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COLD-FORMED STEEL AND CONCRETE COMPOSITE BEAMS: STUDY OF BEAM-TO-COLUMN CONNECTION AND REGION OF HOGGING BENDING

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Abstract. *In Brazil, the cold-formed steel structure has been widely used justified by the large availability of steel sheets in the market (thin sheets) and the search for more competitive structural solutions. Thus as steel and concrete composite structures applied in small constructions, the traditional hot-rolled is replaced by cold-formed members in beams and columns. Even though the composite beams system is known in the hot-rolled and welded shapes field, the structural behavior of the cold-formed steel still needs more accurate investigation. A theoretical and experimental study about cold-formed steel and concrete composite beams was developed, focusing on the beam-to-column connection and the structural behavior on the region of hogging bending. For the experimental program two beam-column connections were analyzed to obtain the moment-rotation curves, the failure modes and check the application of the existent theoretical models.*

1 INTRODUCTION

It is denominated a composite steel-concrete system the one that has a steel (hot-rolled, welded or cold-formed) which works together with the concrete, forming a composed beam, composite column, composite slab or composite connections. The use of composite structures is much diffused internationally, in Brazil this type of structure is gaining space and popularity. Although the use of composite systems has always been restricted to hot-rolled and welded shapes steel, there is a tendency to the use of composite systems with cold-formed steel mainly in small buildings (up to 5 floors).

The behavior of the composite beams in cold-formed steel has particularities in regard to the composite in hot-rolled and welded shapes some researches on the structural behavior of the composite beams in the region at the positive moment were done, though a few studies talk about the region of hogging bending and the connections (beam-to-column connection or beam-to-beam). Both subjects are strongly jointed, since the continuance effect in the supports depends on the structural behavior of beam-to-beam or beam-to-column connection.

This paper aimed to evaluate the behavior of a beam-to-column connection, using double U lipped channels cold-formed steel to compose the beams and using seat angle and web angles to the connections, for the column it was used a HP hot-rolled steel. It is a type of connection analyzed by Leon et al. (1996) [1], but replacing the hot-rolled steel by cold-formed steel.

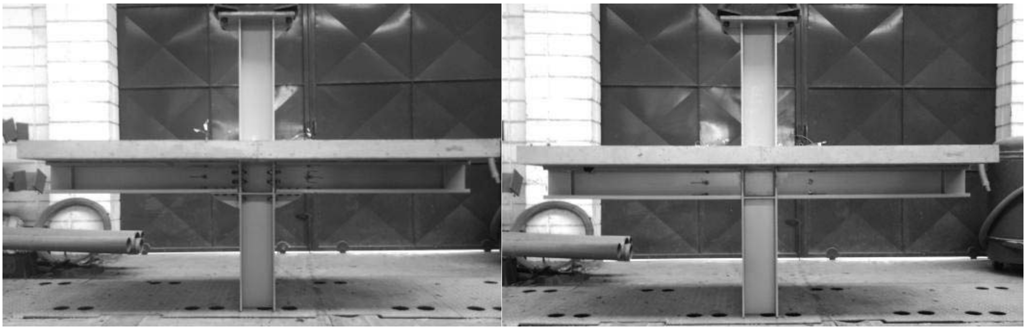
2 CLASSIFICATION OF THE COMPOSITE CONNECTIONS

Leon et al. (1996) [1] nominate the composite connections as an especial case of PR type connection as PR-CC (*partially restrained composite connection*), being made by a simple steel connection or semi-rigid, having its stiffness and resistance moment substantially increased. Considering the hogging bending, the reinforcement steel in the concrete slab to form the top portion of the moment resisting mechanism while the bottom portion is constituted in general by a steel seat angle and steel web angle providing the shear resistance.

3 EXPERIMENTAL PROGRAM

3.1 Models Description

In the experimental program two composite connection were analyzed (Figure 1): model 1 with steel seat angle and web angle to connect the beam to the column, which is detailed in Figure 2 and Figure 3, and model 2 with the same dimensions as model 1, but with a beam welded directly to the column. This model served as a reference since there is total continuance from the connection beam. Both models have a concrete slab (thickness = 100 mm; wide = 850 mm). The detailing of the main reinforcement slab is presented in Figure 4.



a) model 1 b) model 2

Figure 1 – General view of the models

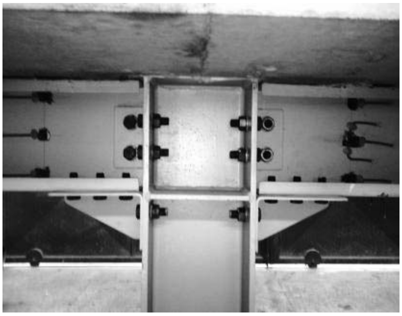


Figure 2 – Connection of the model 1

The channel shear connectors were made of a cold-formed steel (U 75x40x3.00) with 130 mm of length welded to the beam, and in an enough amount so there was complete interaction between the beam and the slab.

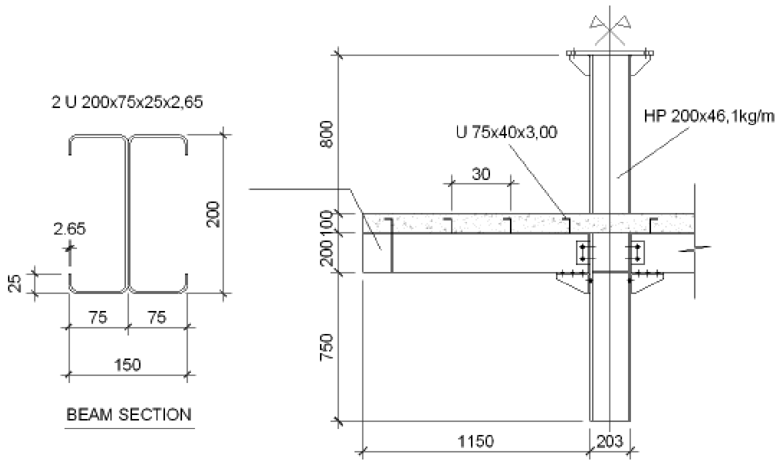


Figure 3 – General drawing of model 1

The beam was making in a way that all the elements had their effective width equal to its own width (compact section).

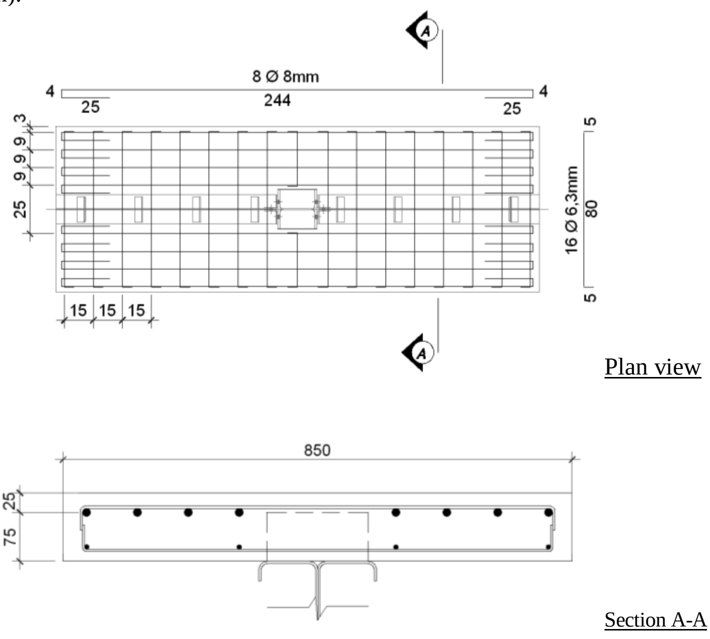


Figure 4 – Detailing of the reinforcement of the slab

3.2 Instrumentation and loading application

The instrumentation used is indicated in Figure 5, and was part of the strain-gages, displacement transducers in the beams and on the column to evaluate the relative rotation as well as the inclinometers put on the top part of the slab.

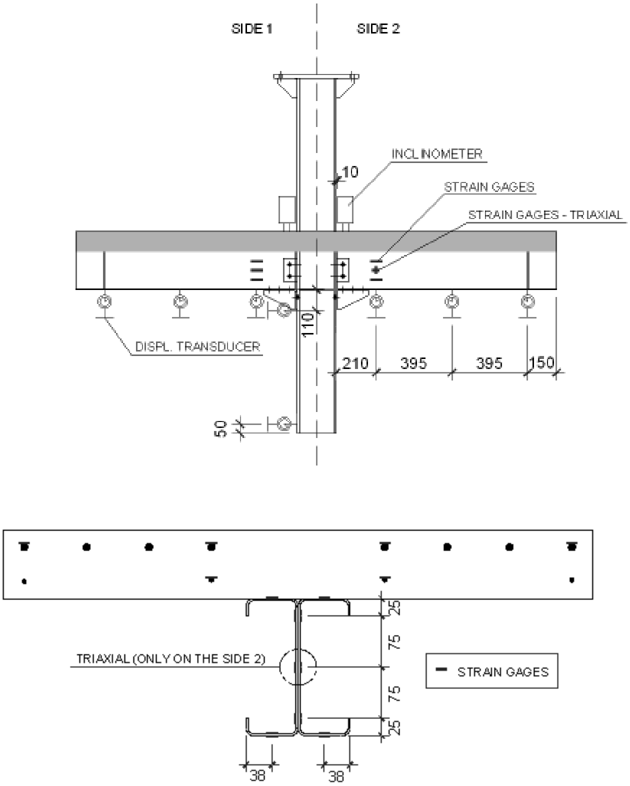


Figure 5 – Typical instrumentation of the models

The force was applied positioning a frame reaction on the model's extremities, to 1000 mm from the column face, and a third frame in the center of the model with hydraulic actuator, which applied an upward generating that way other two downward vertical forces on the extremities and consequently a hogging bending (negative moment) on the beams (see Figure 6).

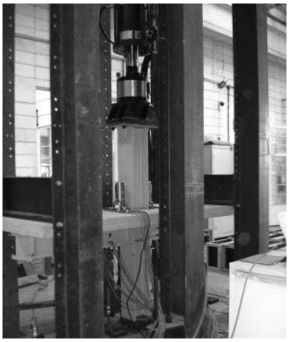


Figure 6 – Force application device

4 EXPERIMENTAL RESULTS

The values obtained from mechanical properties of the beams and reinforcement slab are presented in table 1.

Table 1: Mechanical Properties of the steel

Steel	f_y (MPa)	f_u (MPa)	Elongation (%)
Beams	275	385	36
Reinforcement	613	718	-

The maximum force applied by the hydraulic actuator (F_{max}) and corresponding hogging bending ultimate strength (M_{max}) for each model are presented in table 2.

Table 2: Values of F_{max} and M_{max}

Model	F_{max} (kN)	M_{max} (kN.cm)
model 1	133.5	6,675
model 2	132.1	6,605

The average of the strain obtained by the strain-gages for 25%, 50% and 90% of F_{max} is presented in Figure 7. For model 2 these values were deleted in this paper but they can be found in Mairal & Malite (2010) [2]

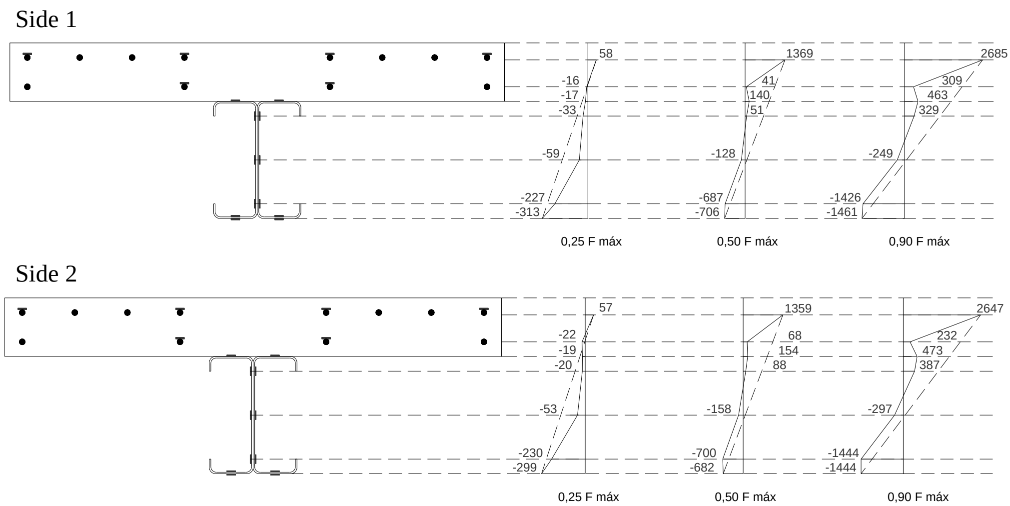
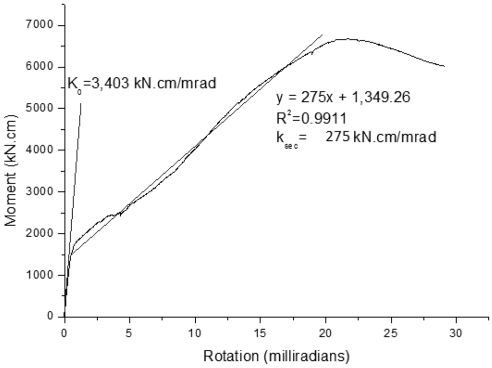
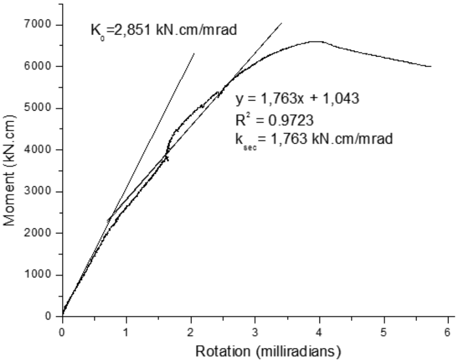


Figure 7 – Strains in model 1 (x 10^6) – average values

Figure 8 shows the Moment-Rotation curve (M-θ) for models 1 and 2.





a) model 1 b) model 2

Figure 8 – Average M-θ curve

The beginning of the cracking process for model 1 occurred for an approximated moment of 1,750 kN.cm now for model 2 this process started at 500 kN.cm.

In both models, the failure mode was characterized by the formation of plastic mechanism on the beams closed to the connection (see Figure 9), simultaneously to the beginning of the yield stress of some slab reinforcements.

a) model 1 b) model 2

Figure 9 – Mechanism of failure observed in the models

5 ANALYTICAL RESULTS

The composite connection was estimated according to ABNT NBR 8800 (2008) [3] (method of components) and according to Leon et al. (1996) [1]. The hogging bending ultimate strength of the composite beam was calculated admitting the steel section and concrete in the cross section are fully plastic (plastic stress distribution method), procedure adopted for hot-rolled and welded steel shapes with compact section, and also admitting a linear distribution of strains across the section (elastic method) recommended by ABNT NBR 14762 (2009) [4].

The value of the shear connector stiffness used in the method of components was obtained through push-out tests accomplished by David (2007) [5].

5.1 – Composite Beam (negative moment)

The value of the hogging bending ultimate strength according to the plastic method was $M_{R,pl.}$ = 7,380 kN.cm and for the strength compatibility method was $M_{R,el.}$ = 4,939 kN.cm.

5.2 – Composite Connection

By the method of components (NBR 8800) it was obtained an secant stiffness k_{sec} = 432 kN.cm/mrad, a hogging bending ultimate strength of the connection M_{max} = 6,654 kN.cm and the rotation capacity θ_u = 24 mrad. According to Leon et al. (1996) an initial stiffness k_i = 3,391 kN.cm/mrad, secant stiffness of k_{sec} = 2,196 kN.cm/mrad, hogging bending ultimate strength of M_{max} = 8,750 kN.cm and the rotation capacity of the connection θ_u = 23.6 mrad. The graphic moment-rotation according to Leon et al. is presented in Figure 10.

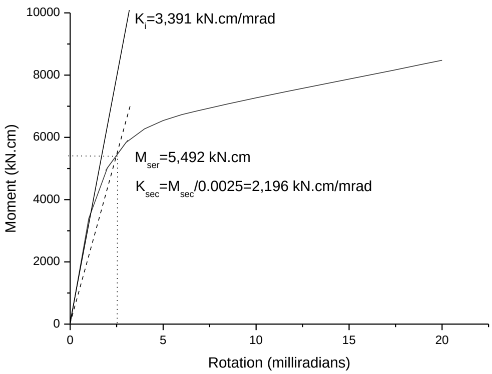


Figure 10 –M-θ curve obtained according to Leon et al. (1996)

6 ANALYSIS OF RESULTS

For the plastic method the value of M_{max} resisted by the composite beam (hogging bending) was 9% superior to the M_{max} experimentally obtained by model 1, and by the strength compatibility method was 26% inferior.

Figure 11 presents moment-rotation curve from models 1 and 2, and Leon et al. (1996).

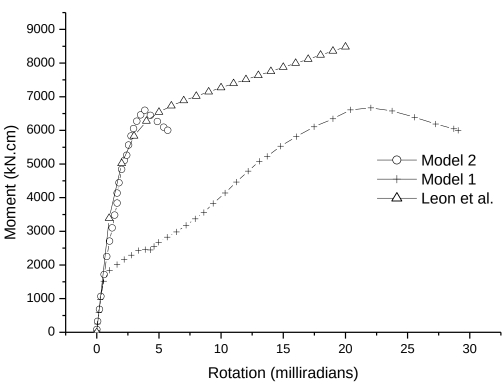


Figure 11 – Comparison of the M-θ Curves (model 1, model 2 and Leon et al.)

Table 3 presents the comparison of the initial stiffness (k_i), secant stiffness (k_{sec}), ultimate strength (M_{max}) and rotation capacity (θ_u) from theoretical and experimental results for model 1.

Table 3: Theoretical and experimental results for model 1

Model	K_i (kN.cm/mrad)	K_{sec} (kN.cm/mrad)	M_{max} (kN.cm)	θ_u (mrad)
Theoretical - NBR 8800 (method of components)	-	432	6,654	24
Theoretical – Leon et al.	3,391	2,197	8,750	23.6
Experimental - Model 1	3,403	275	6,675*	21.5

* Failure mode in the composite beam

7 CONCLUSION

In the case of composite beam the ultimate strength (M_{max}) experimentally obtained presented an intermediate value to the ones obtained by the plastic and elastic method, having its value closer to the plastic method as can be seen in item 5.1. Seeing that the failure mechanism occurred in the beam thus it was not possible to evaluate experimentally the ultimate strength of the connection and can be classified as fully strength connection (FS) when they are capable of transferring the full moment capacity of the beam. From a point of view on the stiffness the connection might be classified as semi-rigid (PR – Partially Restrained).

The moment-rotation curve of model 1 presented an abrupt change of stiffness after the beginning of the process of cracking of the concrete slab, presenting a behavior different of model 2. The rotation capacity of model 1 was about 6 times superior to model 2 as can be seen in Figure 10, showing more ductility of this type of connection when compared to the rigid connection (model 2).

The method of the components in this case estimate relative well the secant stiffness and rotation capacity presenting a much closer value than the experimental one, although have not taken into account the stiffness of the web angle. By Leon et al. procedure it is well estimated the initial stiffness and the rotation capacity, but the value of the secant stiffness is much larger than the experimental one.

REFERENCES

[1] Leon, R.T.; Hoffman, J.J.; Staeger, T., *“Partially restrained composite connections”* AISC Steel Design Guide Series, 8. Chicago, USA, 1996.

[2] Mairal, R.; Malite, M., “Cold-formed steel and concrete composite beams: study of beam-to-column connection and region of hogging bending” (in Portuguese) - School of Engineering of Sao Carlos – University of Sao Paulo – Final Report, 2010.

[3] Brazilian Code, *“Design of steel and composite structures for buildings”* (in Portuguese), ABNT NBR 8800, 2008.

[4] Brazilian Code, *“Design of cold-formed steel structures”* (in Portuguese), ABNT NBR 14762 – Draft 2009.

[5] David, D. L.; Malite, M. *“Theoretical and experimental analysis of shear connectors and composite beams constituted of cold-formed steel beams and floor system with lattice girder beams*” Doctoral thesis (in Portuguese), School of Engineering of Sao Carlos – University of Sao Paulo, 2007.