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Effect of the Degree of Connection on Local Buckling of Continuous Composite Beams

Mohamed Bensoula [1]* & Mohamed Tehami [2]

1. Department of Civil Engineering University of Mostaganem, Algeria.
*Corresponding Author Email: Mohamed.bensoula@univ-mosta.dz
2. Department of Civil Engineering, USTO Oran, Algeria. Email: mohamedtehami@yahoo.fr

Abstract

The study of the effect of the connection on local buckling of continuous composite beams is highly important, especially at the neighborhood of intermediate supports where negative bending affects the connection sizing. Sections at intermediate supports of continuous composite beams become more critical because of the flexural stiffness reduction due to slab cracking and the tendency of the neutral axis to rise up in the steel web leading to the change of class section according to Eurocode 4. The proposed analytical model allows to predict the rotation capacity of sections of class 2 according to Eurocode 4 for full or partial connection with a constant degree of connection. Therefore, the aim of this paper is to verify the validity of the proposed analytical model for various degrees of connection in order to generalize its use. Again, the theoretical results of the established model are confronted to experimental results and then validated for different degrees of connection.

Keywords: Degree of Connection, Local Buckling, Composite Beams, Eurocode, Rotation.

1. INTRODUCTION

Buckling of thin plates is a phenomenon of structural instability, complex and diversified in its form. Care should be taken in the case of a slender plane plate partially or totally loaded by compressive or/and /shearing internal forces in its middle plane. Local buckling influences the strength of a compressive plate with a lower thickness. In this context, codes of practice Eurocode 4 [1] and Eurocode 3 [2] have given a classification of cross sections in terms of bound slenderness of the web and flanges. The degree of connection in continuous composite beams has an effect on local buckling. The use of a partial connection can influence the local buckling of continuous beams, affecting their rotation capacity [3,4]. The connection model of a composite beam can be expressed in an integral format, indicating the nonlocal character of the beam's elastic interaction [5]. The influence of the local buckling instability on the internal moment redistribution coefficient between hogging and sagging zones is studied, and it is shown that the predictions made by Eurocodes may be questionable [6]. The rotation capacity of the central support in negative moment regions of continuous composite beams is affected by local buckling, limiting the beams' ultimate strength. Local buckling has a significant influence on the moment redistribution coefficient for composite continuous beams [7]. The presence of local buckling can affect the internal moment redistribution between hogging and sagging zones, challenging the standard predictions [8]. In the case of slender sections, local and distortional buckling can cause a reduction in the interior support moment prior to failure of the system [9]. The extent of internal force and moment redistribution in a composite continuous beam can be increased by exerting prestressing with external tendons. The moment redistribution in prestressed continuous composite beams is greater than that in non-prestressed composite beams at the ultimate state. Overall, understanding and considering the influence of

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local buckling is crucial for accurate design and prediction of the moment redistribution behavior in composite continuous beams. The vulnerability to local buckling of steel is more emphasized in regions where the hogging moments are generated, and the design in partial connection will be more difficult to control with regard to the required rotation capacity of cross section on intermediate supports. In this objective, code of practice Eurocode 4 [1] does not allow a partial interaction in regions where bending moments are negative. The mechanism of collapse due to local buckling of continuous composite beams is proposed by [3,4]. Their analysis is essentially based on plastic hinges models. Such models should be compatible with experimental behavior and applicable where a partial connection steel-concrete is considered. In reality, a certain slip between the concrete slab and the steel element will be developed, even though a full connection is considered. The aim of this work is to evaluate the influence of the degree of connection on local buckling of continuous composite beams.

2. PRESENTATION OF THE NUMERICAL MODEL

The phenomenon of local buckling of continuous beams with composite sections was undertaken by several investigations [10, 11, 12, 13, 14], mainly on intermediate supports positions where such behavior is more important.

The presented model allows to predict moment-rotation curves. On the intermediate supports, continuous composite sections are susceptible to buckle locally under full connection condition [4], and as well for partial connection [3]. The proposed mechanism is illustrated on Figure 1. It is characterized by the development of several yielding lines and two distorted surface areas in the web and the compression flange.

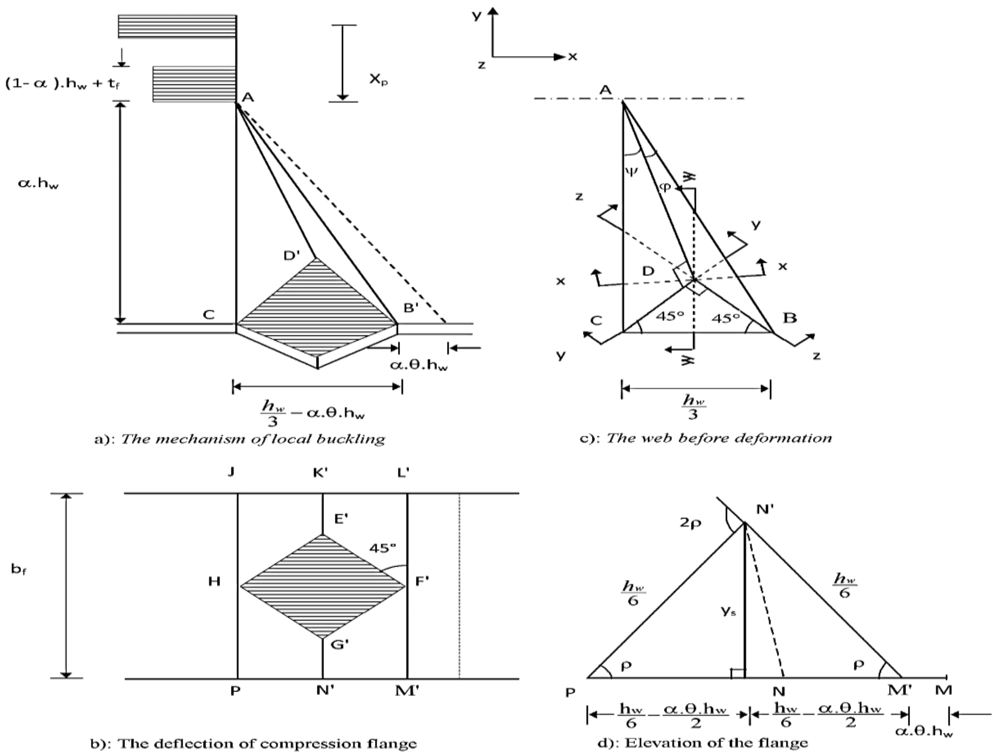

Figure 1: The model of collapse mechanism given by [3,4]

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This analysis is based on models with formation of plastic hinges and takes into account local buckling of the web with section of class 2, with the contribution of possible buckling or a plasticization produced by a deformation of compression flange, according to the class 1 or 2 of the cross section. The relationship M-θ is derived from the principle of conservation of energy during the collapse mechanism, where the work resulting from external forces is equal to the dissipated energy, in this way we may obtain the expression of moment by equalizing the dissipated energy to the absorbed energy such as:

$$M = \frac{\sum_i W_i}{\theta} \qquad (\text{with } \theta\square \neq \square) \tag{1}$$

with

M : The applied bending moment

θ: Neutral axis rotation around point A

$W_A = \sum_i W_i$: The total of the energies absorbed by each deformed part

The dissipated energies in different parts of the composite section at intermediate support (hogging moment) where details of the analysis are given in reference [3] are divided as such:

a) Dissipated energy in part above A, consisting of the longitudinal steel reinforcement area A_s of the steel section under tension.

b) Dissipated energy by the deformation of compression flange developed by a compression and a rotation simultaneously, around a central line of the flange of square EFGH and by energies due to the development of the plastic hinges FM, LF, PH, HJ, KE, GN, EF, FG, GH and HE.

c) Dissipated energy in the compressive web consists of the energy of the triangle BCD producing a plastic distortion and by energies of plastic hinges formation AC, AD, AB, BD and CD.

3. EXPERIMENTAL STUDY

3.1 Mechanical and geometrical properties

In the case of local buckling of continuous composite beam at intermediate supports and plastic hinges positions, the choice of the model characterizing the collapse mechanism is previously described [3]. Such model takes into account the laboratory experiment observations noticed during the tests conducted on many continuous composite beams at the laboratory of structures at INSA of Rennes (France) [4] Geometrical properties of the beam N°01 under consideration, is of the type IPE-A-330, are given in table 1 and illustrated on Figures 2 and 3.

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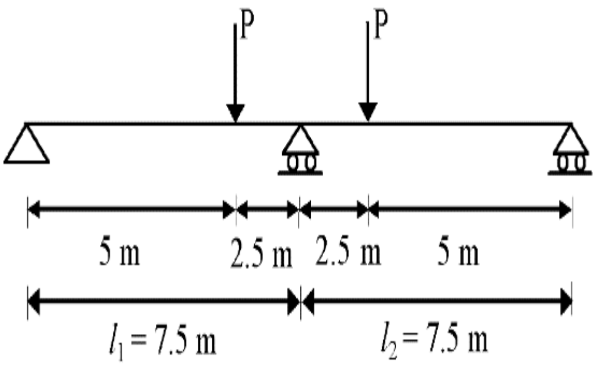

Figure 2: Loading diagram of the instrumented beam N°01

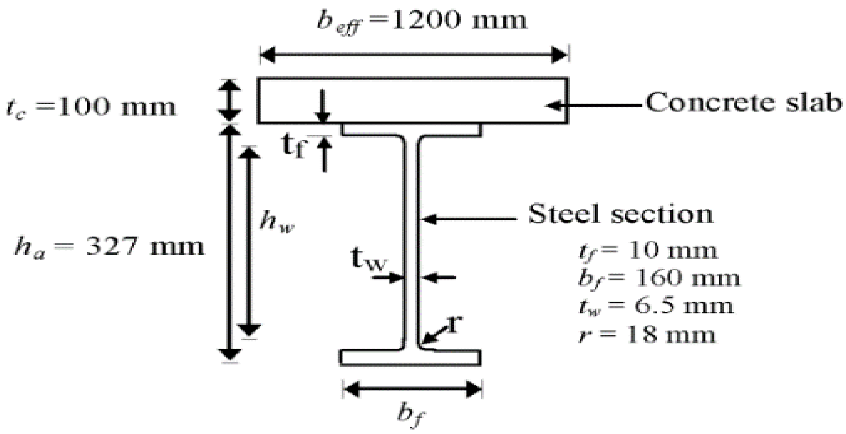

Figure 3: Details of section of the instrumented beam N°01(IPE-A-330)

Table 1: Geometrical properties of the cross section used in both the experiment and the numerical analysis of the developed model

Geometrical properties	value
Steel reinforcement in sagging moment (12ϕ10)	A_s = 942.5 mm^2
Steel reinforcement in hogging moment (12ϕ10)	A'_s = 942.5 mm^2
Mean value of concrete secant modulus of elasticity corresponding to the characteristic strength of concrete f_{ck}	E_{cm}= 29 kN/mm^2
Area of the steel section A_a	A_a = 5470 mm^2
Distance from the center of gravity of the steel reinforcement to the contact steel-concrete	g_s = 50 mm
Distance from the center of gravity of the steel section to the contact steel-concrete	g_a = 163.5 mm
Height of the steel beam	h_a = 327 mm
Web height of the steel beam	h_w = 271 mm
Web thickness of the steel beam	t_w = 6.5 mm
Flange width of the steel beam	b_f = 160 mm
Flange thickness of the steel beam	t_f = 10 mm
Reinforced concrete slab thickness	t_c = 100 mm
Reinforced concrete slab effective width	b_{eff} = 1200 mm
Radius of fillet	r = 18 mm
Area of the two root fillets	S_C = 139.06 mm^2
Moment of inertia of the steel beam section	I_a = 10230cm^4
Plastic strength modulus	Z = 702 cm^3

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Values of mechanical properties of the materials used in the theoretical analysis as well as in the experimental work are summarized in Table 2.

Table 2: Mechanical properties of the used materials

Concrete		Structural steelwork		Steel reinforcement	
f_{ck}(MPa)	f_{cm}(MPa)	f_y(MPa)	f_{ym}(MPa)	f_{sk}(MPa)	f_{sm}(MPa)
23.50	26.74	280	300	470	480

3.2 Loading Arrangements

The experimental tests were conducted on a test rig at the laboratory of structures at INSA of Rennes (France) under two portal steel frames, each one is equipped by a hydraulic actuator to load the instrumented beam.

The first hydraulic actuator (noted P_1) QUIRI brand has a maximal force of 1000 KN in compression and 500 KN in tension, with a stroke rod of 20 cm.

The hydraulic actuator (noted P_2) SCENCK brand has a maximal force of 1000 KN in compression and 600 KN in tension, with a stroke rod of 60 cm.

3.3 Measurements Monitored

The applied load and the resulting measurements during the test are taken simultaneously, and they are recorded automatically as shown in Figure 4 Such records concern:

- Longitudinal deflection (14 potentiometer sensors are positioned in the experimented beam).
- Vertical and longitudinal displacements of the intermediate supports (5 potentiometer sensors).
- Transverse displacement of the web at the intermediate support position (20 potentiometer sensors placed at either side of the support).
- Linear deformations of steel and concrete at different sections (around a hundred strain gauge).

During the tests, the loading is increased gradually at regular interval (from 50 KN at the start, then 10 KN in the nonlinear domain) until the collapse.

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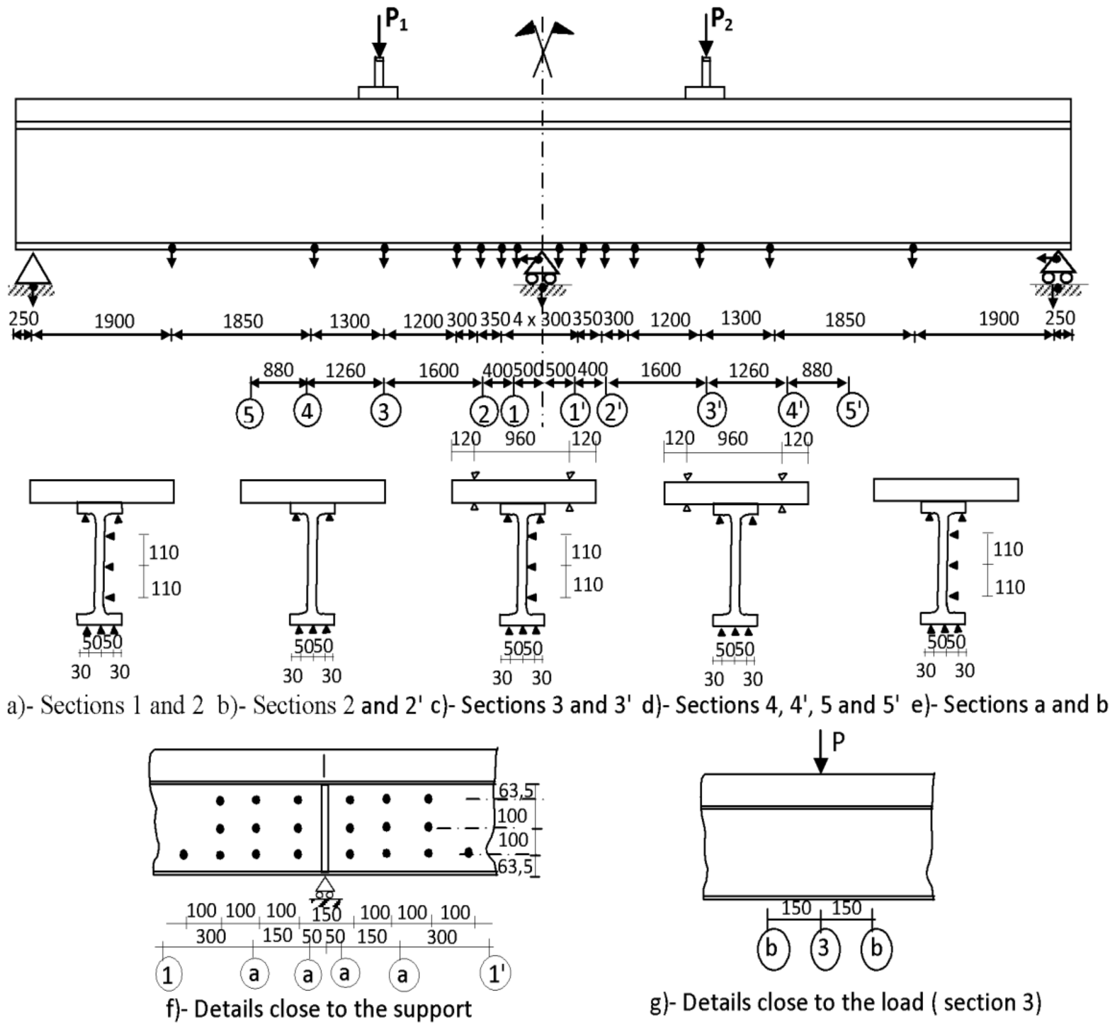

Figure 4: Details of test set-up of beam

3.4 Observations during the tests

During the tests, observations can be reported. Such constatations concern the behavior of structural material constituting the experimented beam, especially visible on particularly loaded sections.

Transverse cracking's resulting from longitudinal bending are noticed at the start of the loading on sections at intermediate support, and then cracks spread on all the slab height.

Local buckling was visible. Firstly, in the web beside the load P_2, it is generated by a wave formation at the support of the middle and then steel scale was released at the web of the other side of the support.

After the web local buckling, a beginning of a lower flange buckling has been noticed at the intermediate support, then a wave formation can be clearly observed. The shape of this wave resembles to the wave produced in the web.

Then, local buckling of the full cross section is highly accentuated. The collapse load is normally given by the unloading of the hydraulic actuators.

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4. THE CONNECTION UNDER SHEAR

The connectors should resist to the longitudinal shear force between the slab and the steel beam. The used connectors are of type Nelson headed concrete anchors of 16 mm diameter and of 75 mm in height. They can be considered as ductile according to paragraph 6.1.2(2) of Eurocode 4[1], and their ultimate strength in tension is: f_u= 450 N/mm^2. Connection under shear at limit plastic state allows to determine the number of connectors at different critical length of the beam.

In reality, the critical length between the applied load P and the central support is divided into two parts, the first part exists in the region where the bending moment is positive and the second part is located in the region where the bending is negative. These lengths are calculated from the bending moment diagram by the elastic analysis of the « cracked » structure, where the following results are obtained:

- The number of connectors on the critical length l_c = 5 m on the side of the support end: N_f = 26 connectors.
- The number of connectors on the critical length l_c = 2.5 m between the load P and the central support:
  - Sagging moment: $N_{f\,1}$ = 26 connectors
  - Hogging moment: $N_{f\,2}$ = 10

Hence, it is necessary to introduce the concept of degree of connection, which will be equal to: N/N_{f2} where N is the number of connectors chosen to be less than the required number $N_{f\,2}$.

In this work, we notice three connection modes:

- Case 1: Full connection (N =10 connectors): N/N_{f2} = 1
- Case 2: Partial connection (N =7 connectors): N/N_{f2} = 0.7
- Case 3: Partial connection (N =5 connectors): N/N_{f2} = 0.5

5. RESULTS AND DISCUSSIONS

The cross sections of class 2 are defined by Eurocode 4 [1] as compact sections susceptible to develop a plastic moment of resistance, even though local buckling restricts the rotation under a constant bending moment. The curve M_2-θ is an important testing property in the study of local buckling of continuous composite beams. It shows the evolution of bending moment on intermediate support in term of rotation. In the aim to study the capacity of rotation of the intermediate support in term of the loading of the beam, it is essential to evaluate the rotation under testing of the either side of the intermediate support. From the values of deflection, measured by means of displacement transducers between the load and the intermediate support, the expression of longitudinal deflection will be obtained for each span of the beam, in which the coordinates origin is located at that intermediate support, and therefore P-θ curve representing the change of the load in term of rotation will be deduced.

Increase of the rotation is highly accentuated as long as the loading moves to the collapse load. This shows the influence of local buckling on the ultimate strength of continuous composite beams, particularly on intermediate supports positions. At first, longitudinal deflections allow to appreciate the symmetric form of the beam comparing to the observed

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mode of collapse, whereas, the experimental values of the deflection will be used to determine the rotations of the intermediate support for the experiments in order to study an important property of local buckling, characterized by moment-rotation characteristics curve. The inelastic rotation on the intermediate support consists of a plastic rotation, due to a plastic deformation of the cross section, and another rotation generated by local buckling. This latter increases considerably, when moving closer to the collapse. For the three cases, the arrangement of connectors are, a full connection N/Nf = 1, a partial connection N/Nf = 0.7 and a partial connection N/Nf = 0.5 are respectively illustrated on Figures 5, 7 and 9. Experimental results of bending moments as well as the results of the developed model are drawn on Figures 6, 8 and 10 for the three cases under consideration. On the moment-rotation curve at full connection (Figure 6), it can be noticed that the plastic moment of resistance will be reduced by the interaction of combined effect of local buckling and vertical shear, it is about 250 KN.m, while on moment-rotation curves (Figures 8 and 9), the plastic moment of resistance is reduced by both, the shear force and the partial connection effects and it is evaluated to 230 KN.m when the connection N/Nf = 0.7 and 210 KN.m when the connection is N/Nf = 0.5.

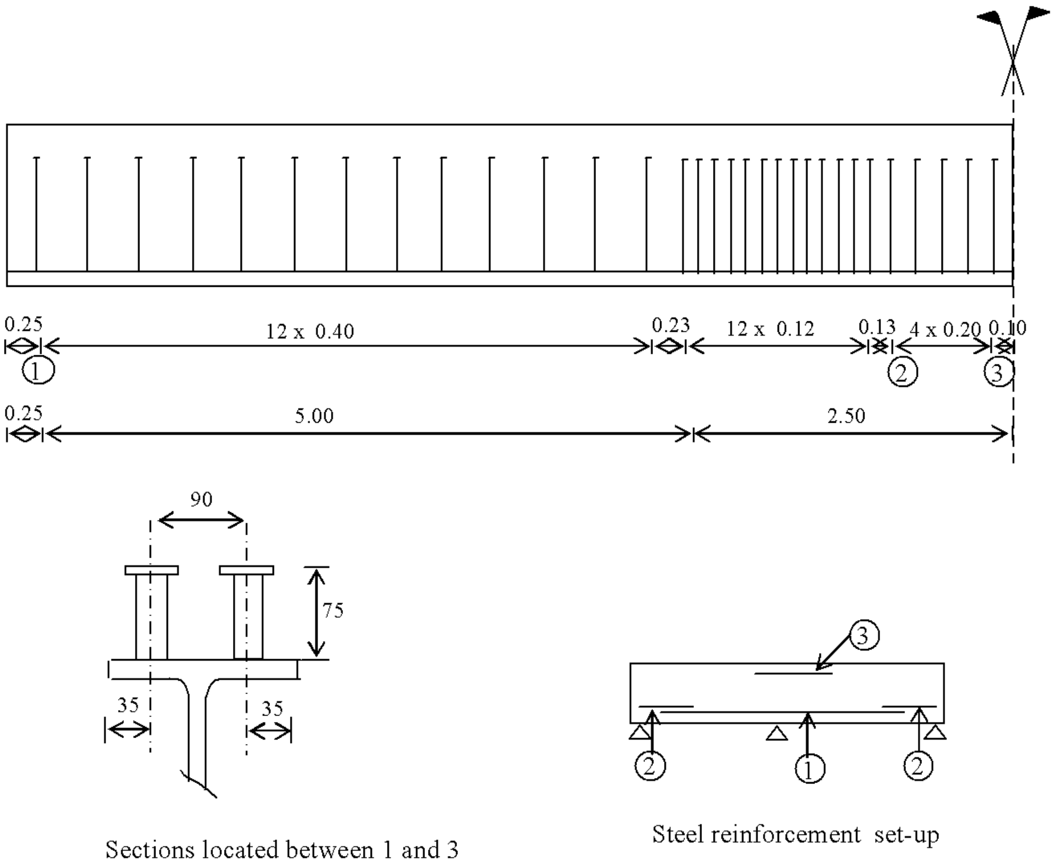

Figure 5: The connectors arrangement in the beam under test for Full Connection N/N_f = 1

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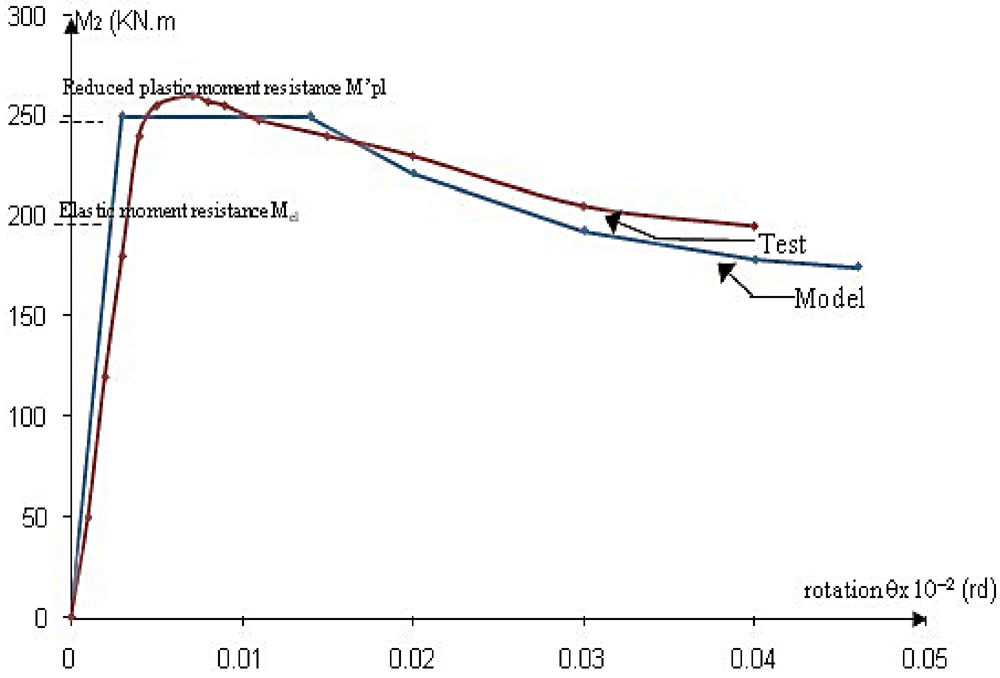

Figure 6: Analytical and experimental results for full connection N/Nf = 1

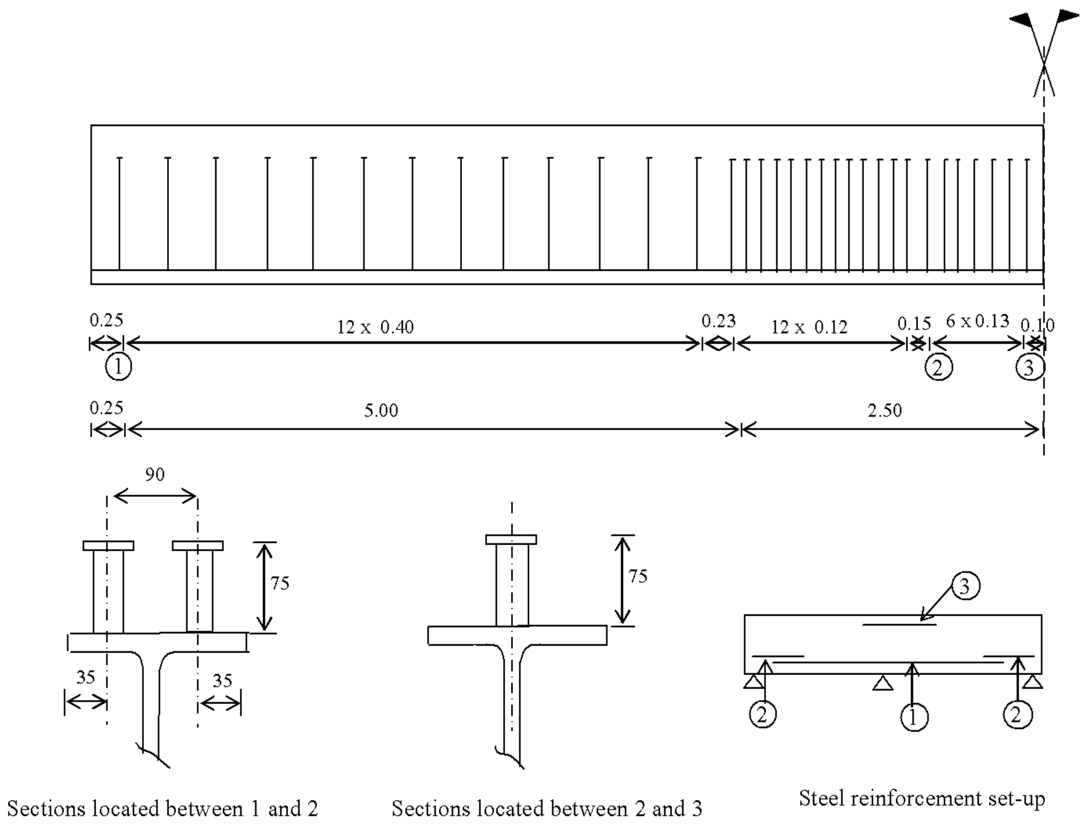

Figure 7: Connectors arrangement in the beam under test for partial connection N/N$_f$ = 0.7

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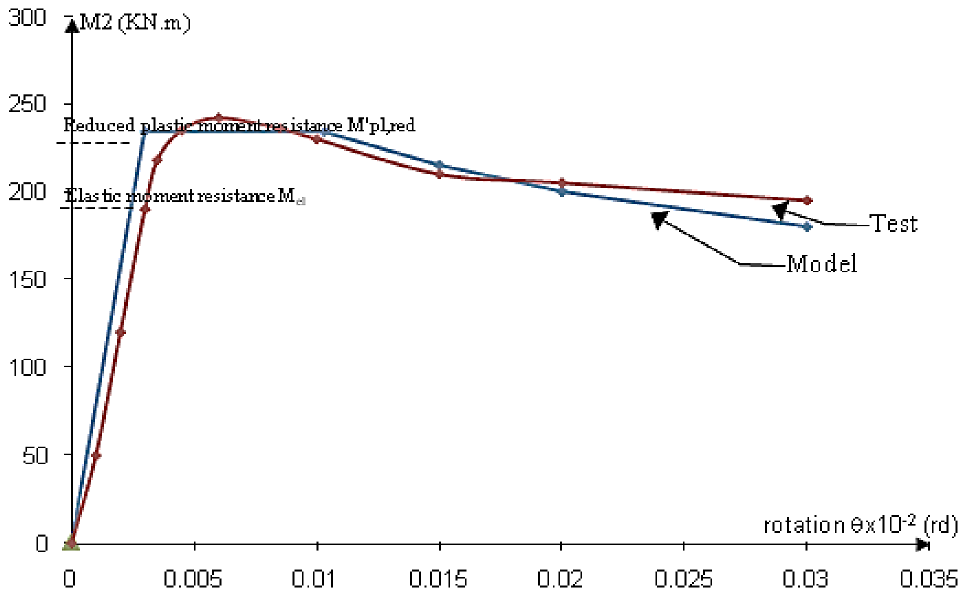

Figure 8: Analytical and experimental results for partial connection N/Nf = 0.7

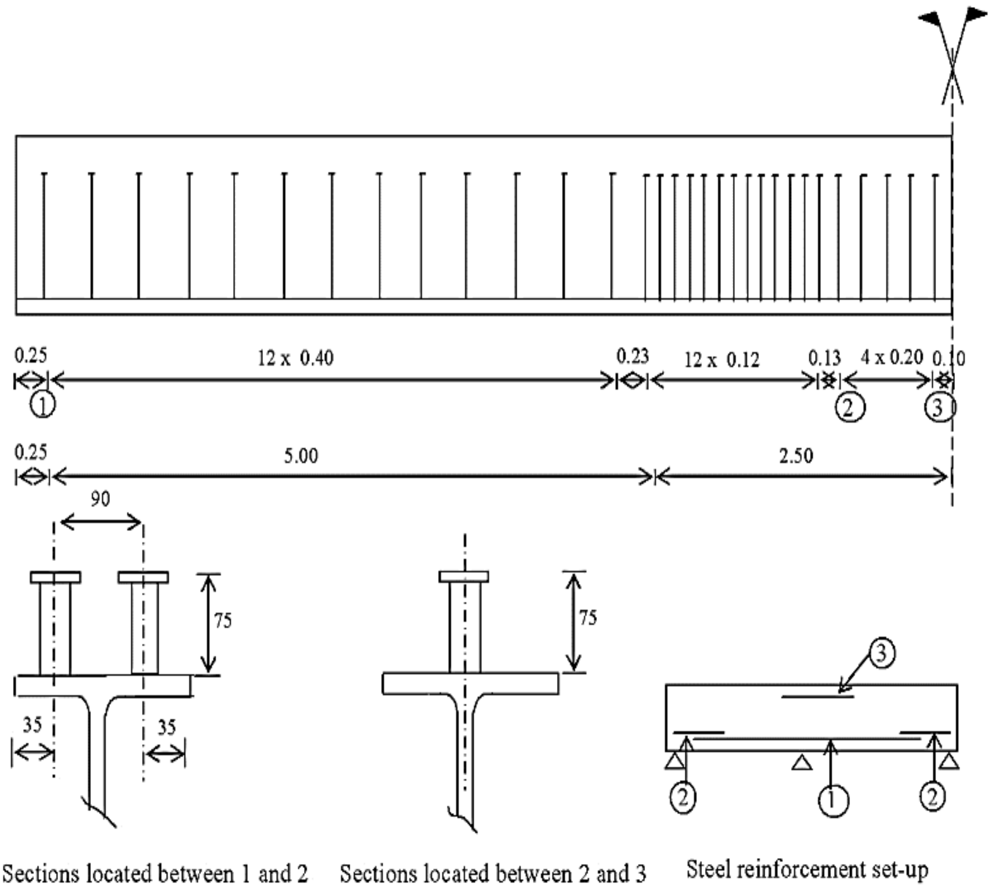

Figure 9: Connectors arrangement in the beam under test for partial connection N/N_f = 0.5

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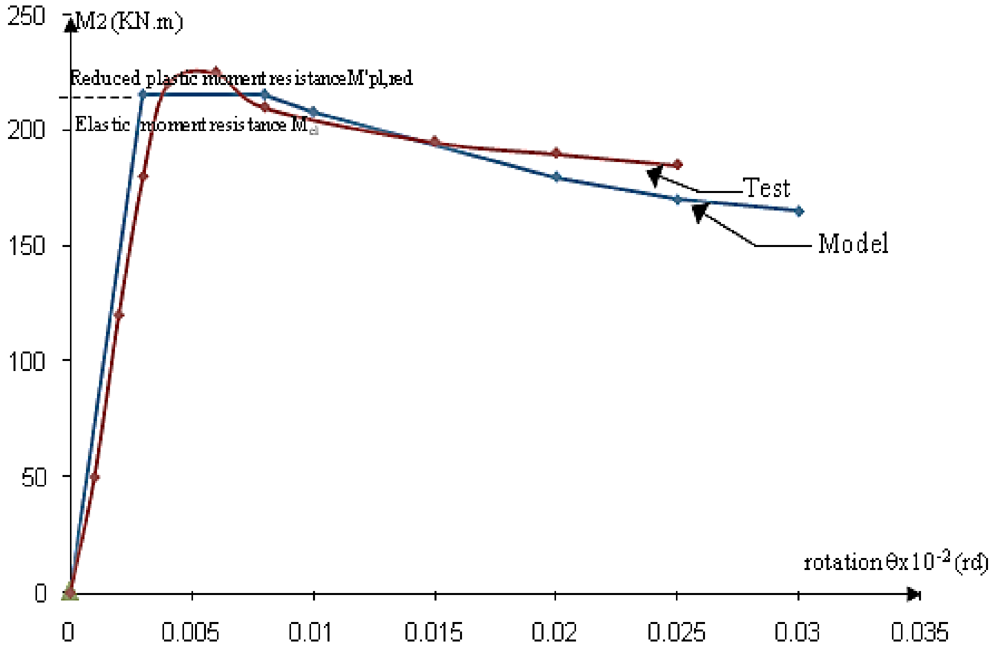

Figure 10: Analytical and experimental results for partial connection N/Nf = 0.5

Figure 11 shows the performance of the developed model for different connections confronted to the experimental results. They are represented in the same (Figure. 12).

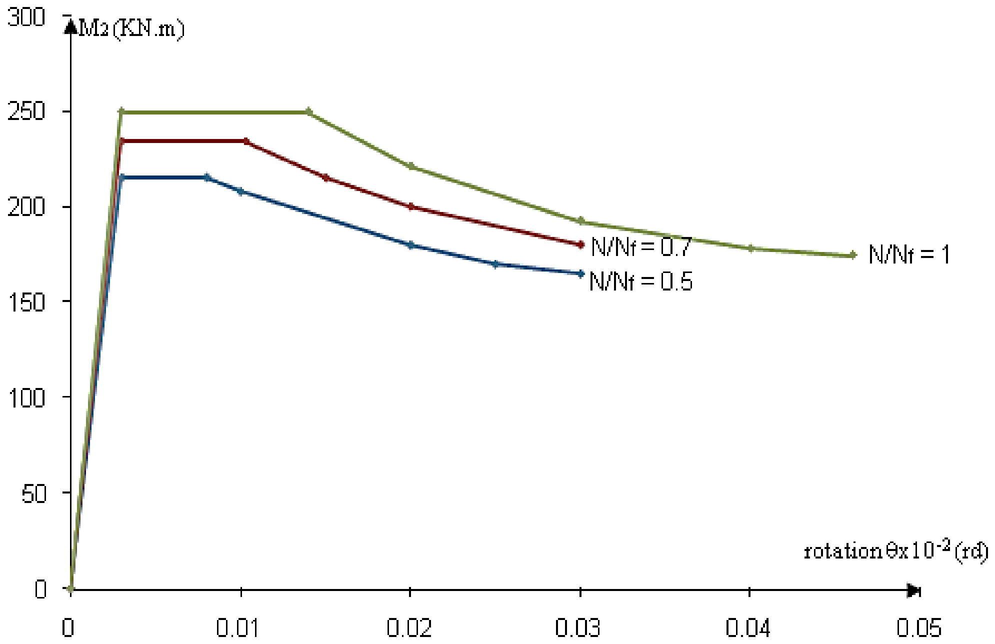

Figure 11: Model curves in term of the degree of connection

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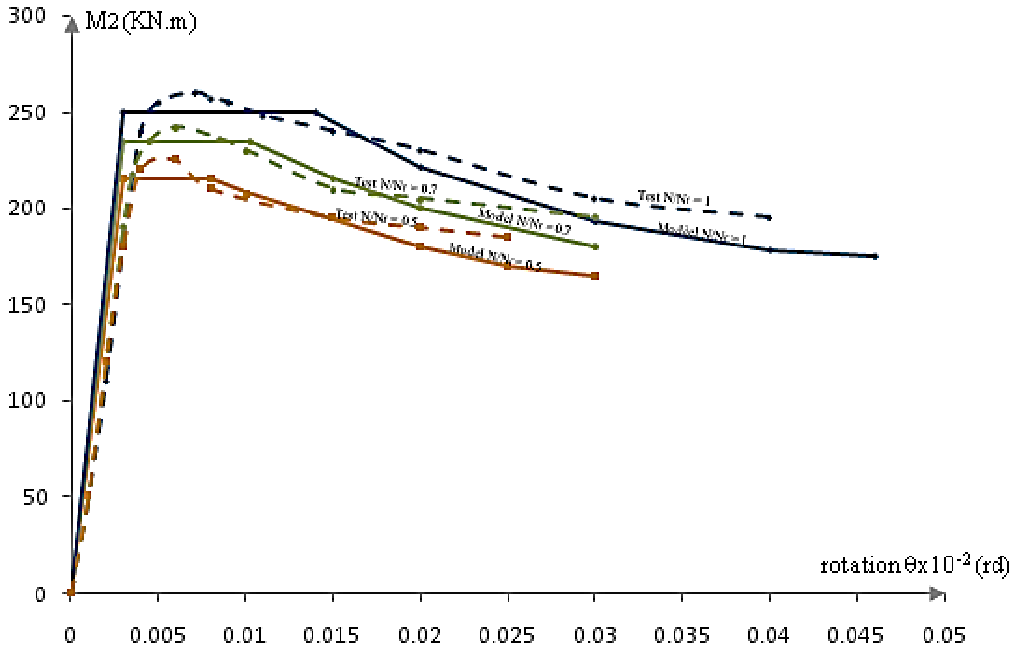

Figure 12: Analytical and experimental results in term of the degree of connection

6. CONCLUSIONS

The developed model may predict the moment-rotation characteristics curves on intermediate supports of continuous composite beams, where its cross sections, are vulnerable to buckling, on supports of class 2 for full and partial connection.

The established mathematical model expressing the collapse mechanism of local buckling of continuous composite beams is based on the limit theorems of plasticity. Such model confirms the representation of experimental observations and M-θ curves obtained (Figure 12) shows clearly such compatibility.

The present analytical model is able to reproduce the tests results for all cross sections of continuous composite beams of class 2 on intermediate supports susceptible to buckle with a degree of connection more or equal to 0.5.

A discrepancy between the developed model and the test can be reported. This is due to the rotation generated by local buckling, which increases considerably when loading moves closer to the collapse. This difference is of $\theta > 3.10^{-2}$ rd in the case of full connection, of $\theta > 2.10^{-2}$ rd when the connection is about 70% and of $\theta > 1.5.10^{-2}$ rd when the connection is 50%.

In the case of slab under tension, the experimental study confirms, that the connection is less rigid and its ultimate strength is reduced, moreover, this is the reason for which certain codes of practices codes (for example British standards BS 5950 [15]) suggest, in the case of composite beams of buildings, to adopt an ultimate strength of connectors P_{Rd}, in regions where the bending moments are negative in continuous beams less than 20 % approximately to the considered value in regions where the bending moments are positive. While, Eurocode 4[1] does not specify such requirement, but it does not completely allow partial interaction in regions where bending moments are negative in order to prevent local buckling susceptibility of steel.

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