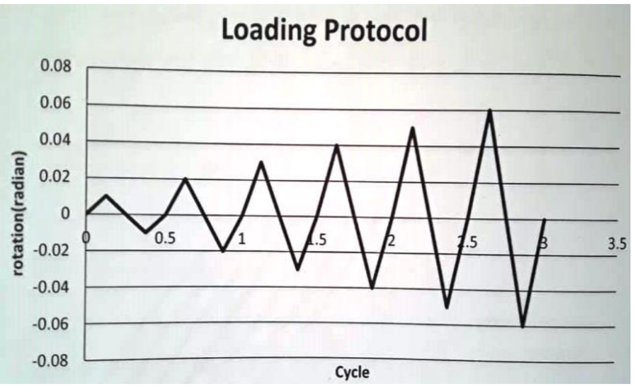

Figure 22. Hysteretic loading protocol

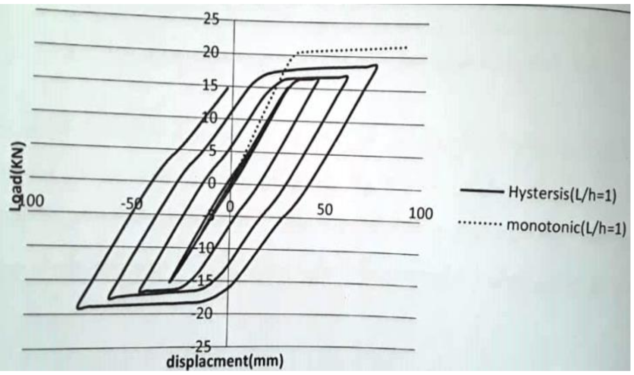

Figure 23. Comparison of connection capacity under hysteretic and monotonic loadings (L/h=1)

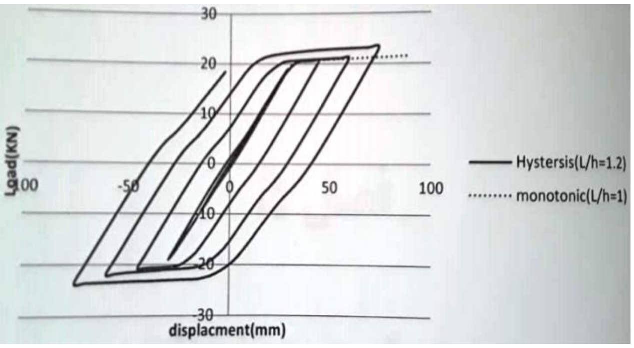

Figure 24. Comparison of connection capacity under hysteretic (L/h=1.2) and monotonic loadings (L/h=1)

CONCLUSION

In this paper, a numerical analysis through ABAQUS software was carried out to investigate behavior of embedded steel beam in reinforced concrete girder by various approaches consisting of influences of embedment ratio, double web angle, size of web angle, tie distances, studs, steel beam with flanges cut in connection zone and plates in web of steel beam. Embedment ratio of L/h=1.78 without any shear connectors provides the maximum bending capacity of the connection. Moreover, use of 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. Using shear connectors and studs at connection zone can decrease length of embedment and help connection be more practical. Bending capacity of the concrete girder was also reduced by 10% in presence of steel joist under analysis of concrete damage plasticity. Further conclusions can be drawn as below:

1. Using low tie distances at connection zone increases capacity by 10%.
2. The more studs at connection area, the more rigidity of connection
3. The ratio of L/h in hysteretic behavior of connection was 20% higher than that of monotonic loading.
4. In couple steel beam embedded in concrete shear wall, presence of shear connector can be ignored by embedding length of beam as much as twice of steel beam height.
5. Using plates installed in steel beam web at connection zone shows better capacity than that of steel beam with flanges cut at connection area.

DECLARATIONS

Authors' contributions

Authors of this research paper have directly participated in the planning, execution, or analysis of this study and have read and approved the final version submitted.

Conflict of interest

We hereby state that, there is no conflict of interest whatsoever with any third party.

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