The Role of the Studs Configuration in the Structural Response of Composite Bridges

M. M. Mohammadi Dehnavi, A. De Angelis, M. R. Pecce

***Abstract*—**This paper deals with the role of studs in structural response for steel-concrete composite beams. A tri-linear slip-shear strength law is assumed according to literature and codes provisions for developing a finite element (FE) model of a case study of a composite deck. The variation of the strength and ductility of the connection is implemented in the numerical model carrying out nonlinear analyses. The results confirm the utility of the model to evaluate the importance of the studs capacity, ductility and strength, on the global response (ductility and strength) of the structures but also to analyse the trend of slip and shear at interface along the beams.

***Keywords*—**Shear Load, slip, steel-concrete composite bridge, stud connectors.

I. Introduction

The composite beams made of steel profiles and concrete slabs have been widely used as solutions for buildings and bridges in the last decades because of the high ratio between strength/stiffness and weight and the fast and easy construction procedure. However, the full use of the mechanical properties of the composite elements is based on the correct design of the connection devices that have the key role of transferring the shear stress between the steel and concrete parts [1]. Therefore, the current design codes provide the approach for dimensioning the most common and assessed type of connectors that are the headed studs. In case of steel-concrete composite bridges the connection extends on long beams, therefore it is important to assess a model to analyse the influence of the stud's shear-slip relationship on the response of the deck considering the importance of stiffness, strength and ductility.

The model can be particularly useful to check the design provisions of the current codes that give the limitations and details to attain a ductile behaviour of the connection [2]. In this paper the FE modelling of composite bridge decks is proposed introducing the effect of the studs through nonlinear links with a shear-slip relation-ship according to the provisions of codes and the technical literature.

II. Stud Connectors Behaviour

Shear connectors are commonly employed at the steel-concrete interface for reducing horizontal slippage and vertical separation between the concrete slab and steel beam and assure the shear transfer at the interface [3], [4]. The main reason for utilizing shear connectors is to transfer the longitudinal shear forces at the steel-concrete interface. In the current approach provided by the Eurocode 4, the numbers of connectors applied on the steel beam can be designed to attain full or partial interaction, according to the capacity of transferring the entire shear force necessary to activate the full or partial resistance of the composite section calculated with a perfect bond between the two materials. This approach is applicable only in case of connections that require a ductile failure mode due to the interaction mechanism of the connector and the surrounding concrete. In the European code the ductile connection can be attained using headed studs with specific characteristics welded by a collar to the steel beam. In fact, the failure mechanism is complex and can be due to the steel shank flexure, shear interaction or the crushing of the surrounding concrete. The effect of the welded collar also can be taken into account [5].

Many experimental results by using push-out tests evidenced the effect of various parameters as reinforcement mesh position, shear stud height, stud spacing and diameter, concrete and steel strength [4]- [8]. Also, numerical FE models have been carried out confirming the role of the various parameters. Many formulations have been proposed during the time to estimate the shear strength considering the main mixed mechanism of steel and concrete, in the two cases of steel or concrete failure, while the concrete pry out [9] is usually neglected because it occurs only for squat studs not commonly used in composite beams because codes provide a minimum height of 3 times the diameter.

Some of the formulation proposed by researchers and codes for evaluating the shear strength PR of headed studs are summarized in the following, where the symbols used are fc for the compression cylindrical strength of concrete in MPa, d and As the diameter and section of the stud shank, fu the ultimate tensile strength of stud in MPa, Ec and Es are the elastic modulus of concrete and steel respectively.

Viest [10] proposed a first simple relation based on the experimental results and the mechanical behaviour of the connection:

$$P_R = 27.6\, d^2\, (f_c)^{0.5} \tag{1}$$

Based on the work by Ollgaard et al. [11], the following formulation is currently used in the Canadian steel design code [12] and AASHTO [13]:

M.M. Mohammadi Dehnavi is with the Department of Engineering, University of Sannio, Benevento, 82100, Italy (corresponding author, e-mail: mah.moh7@gmail.com).

A. De Angelis is with the Department of Engineering, University of Sannio, Benevento, 82100, Italy (e-mail: adeangelis@unisannio.it).

M.R. Pecce is with the Department of Structures for Engineering and Architecture, University of Naples Federico II, Naples, 80215, Italy (e-mail: pecce@unina.it).

$$P_R = 0.5\phi_{sc} A_s (f_c E_c)^{0.5} \leq \phi_{sc} f_u A_{sc} \quad (2)$$

where ϕ_{sc} is a performance factor and is taken as 0.8.

Push tests by Hawkins [14] showed that the dowel strength increases when the tensile strength of the stud material f_u is increased. Oehlers and Johnson [15] therefore modified Ollgaard's work to allow for variations in f_u considering the regression of experimental results as reported in (3):

$$P_R = (5.3\text{-}1.3/n^{0.5}) A_s f_u (E_c/E_s)^{0.4} (f_c/f_u)^{0.35} \quad (3)$$

where n is number of shear connectors in a group. In AISC [17] the following formulation is reported:

$$P_R = 0.5A_s (E_s f_c) \leq A_s f_u \quad (4)$$

Eurocode 4 [2] introduces an approach with two formulations as in (5) for the failure of steel or concrete, always in case of a mixed mechanism, and the lower result is the design strength of the stud:

$$P_R = (0.29\alpha d)^2 (E_c f_c)^{0.5} / \gamma_v \quad \text{or} \quad P_R = 0.8 f_u (\pi d^2)/4)/\gamma_v \quad (5)$$

The shear-slip law was developed only in few studies; the most common shape was suggested by Ollgaard et al. [11], but a simpler representation can be also a trilinear one assuming a first point at elastic limit ($P_{max}/2$, S_e), a second one at (P_{max}, S_m), and the ultimate condition at (95% of P_{max}, S_u).

The slip in some main points can be evaluated by the equations suggested by Johnson and Molenstra [18] (in which considered the concrete with normal-density with $20 < f_c < 70$ MPa), these equations are then reviewed by Oehlers and Bradford [5] and other authors [19]:

$$S_e = (80 \cdot 10^{-3} - 86 \cdot 10^{-5} f_c)d; \quad S_m = (0.39 - 0.0023 f_c)d;$$
$$S_u = (0.48 - 0.0042 f_c)d \quad (6)$$

III. Modelling Approach

The FE modelling can be approached with various levels of details that depend on the aim of the analysis and complexity of the problems. For composite beams, a model of the entire structure that introduces the connection between steel and concrete parts by nonlinear links with adequate relationship between displacements (slip and lift) and forces (normal and shear ones) [6], [15], [16], is a good compromise between a result about the role of the connection on the global behaviour of the structure and an acceptable computational effort. Therefore, in this work a global model was adopted since the aim is to better understand the effect of the main parameters of the connection on the deformability and resistance of the composite beams, and a simple supported steel-concrete composite deck of a bridge 36 m long was considered. The structure consisted of a concrete solid slab and 3 I-shape beams which were connected by headed studs. The FE model was implemented by using four sides shell elements with six-degree of freedom nodes for the steel profiles and the concrete slab, while nonlinear links were introduced to connect the two parts considering the load-slip properties of the connector.

A. The Geometrical Model

The 3D nonlinear model was developed using the software SAP2000 [20]. The cross section of the deck (Fig. 1) was chosen according to typical existing composite bridge in Italy; it is made by 3 double T steel profiles (with h = 1300 mm, top flange width and thickness b_{tf} =350 and t_{tf} =25mm respectively, bottom flange b_{bf} =650 mm and t_{bf} = 55mm respectively, web thickness t_w = 12 mm) spacing of 2950 mm with a concrete slab of 300 mm thickness and 6250 mm width.

Four transverse elements are located at the ends and 1/3 and 2/3 of the length of the bridge. The mesh dimensions of concrete shell elements are 295×220 mm^2.

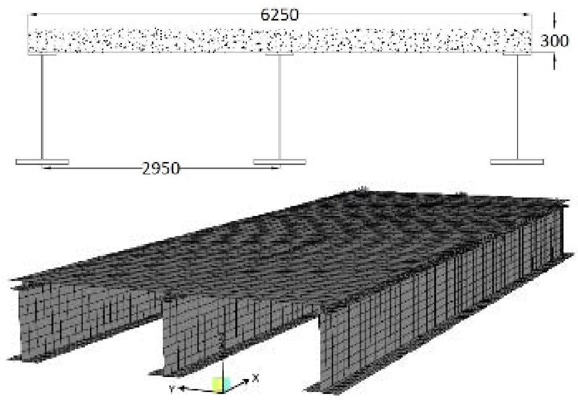

Fig. 1 The section and a segment of the 3D model (measures in mm)

B. Materials and Connection

The material properties were defined according to the typical ones used for composite bridges in Italy according to experimental campaigns on existing structures. In particular for the case study, a mean cylinder strength of concrete in compression of fc = 30MPa and yielding strength of 355MPa for the construction steel was estimated.

According to [3], the behaviour of concrete (Fig. 2) was assumed linear perfectly plastic in compression, as suggested also by codes, with an ultimate strain of 0.0035. It is important to underline that in nonlinear analysis of RC structures the behaviour of concrete in tension is very important but in this type of model of composite beams, the concrete slab is in compression and tension stresses are due only to bi-dimensional effect in the global analysis.

A bilinear hardening stress-strain behaviour was defined for steel (Fig. 2) with the yielding and ultimate strength of 350 and 540 MPa, respectively.

For each one shear stud connector, a multilinear plastic link was introduced to joint shells of the steel flange and concrete slab. A simplified bi-linear load-slip relationship was used for the links instead of the tri-linear one introduced in Section III *B*, assuming a linear first branch up to (PR, S1), being S1 = 2·Se, a plateau to Su, however a descending branch is added up to 20 mm. For understanding the effect of the connector's properties on the global behaviour of the bridge, the main parameters of the shear-slip law were varied. A reference law

(RS), according to code and literature provision was established with a slip at the elastic limit of 2 mm, a plateau and a descending branch up to 6 mm and 20 mm, respectively. Then strength (2 times, OS case) and ductility (elastic perfectly plastic, EP case) were changed. The shear-slip laws of the various cases are reported in Fig. 3, in case of double strength the double stiffness of the reference case is assumed.

The headed studs with a diameter d = 20mm and a height higher than 4 times diameter were chosen and their spacing was designed assuming a ductile behaviour of the connection and a full interaction.

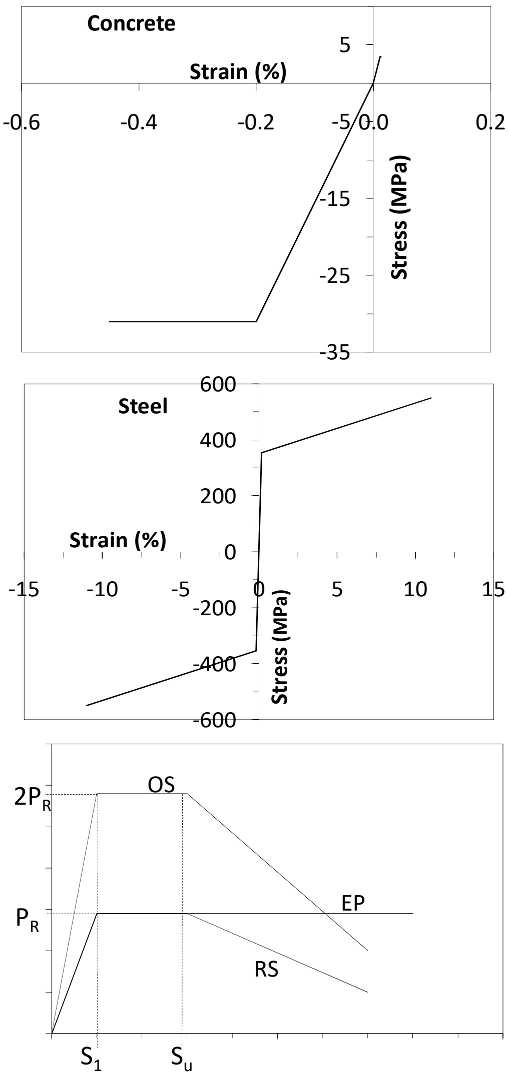

Fig. 2 Stress strain relationship of concrete and steel, shear slip law of studs (reference stud (RS), elastic-plastic connection (EP) and over-strength stud (OS))

The strength of the studs was calculated by the formulations given in Eurocode 4 [2], but the characteristics values of the materials strength are assumed equal to the mean values and the partial safety factor $\gamma_v = 1$, because the actual response of the beam has to be analysed without taking into account the safe effect of the design method. The strength was $P_R = 111$ kN and resulted from the concrete failure formulation.

To obtain the spacing of the studs, the horizontal shear force $V_{l,Rd}$ that has to be transferred at the steel-concrete interface is calculated as the minimum value of the maximum compression and tension internal forces that give the axial equilibrium of the composite section as reported in (7):

$$V_{l,R} = min\ \{b\cdot d\cdot f_c = 57150kN;\ A_s\cdot f_y = 64006kN\} = 57150\ KN \quad (7)$$

Therefore, the full shear interaction requires the minimum number of stud N_f for the 3 beams equal to 499 that correspond to coupled studs with a spacing of 220 mm along the span.

IV. The Numerical Analysis

The nonlinear analysis of the case study was carried out considering a uniform load applied on the entire slab because the main aim is to compare the effect of different properties of the connectors on the global behaviour having some information about the distribution of the interface shear and the ductility demand in the studs. Surely the shape of the live load, as for example the concentrated axes of trucks on the bridge, can give different results but the role of the studs remains the same.

A. The Global Behaviour of the Deck

The analyses were developed for the three cases of studs behaviour but considering the same numbers of connectors, N_f, designed for the full interaction of the reference case. Furthermore, a case of partial interaction with the half number of reference case ($N/N_f = 0.5$) was added. According to the modelling of a simple supported deck under a uniform load, the maximum moment and deflection occurred at the mid-span of the bridge. The load-deflection relationship was elaborated considering the load non-dimensional respect to strength capacity of the deck that is the load q_p corresponding to the plastic moment in the middle span ($q_p = 8Mp/l2$). The non-dimensional load-deflection curves are depicted in Fig. 3; the value 1 on the vertical axis corresponds to the load q_p, the horizontal and vertical dashed lines correspond to the yielding moment (evaluated by the analytical behaviour of the composite section) and the corresponding deflection given by the model, the vertical dotted line A indicates the limit elastic behaviour of the case with partial interaction ($N/N_f = 0.5$).

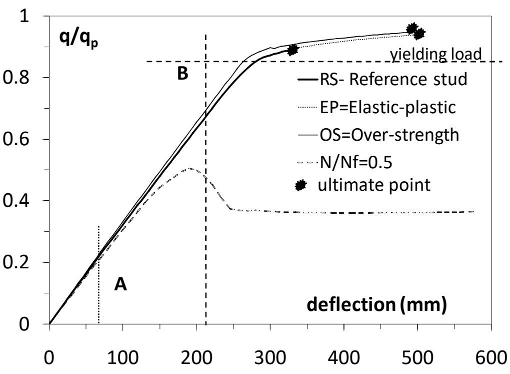

Fig. 3 Load-deflection curve of models

The load q_p is related to the plastic moment evaluated by stress blocks while the elastic-plastic behaviour is assumed for concrete, therefore it could be only asymptotically reached by the models. The results show that the case RS attains the 90% of q_P, i.e., the failure of the connection occurs before the complete plastic behaviour of the most stressed section of the deck reducing the global ductility of the structure; conversely a higher global capacity in strength and ductility is realized with EP or OS studs, which means a better performance with greater ductility or strength. The case OS shows also an increment of stiffness due to the higher stiffness of the studs.

The case designed with $N/N_f = 0.5$ confirms that $0.5 \cdot q_P$ was really arisen but the global failure was brittle. It means that the design of ductile connection in partial interaction gives a brittle global failure of the structure.

The results underline that the ductile stud defined by Eurocode 4 [2] as a slip capacity of 6 mm could not be safe. Surely the effect of the partial safety factors of materials and connection strength can give a final safe design but really these factors would be covering the uncertainties of the materials and connectors behaviour, but not necessarily the ones of the global behaviour of the beam.

B. The Local Response of the Connection

The model allowed to check the distribution of the shear and slips in the studs along the deck. In Fig. 4 the slip and the shear (non-dimensional respect the reference stud strength PR) in the studs along the central beam is reported for the various types of connectors at the main points indicated in Fig. 3 that are: point A and the ultimate condition (maximum load) for all 4 cases, point B at the yielding moment clearly excluding the N/Nf = 0.5.

The cases of RS and EP give the same results in the elastic field (point A) and up to yielding point because the maximum slip is lower than 6 mm, while a little difference is clear at the ultimate condition because the slip overcomes 6 mm, and the EP connection allows to distribute the maximum shear (the strength of the stud) along the deck while the RF shows the reduction of the shear at the beam end. The case of OS gives very different results because the higher strength allows a relevant reduction of the slip that remains lower than 6 mm at the ultimate condition with a quite linear distribution of slip and shear.

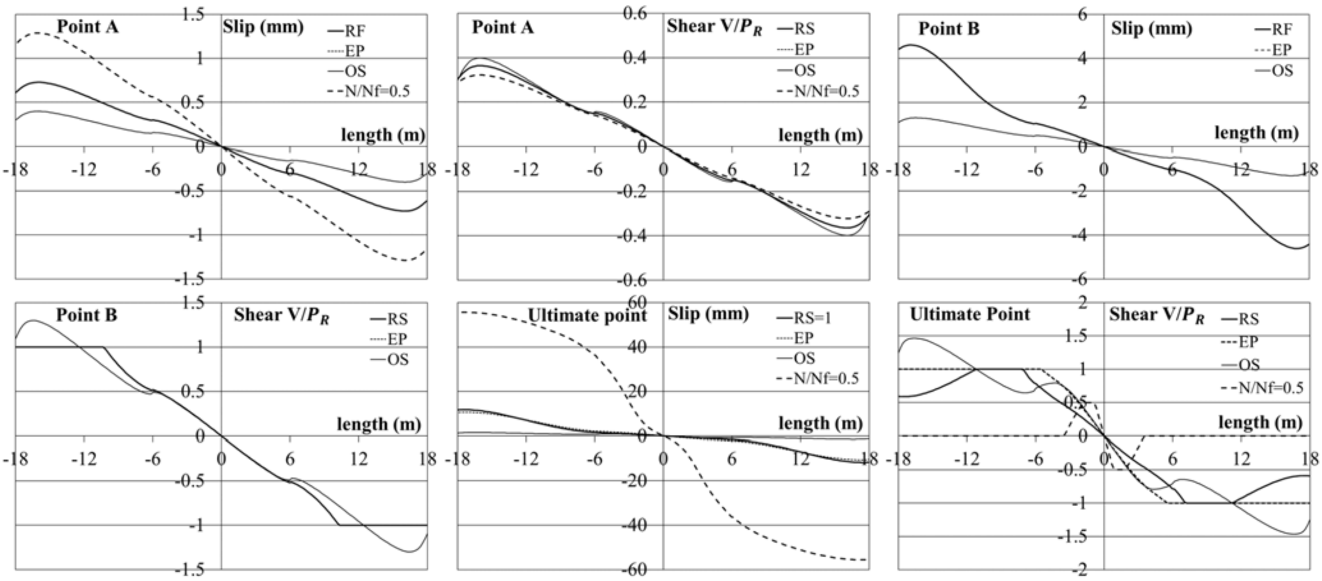

Fig. 4 Slip and shear distribution in the studs along the deck

V. Conclusions

In this work a nonlinear FE model has been implemented to investigate the effect of ductility and strength of headed studs on the behaviour of a composite steel-concrete deck. The models took into account the nonlinear material properties of concrete, steel beams and shear connectors.

The model was efficient to study the effect of the shear-slip law of the connection on the response of the structure in terms of global load-deflection of the deck and slip/shear distribution along the steel beams interface. The results of the case study showed some interesting features of the problems, especially that the strength and ductility of the deck is limited using the ductile studs as defined by Eurocode 4 with a slip capacity of 6 mm, but this effect can be compensated also using studs with over strength respect to the design. The design with a partial interaction reduces not only the strength of deck but also its ductility. However further analyses are in working to better assess the role of the studs' ductility, that is a key issue in the reviewing of Eurocode 4 currently in progress, considering the variation of the length of the beams, the studs on the transversal elements and the skew of the slab.

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