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Finite Detail Modeling Of Metallic Concrete Beam Thinking About Double Composite Movement

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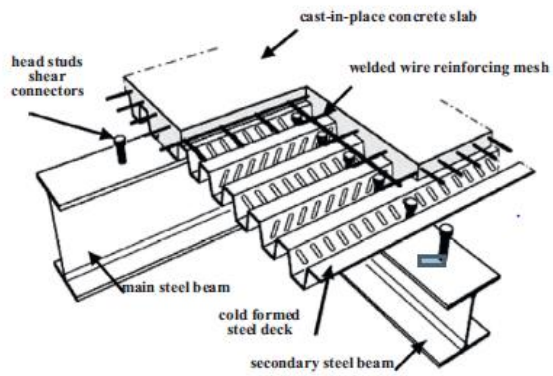
Abstract :

Steel concrete composite production has gained huge popularity as an opportunity to natural metallic or concrete creation. Ansys 11 laptop program has been used to increase a 3-dimensional nonlinear finite detail model in order to analyze the fracture behaviors of non-stop double steel-concrete composite beams, with emphasis on the beam slab interface. 3 beam fashions with various quantity of the pinnacle studs have been addressed. The related constitutive outcomes consisting of the remaining hundreds, the most deflections, the interface slip and slip strain values are supplied. A parametric look at has been completed so as to research the effect of some parameters on their fracture abilities, such as metallic beam top, lower slab thickness and period, studs diameter and arrangement technique. Through comparing these consequences with the to be had experimental records, the proposed version is discovered to be capable of reading steel-concrete composite beams to a suitable accuracy.

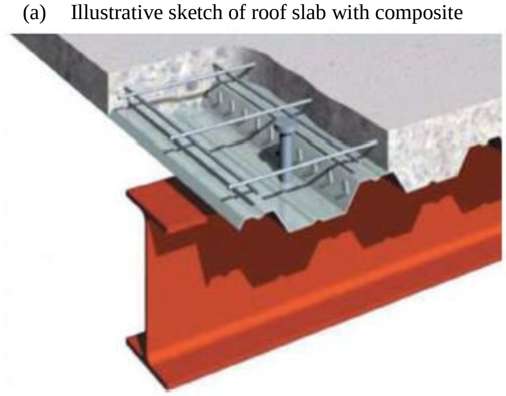
1. Introduction

The use of composite structures is increasingly present in civil construction works. Steel-concrete composite beams, particularly, are structures consisting of two materials, a steel section located mainly in the tension region and a concrete section, located in the compression cross-sectional area, connected by metal devices known as shear connectors. One type of these connectors is called head studs as shown in Fig. 1. The main functions of these studs are to allow for the joint behavior of the beam-slab, to restrict longitudinal slipping and uplifting at the elements interface and to take shear forces. Double steel-concrete composite continuous beam is a new structural system developed on the basis of single steelconcrete one, in which there is also a bottom reinforced concrete slab connected to a steel profile in the negative moment regions through the head studs, therefore with two interfaces.

Comparing with the traditional single steel-concrete composite continuous beam, its advantage is that effectively limits the crack width of the negative moment area, and also improves the stress state of section, so that it is suitable to the composite continuous beam with a larger span. The mechanical properties of the double composite beam obviously depend on their respective properties and interactions. In the negative applied.



(a) Illustrative sketch of roof slab with composite



(b) Composite beam system with head studs shear connectors.

Figure 1 Steel-concrete composite section with studs shear connectors.

bending moment area, the concrete slab cracks under tension and then the interface slip occurs between steel profile and concrete slab, with non-linear features, it makes great impact on the structure of the internal forces and deformation. Therefore, it is necessary to present a finite element model to study the mechanical properties of the double steel-concrete composite beam in negative moment regions. Although many experimental and theoretical studies for the traditional single steel-concrete composite beam have been done, few research studies have been found in references to the double steel-concrete composite continuous beam. Rozsas [1] investigated the plastic reserve of composite plate girder bridges due to the synergetic combination of the concrete and steel. The plastic design in the framework of the

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Eurocode through an existing elastically designed bridge is also introduced. Xu et al. [2] discussed the improvement of the local buckling strength of continuous double composite box girders by adding a concrete slab to the steel bottom flange. The mechanical properties in concrete crack, formation of sectional plastic hinge are also investigated. Tan et al. [3] utilized experimental tests to provide further information and conclusions regarding composite steel-concrete beam specimens by examining the behavior of multi-span composite steel-concrete beams. These beams are subjected to combined actions of torsion and flexure for both full and partial shear connection and comparing the disparity in the varying degrees of shear connection. Lin and Yoda [4] studied the mechanical performance of the horizontally curved continuous composite steel-concrete beams subjected to combined hogging (negative) bending and torsion, in order to investigate the effect of curvature on both elastic and inelastic behaviors of these beams in the interior support regions. Henriques et al. [5] presented a generalized beam theory (GBT) formulations specially designed for performing efficient linear analysis of steel-concrete composite bridges and elastoplastic collapse analysis of thin-walled steel members and extended for including the non-linear reinforced concrete material behavior of steel-concrete composite beams. Liang et al. [6] have undertaken nonlinear finite element analysis on continuous composite beams in combined bending and shear. In their study, design formulas incorporating contributions from the concrete slab and composite action were proposed for vertical shear strength and the ultimate shear interaction of continuous composite beams. A finite element model is presented by Liang et al. [7] to investigate the flexural and shear strengths of simply supported composite beams under combined bending and shear. In this research, the numerical results are verified and compared with the available experimental results. Sebastian and McConnel [8] described a nonlinear finite element program for modeling composite beams. Axial springs with empirical shear slip relations were used to model discrete shear connectors. Hirst and Yeo [9] used a standard finite element program to analyze composite beams with partial and full shear connection. Quadrilateral elements were employed to simulate discrete and stud shear connectors. The material properties of stud elements were modified to make them equivalent in strength and stiffness to the actual shear connectors in composite beams. Al- Amery and Roberts [10] presented a nonlinear analysis of composite beams with partial shear connection by using a finite difference method. Salari et al. [11] formulated a composite beam element based on the force analysis method for the nonlinear analysis of composite beams with deformable shear connectors. Thevendran et al. [12] utilized the finite element software ABAQUS to study the ultimate load behavior of composite beams curved in plan. Shell elements were used to model the concrete slab and the steel beam while a rigid beam element was employed to simulate stud shear connectors. Reiner [13] and Stroh and Sen [14] presented a double steelconcrete composite continuous beam as a new structural system developed on the basis of single steel-concrete composite beam, in which there is also a bottom reinforced concrete slab connected to a steel profile in the negative moment regions through the shear connectors, therefore with two interfaces. This research was accompanied by the determination of the crack width limits of the negative moment area, and the improvement of the stress state of section, and later applied for the composite continuous beam with a larger span. Newmark et al. [15] introduced the partial collaboration theory which is used later for deriving the elastic stiffness matrix in the negative moment region for a double composite beam element and for studying and verifying the double composite continuous beam models, and consequently the composite action effect as illustrated by Duan et al. [16–18]. Nagai et al. [19] tested a double composite girder under pure hogging moment and measured its ultimate bending moment strength. Duan et al. [20] and Yang and Duan [21] focused on the problems of interface slip, deformation, ultimate bearing capacity, and the effective flange width of concrete slab for the double steel-concrete composite beams. Wang et al. [22] presented the elastic analysis of double composite beam deformations using the Goodman elastic sandwich method. Yen et al. [23] discussed the ultimate load behavior and elastic deformations of steel box girders containing composite bottom flanges. Duan et al. [24] performed beam collapse tests for three models of double steel-concrete composite continuous beam. These tests aimed to report the load–deflection curve, the ultimate flexural capacity, and the interface slip and slip strain values between steel and concrete along the span direction. The objective of the current paper was to demonstrate a proposed analytical finite element model of continuous double steel-concrete composite beams to estimate the fracture behavior and interface slip values of tested specimens produced by Duan et al. [24], through Ansys 11. The analytical model and the results of system level study can be of interest in assessing progressive collapse resistance of existing structures contain double steel-concrete composite beams and in the design of new structures.

2. Research significance

The target of this research is to demonstrate a better analytical understanding of double steel-concrete composite beams. Thereby, the focus should be set on the analysis of the maximum increase in strength and deflection capacity due to the existing of double composite action. Therefore, the principal purpose is the nonlinear finite element analysis of continuous steel-concrete composite beams containing double composite action and head studs shear connectors. Within this framework, several aspects should be investigated such as the load–deflection response of the composite beam, and the gradual evolution of slip and slip-strain values at the beam-slap interface up to failure considering double composite action. Based on this investigation, a simplified analytical model through Ansys 11 software is developed in order to enables the prediction of the fracture behavior. Its results are compared with the previously available experimental investigated models introduced by Duan et al. [24]. The results demonstrate a better approximation for the failure criteria in both cases.

3. Methodology and the analytical model

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The objective of this section is to describe the finite element model features common to double steel-concrete composite beams being considered. The Ansys 11 finite element package was used to carry out the modeling. The applied load was iterated step by step using the Newton-Raphson method. Solid65 element was used to model the concrete. This element has eight nodes with three degrees of freedom at each node translations in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. A schematic of the element was shown in Fig. 2a. A Link8 element was used tomodel steel reinforcement. This element is a 3D spar element and it has two nodes with three degrees of freedom translations in the nodal x, y, and z directions. This element is capable of plastic deformation and element was shown in Fig. 2b. The modeling of the head studs shear connectors was done by the BEAM 188 elements, which allow for the configuration of the cross section, enable consideration of the nonlinearity of the material and include bending stresses. This element was indicated in Fig. 3a. SOLID185 is used for the modeling of the steel beam. It is defined by eight nodes having three degrees of freedom at each node, translations in the nodal x, y, and z directions. The element has plasticity, hyperelasticity, stress stiffening, creep, large deflection, and large strain

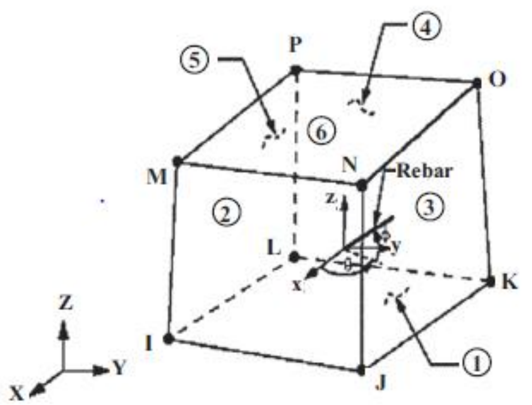

Figure 2a Solid65 – 3D solids modeling.

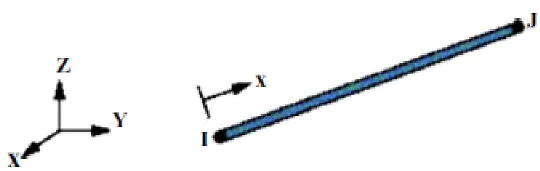

Figure 2b Link8 –3D spar modeling

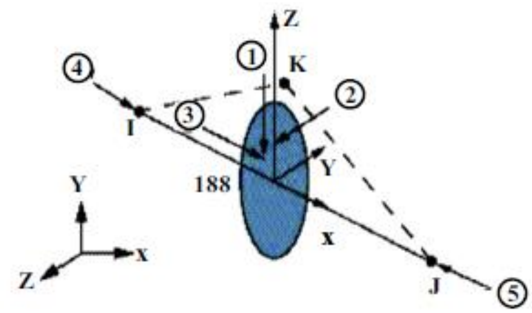

Figure 3a Beam188 – 3D quadratic beam modeling

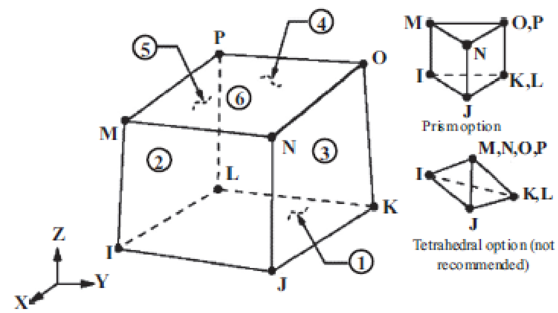

Figure 3b Solid 185 – 3D solids modeling

capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyperelastic materials as shown in Fig. 3b. TARGE170 and CONTA173 elements were used to represent the contact slab-steel beam interface. These elements are able to simulate the existence of pressure between them when there is contact, and separation between them when there is not. The two material contacts also take into account friction and cohesion between the parties. The study shear connector was considered as a clamped metal pin in the steel section, with rotations and translations made compatible. On the slab connector interface, translational referring to the Y and Z axes was also made compatible and, at the Node below the pin head, there was a consideration of coupling in the X direction to represent the mechanical anchoring between the head of the connector and the concrete slab. The geometry of these elements is as shown in Fig. 4a–c. An eight-node solid element, Solid 45, was used to model the steel plates under the load. The element is defined with eight nodes having three degrees of freedom at each node in the nodal x, y, and z directions. The geometry and node locations for this element type are as shown in Fig. 5. Three double steel-concrete composite beam models with the same material properties and cross section shape were analyzed. The only difference between them is that the arrangement of the head studs. Two lines with different number of head studs for the top and the bottom slabs were proposed as eported in Table 1. The geometry of the proposed model components is as shown in Fig. 6a–g. In order to saving Ansys 11 – computational time significantly, a quarter of full composite

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beams have been modeled as shown in Fig. 7a and b. All the investigated models are constrained at edge ab in the directions y and z, while edge cd is constrained in the directions x and z. In addition, other directions were free of constraints as indicated in Fig. 7a and b. Thus, the research concerns solely symmetrically continuous double steel-concrete composite beams. The cross sections for all the models, namely SCB1, SCB2, and SCB3, are constructed by a top concrete slab along the whole beam length with tension reinforcement 7U8/m' in each direction, and by a 1000 mm length bottom concrete slab over interior support, whereas the upper and lower slab thickness was 80 mm.

4. Material properties of the proposed model

Table 2 summarizes the values of the material properties for all composite beam model components, i.e., reinforced concrete slab, steel beam, and head studs. For the steel beam and head studs, the maximum tensile strength obtained from the experimental test as ft= 235 MPa and the young modulus of elasticity as 2.06 · 105 Mpa. As mentioned above, Solid65 element is used to simulate the concrete. According to Fanning [25], this element requires linear and multilinear isotropic material properties to properly model concrete. For the linear isotropic material, the concrete cube compressive strength obtained from the experimental test as 47 MPa, and the young modulus of elasticity as 4.62 · 104 Mpa. The multilinear isotropic material uses the Von Mises failure criterion along with the William and Warnke [26] model to define the failure of the concrete. A three-dimensional failure surface for concrete is shown in Fig. 8. The most significant non-zero principal stresses are in the x and y directions respectively. Three failure surfaces are shown as the projections on the rxp - ryp plane. The mode of failure is the function of the sign of rzp (principal stress in Z direction). For example, if rxp and ryp, both are negative (compressive) and rzp is slightly positive (tensile), cracking would be predicted in a direction perpendicular to rzp. However, if rzp is zero or slightly negative, the material is considered as crushed. Implementation of the William and Warnke [26] material model in Ansys 11 requires different constants that must be defined. Shear behavior of SOLID65 element in Ansys 11 is controlled by two-shear transfer coefficient for open and closed cracks. These coefficients represent conditions at the crack allowing for the possibility of shear sliding across the crack face. A number of preliminary analysis were attempted in this study with various values for the shear transfer coefficients (for open and closed cracks) within the below indicated ranges, but Ansys convergence problems were encountered at the following entering values of the William and Warnke [26] constants:

1. Shear transfer coefficient for open crack was entered as 0.5. Its recommended range is from 0.2 to 0.5 as presented by Razaghi et al. [27].
2. Shear transfer coefficient for closed crack was entered as 1. Its recommended range is from 0.0 (for representing a smooth crack, i.e., complete loss of shear transfer), to 1 (for representing a rough crack, i.e., no loss of shear transfer), as suggested by Razaghi et al. [27].
3. Uniaxial tensile cracking stress which was based upon the modulus of rupture; and was entered as 4.70 Mpa.
4. Uniaxial crushing stress was based on the uniaxial unconfined compressive strength, and was entered as 47.0 Mpa, to turn on the crushing capability of the concrete element as discussed by Kachlakev and Miller [28].
5. Biaxial crushing stress.

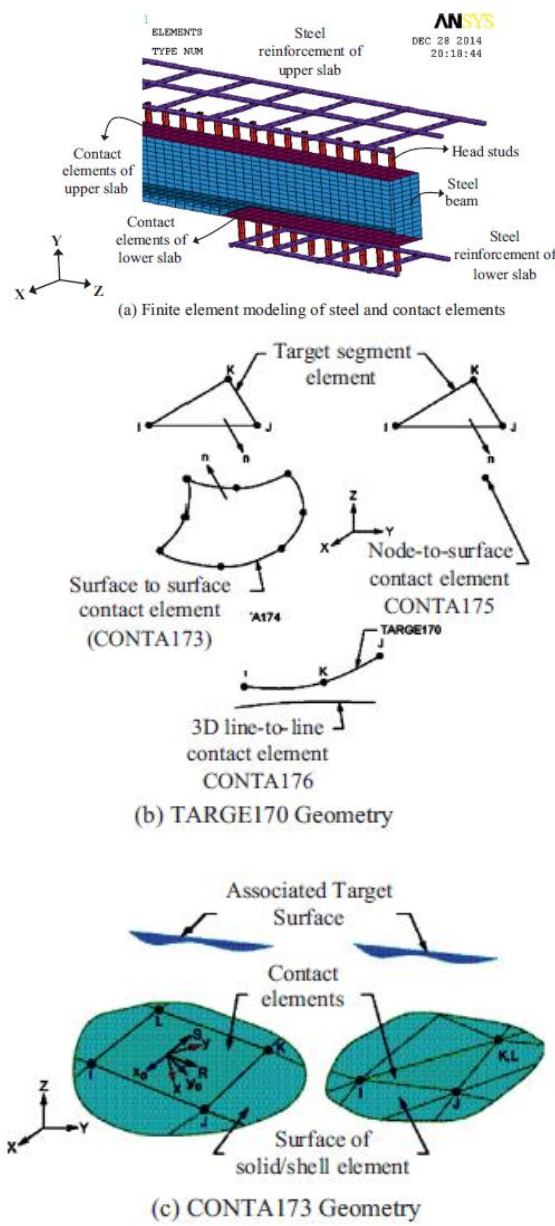

Figure 4 Geometry of TARGE170 and CONTA173 elements.

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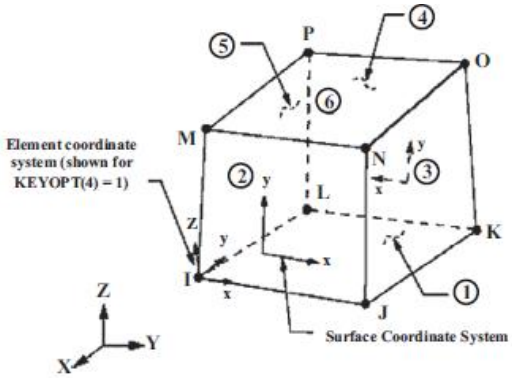
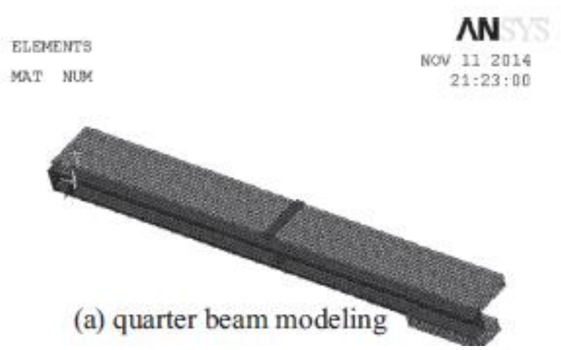


Figure 5 Solid45 – 3D solids modeling.

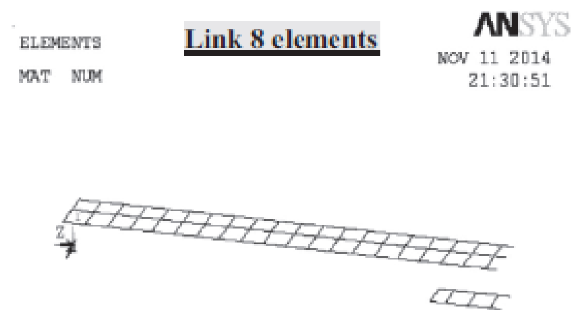
Table 1 Studs arrangement for the upper and lower slabs.

Model	Number of studs in each line	
	Upper slab (N_1)	Lower slab (N_2)
SCB1	94	28
SCB2	82	28
SCB3	82	24

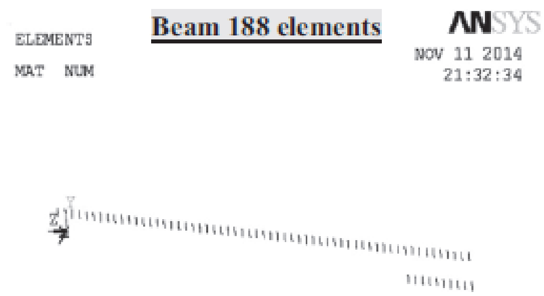


(a) quarter beam modeling

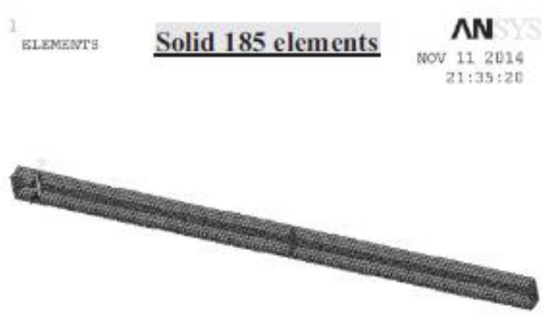
(b) Slab modeling



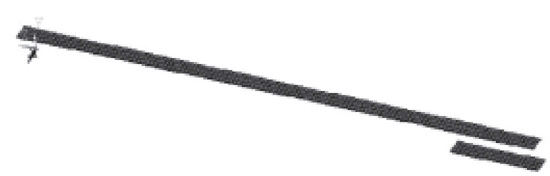
(c) Reinforced steel modeling



(d) Studs modeling



(e) Steel beam modeling



(f) Contact elements modeling

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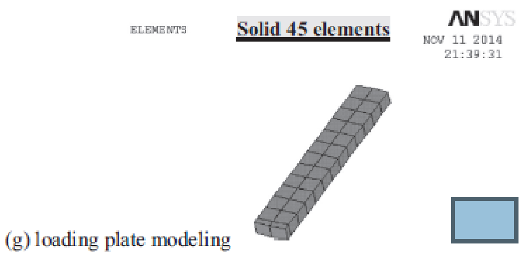


Figure 6 Geometry components of the all beam models.

6. Ambient hydrostatic stress state for use with constant 7 and 8.
7. Biaxial crushing stress under the ambient hydrostatic stress state (constant 6).
8. Uniaxial crushing stress under the ambient hydrostatic stress state (constant 6).
9. Stiffness multiplier for cracked tensile condition.

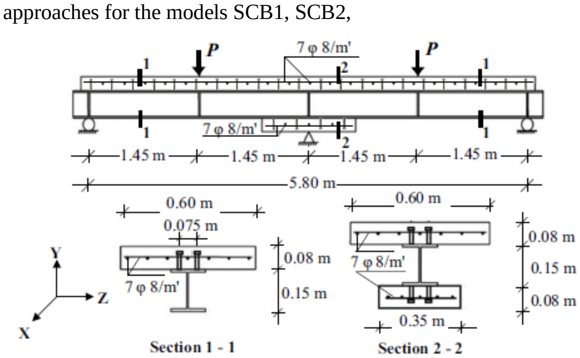
Coefficients from 5 to 9 were implemented as zero value, as discussed by Wolanski and B. [29], in order to encounter the Ansys convergence problem.

5. Validation of the analytical model

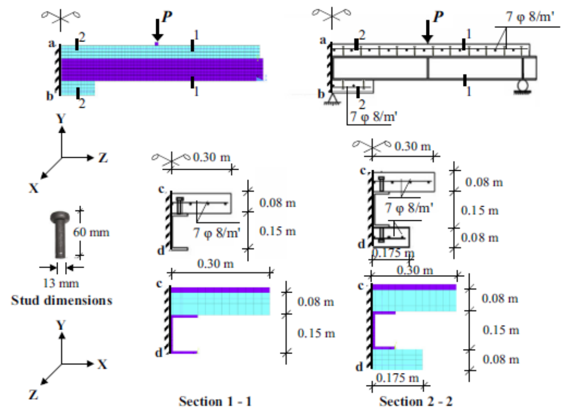
The comparison of the results from the analytical model to the experimentally obtained results enables the validation of the performance of the proposed model. The comparison consists of the tests performed by Duan et al. [24] and the results obtained by the proposed finite element model. The proposed model delivered valuable outputs concerning the behavior of the continuous double steel-concrete composite beams such as the strength capacity, the maximum deflection, the interface slip and slip strain of the upper and lower slab of the double composite beam models.

5.1. load–deflection relationship

The load–deflection curves analyze the different performance of the double steel-concrete composite model with respect to the strength and deflection capacities. Figs. 9–11 illustrate the load–deflection curves obtained by both the proposed and experimental approaches for the models SCB1, SCB2,



(a) Geometry of proposed full-scale model



(b) Geometry of proposed quarter model

Figure 7 Geometry and cross sections dimensions of all beam models

Table 2 Material properties of the proposed model.

(1) Concrete	
Concrete strength (f_c)	47 Mpa
Young modulus of elasticity (E_c)	4.62×10^4 Mpa
Poison's ratio (γ)	0.3
(2) Steel	
Maximum tensile strength (f_t)	235 Mpa
Young modulus of elasticity (E_t)	2.06×10^5 Mpa
Poison's ratio (γ)	0.2
(3) Studs	
Maximum tensile strength (f_t)	235 Mpa
Young modulus of elasticity (E_t)	2.06×10^5 Mpa
Diameter (Φ_{stud})	13 mm
Height (h_{stud})	60 mm

and SCB3 respectively. An increase in the proposed strength capacity values of approximately 32%, 27%, and 29% compared to the experimentally obtained one is observed. Tables 3 and 4 show the significant comparison of the maximum load capacity and the maximum deflection values for the three proposed models. Good agreement is noticed between the values of the two approaches. Also, it has to be noticed that the developed models exhibited a softer performance than that of the experimental results. This is due to the following reasons:

1. The William-Warnke failure criteria in Ansys cannot suitably predict the behavior of reinforced concrete structures, as it does not consider the material softening properly due to the varying range of its constants values, such as the shear transfer coefficient for open and closed crack. In addition, for this kind of failure criteria, the crushed elements are removed from the model and that could lead to premature failure, which is not consistent with the real behavior of reinforced concrete structures.
2. Due to the possibility of the inaccuracy in modeling the postyield behavior of steel rebar material, there is somewhat none agreeable between the finite element results and those of experimental results for postyield behavior.

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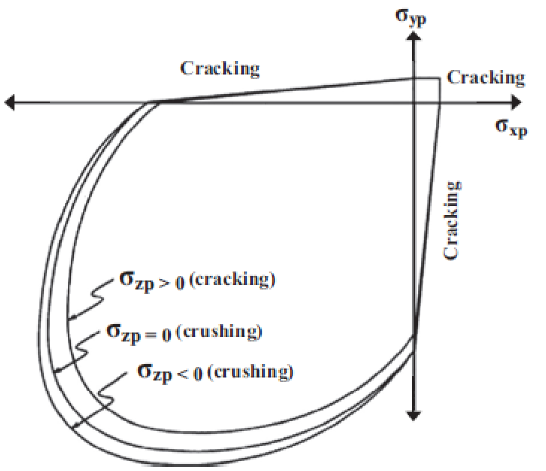


Figure 8 Failure surface for concrete, William and Warnke material model [26].

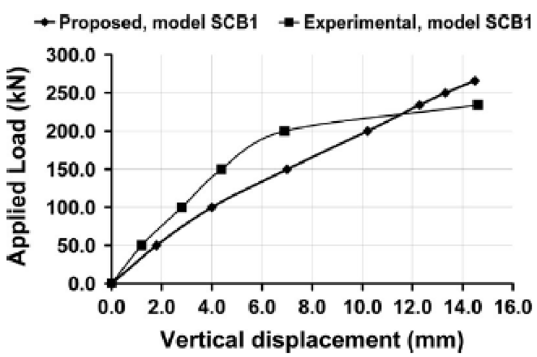


Figure 9 Load verses deflection curve for beam model SCB1.

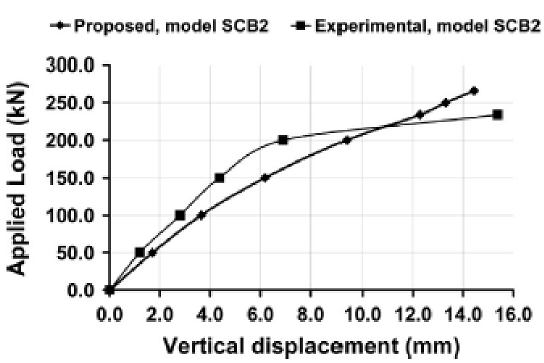


Figure 10 Load verses deflection curve for beam model SCB2.

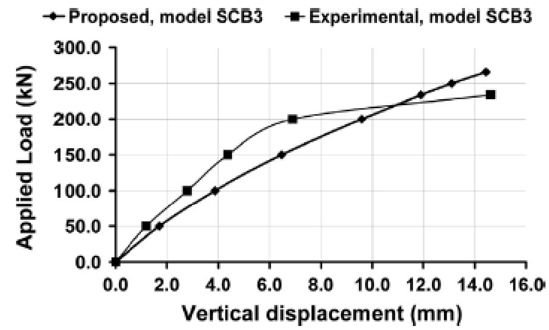


Figure 11 Load verses deflection curve for beam model SCB3.

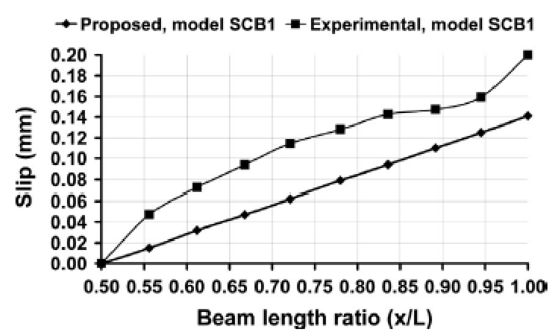
Table 3 Comparison of the load capacity results at collapse.

Beam model	Load capacity, $P_{ult.}$ (kN)		% Difference
	Proposed	Experimental	
SCB1	266.00	234.00	13.67
SCB2	265.50	233.00	13.73
SCB3	264.30	232.00	13.79

Table 4 Comparison of the maximum deflection results at collapse.

Beam model	Maximum deflection, $\Delta_{max.}$ (mm)		% Difference
	Proposed	Experimental	
SCB1	14.49	14.61	0.80
SCB2	14.54	15.37	5.40
SCB3	14.43	14.61	1.20

Figure 12a Interface slip values



of the upper slab for beam model SCB1

As a result of these two statements, there is disparity between the proposed model results and those of Duan et al. [24] for the pre- and postyield behavior.

5.2. Interface slip values along the beam length

The slip-beam length curves analyze the different performance of the double steel-concrete composite model with respect to the slip values at collapse along the composite beam model length. Figs. 12a–14b illustrate the slip-beam length curves obtained by both the proposed and experimental approaches for the models SCB1,

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SCB2, and SCB3 respectively. A reduction in the proposed slip values of approximately 37%, 31%, and 47% compared to the experimentally obtained one is observed for the upper slabs. In contrast, an increase of approximately 21%, 30%, and 28% for the lower slabs is noticed. Good agreement is noticed between the values of the two approaches for the cases of the upper and lower slabs.

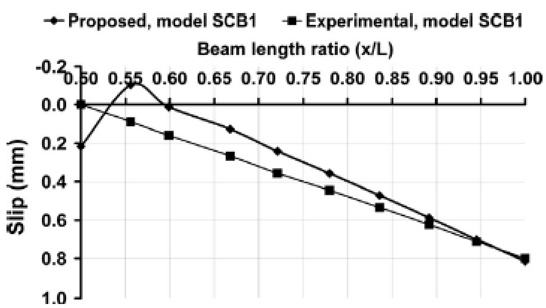

Figure 12b Interface slip values of the lower slab for beam model SCB1.

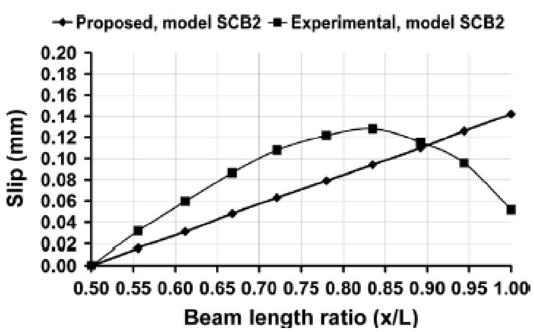

Figure 13a Interface slip values of the upper slab for beam model SCB2.

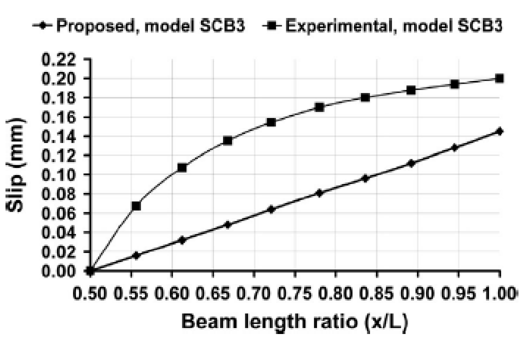

Figure 14a Interface slip values of the upper slab for beam model SCB3.

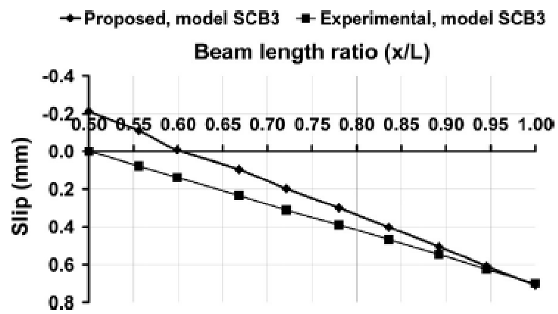

Figure 14b Interface slip values of the lower slab for beam model SCB3.

values of approximately 34%, 52%, and 63% compared to the experimentally obtained one is observed for the upper slabs. In addition, an increase of approximately 35%, 74%, and 62% for the lower slabs is noticed. Somewhat notable non-agreeing values are observed between the values of the two approaches for the cases of the upper and lower slabs. Fig. 19 shows the steps of the interface slip strain calculation for the upper slab of proposed model SCB1 as an example of the others.

6. Parametric studies

To further improve the understanding of the strength capacity and the fracture behavior of the continuous double steel-concrete composite beams having head studs shear connectors, parametric studies were performed to investigate the impact of the presence or absence of lower slab at the interior support, and the variation of the steel beam height. In addition, the variation of the lower slab length and thickness, and the variation of the studs arrangement and diameter are also studied.

6.1. The influence of removing the lower slab

The case study under consideration involves the influence of removing the lower slab on the mechanical and geometrical characteristics of the beam models at failure, such as the strength and the deflection capacity values. The study was conducted on three proposed models SB1, SCB2, and SCB3 respectively. Fig. 20 illustrates the effect of varying composite action on the fracture characteristics (strength and maximum deflection) of the proposed model. Figure 12b Interface slip values of the lower slab for beam model SCB1. Figure 13a Interface slip values of the upper slab for beam

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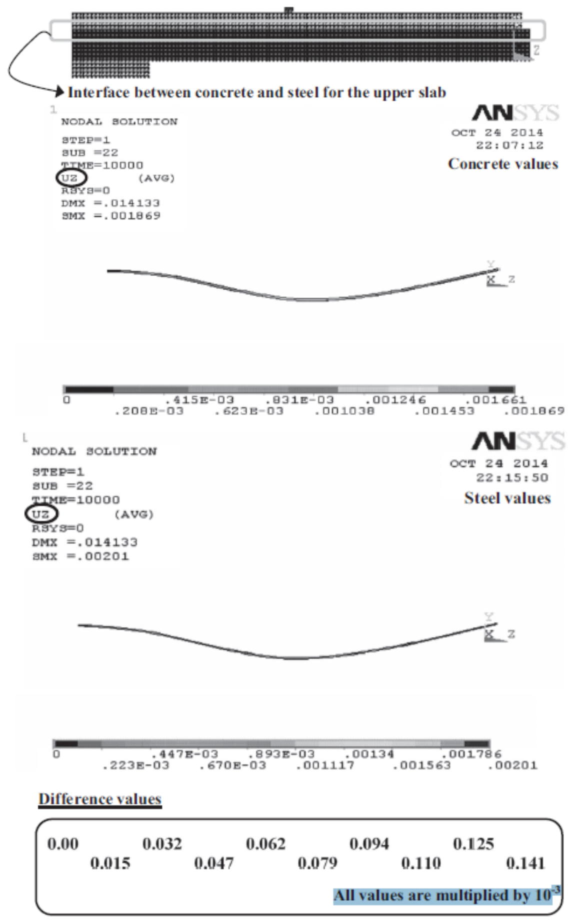

Figure 15 Difference between the interface longitudinal displacements of concrete and steel along the beam length direction for the upper slab of the beam model SCB1 (slip values).

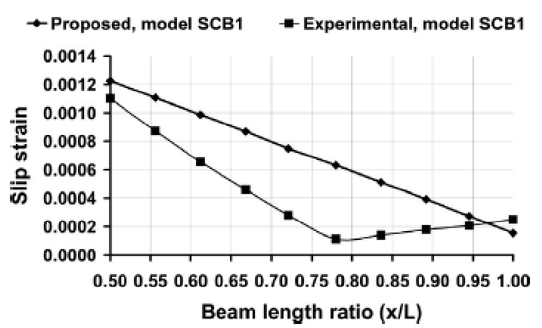

Figure 16a Interface slip strain values of the upper slab for beam model SCB1.

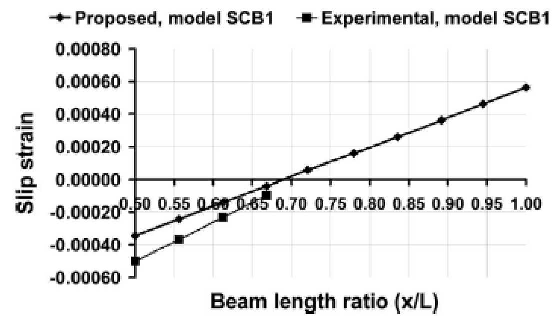

Figure 16b Interface slip strain values of the lower slab for beam model SCB1.

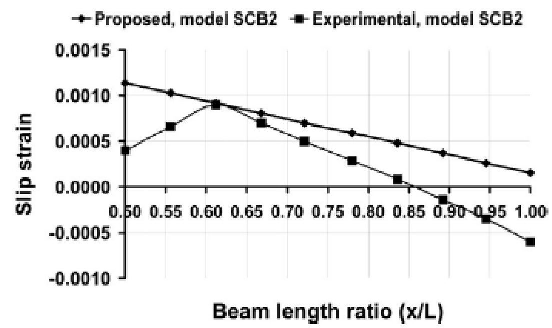

Figure 17a Interface slip strain values of the upper slab for beam model SCB2.

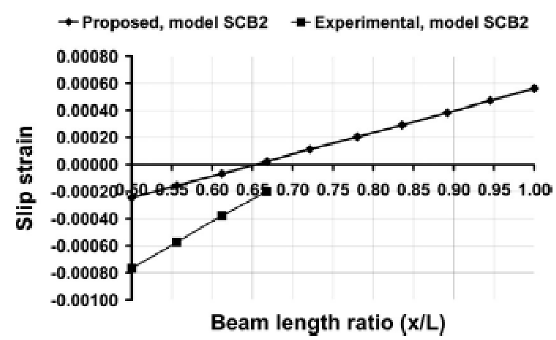

Figure 17b Interface slip strain values of the lower slab for beam model SCB2.

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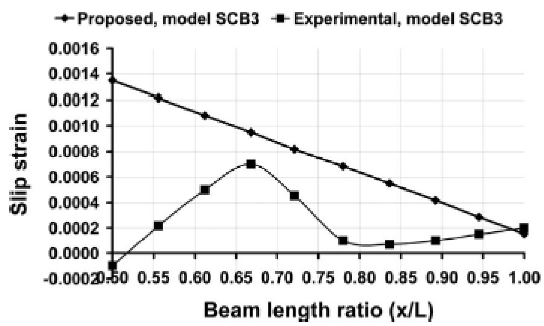


Figure 18a Interface slip strain values of the upper slab for beam model SCB3.

Fig. 20.a compares the results obtained for the ultimate load values, taking into account the presence or absence of the lower slab. It has to be noted that in case of existing the lower slab, the proposed ultimate load values increase by an amount of 0.075% in the case of model SCB1, and by an amount of 0.11% in the case of model SCB2. This increasing value is become 0.068% in the case model SCB3. One can observe that the presence of the lower slab increases the strength capacity by an average amount 0.08% for all experienced composite models. This means that the existence of the lower slab has a minor effect on the strength capacity values.

Fig. 20.b presents a comparison of the results of the maximum deflection values. Again, the above three models were investigated twice in order to experience the effect of removing

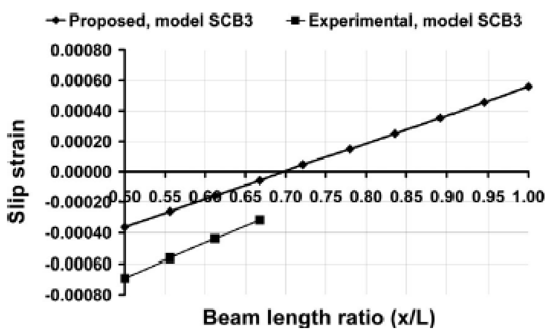


Figure 18b Interface slip strain values of the lower slab for beam model SCB3.

the lower slab. It has to be noted that in case of existing the lower slab, the proposed values of the maximum deflection decrease by a significant average amount of 20% for all beam models.

6.2. The influence of varying the steel beam height

In this part, the effect of changing the height of steel beam on the characteristics of the collapse stage for the continuous double steel-concrete composite beam is investigated. Five steel beam heights of values 110 mm, 130 mm, 150 mm, 170 mm, and 190 mm were proposed and applied to the model SCB1, as a case study. Fig. 21 demonstrates the effect of varying steel beam height on the fracture characteristics of the proposed model.

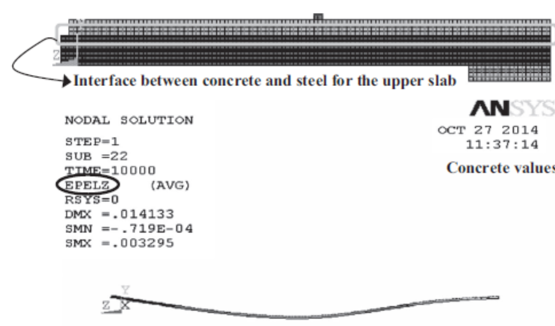
Fig. 21.a presents a comparison of the results of the ultimate load values. This study was applied to the model SCB1 with the same height values as indicated previously. It has to be noted that the case of the beam model with steel beam height of 110 mm had the minimum ultimate load value, whereas the case of the steel beam height of 190 mm had the maximum one. The increase in the ultimate load for two consecutive heights (e.g. 130 mm and 150 mm) reached a significant value of approximately 30% for all models.

Fig. 21.b compares the results obtained for the maximum deflection values, taking into account the same model and the proposed steel beam heights as indicated above. It has to be observed that the case of the 110 mm steel beam height had the maximum value of the maximum deflection, whereas the case of the steel beam with height of 190 mm had the minimum one. The decrease in the maximum deflection values for two consecutive heights (e.g. 130 mm and 150 mm) reached a significant value of approximately 12% for all models.

6.3. The influence of varying lower slab length

This part contains study of the impact of changing the lower slab length on the collapse stage characteristics for the continuous double steel-concrete composite beam. Four lower slab lengths of values 1000 mm, 1200 mm, 1400 mm, and 1600 mm were proposed and applied to the model SCB1, as a case study. Fig. 22 exhibits the effect of varying the lower slab length on the fracture characteristics of the proposed model.

Fig. 22.a presents a comparison of the results of the ultimate load values. This study was applied to the model SCB1 Figure 17a Interface slip strain values of the upper slab for beam model SCB2.



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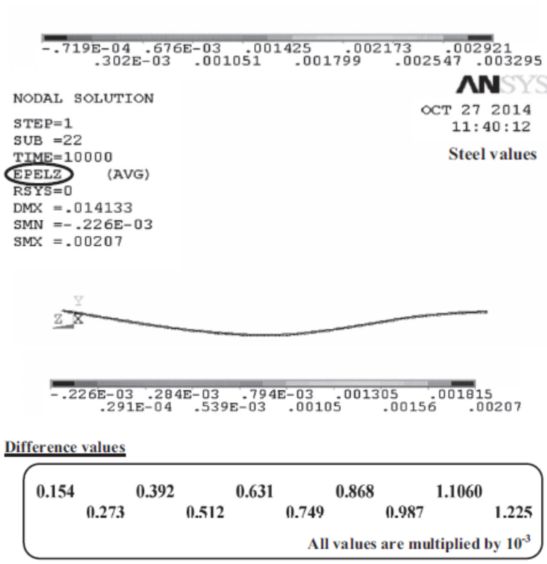


Figure 19 Difference between the interface longitudinal strains of concrete and steel along the beam length direction for the upper slab of the beam model SCB1 (slip strain values).

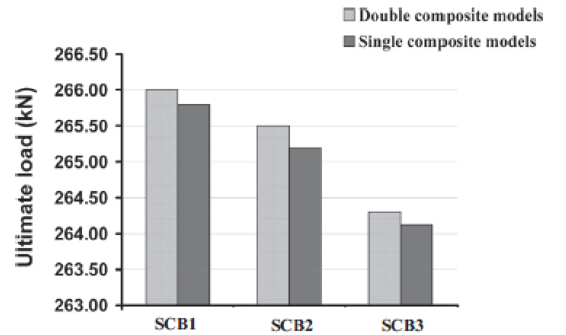
with the same lower slab length values as mentioned above. It has to be observed that the case of the beam model with lower slab length of 1600 mm had the maximum ultimate load value, whereas the case of the lower slab length of 1000 mm had the minimum one. The increase in the ultimate load for two consecutive slab lengths (e.g. 1200 mm and 1400 mm) reached non-notable value of approximately 0.6% for all models.

Fig. 22.b compares the results obtained for the maximum deflection values, taking into account the same model and the proposed lower slab lengths as indicated above. It has to be noted that the case of the beam model involving lower slab length of 1600 mm had the minimum value of the maximum deflection, whereas the case of the lower slab length of 1000 mm had the maximum one. The decrease in the maximum deflection values for two consecutive slab lengths (e.g. 1200 mm and 1400 mm) reached a remarkable value of approximately 5% for all models.

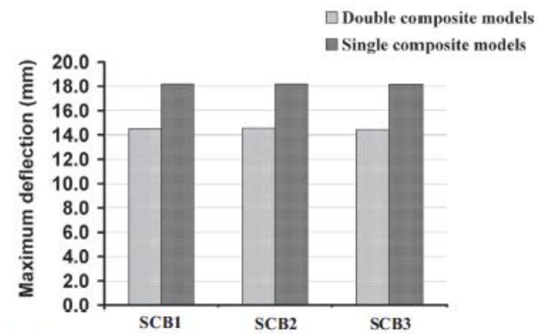
6.4. The influence of varying lower slab thickness

The influence of changing the lower slab thickness on the characteristics of the collapse stage for the continuous double steelconcrete composite beam is studied herein. Four lower slab thicknesses of values 80 mm, 100 mm, 120 mm, and 140 mm were proposed and executed to the model SCB1, as a case study. Fig. 23 explicates the effect of varying the lower slab thickness on the fracture characteristics of the proposed model.

Fig. 23.a presents a comparison of the results of the ultimate load values. This study was applied to the model SCB1 with the same lower slab thickness values as mentioned above. It has to be noted that the case of the beam model with lower slab thickness of 80 mm had the minimum ultimate load value, whereas the case of the lower slab thickness of 140 mm had the maximum one. The increase in the ultimate load for two

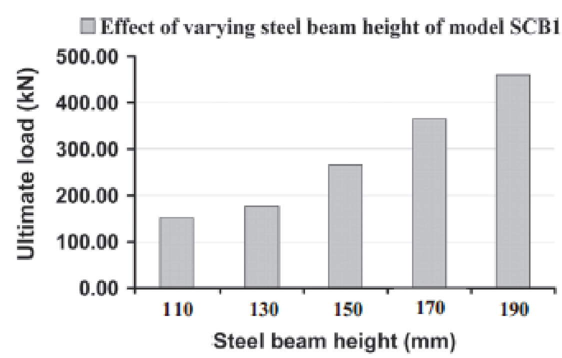


a. Ultimate load as a function of the composite action



b. Maximum deflection as a function of the composite action

Figure 20 Fracture characteristics as a function varying composite action.



(a) Ultimate load as a function of the steel beam height of model SCB1

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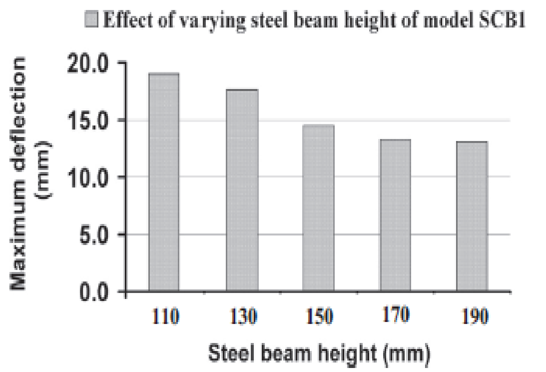
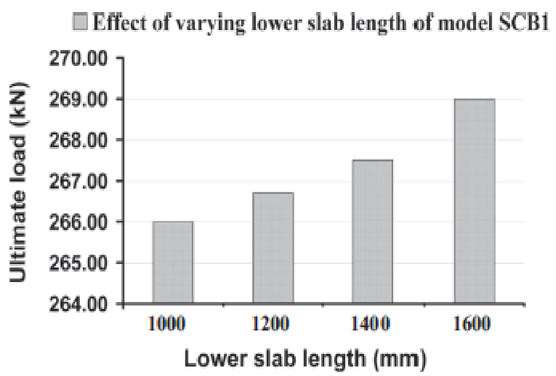
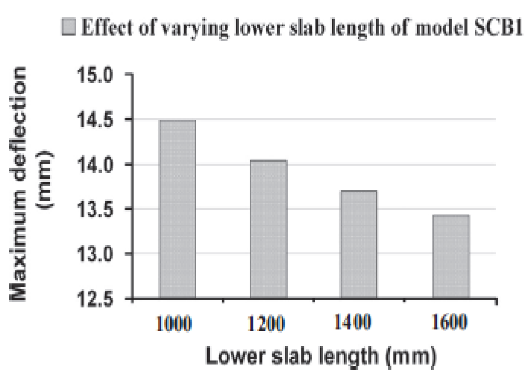


(b) Maximum deflection as a function of the steel beam height of model SCB1

Figure 21 Fracture characteristics as a function varying steel beam height of model SCB1



(a) Ultimate load as a function of the lower slab length of model SCB1



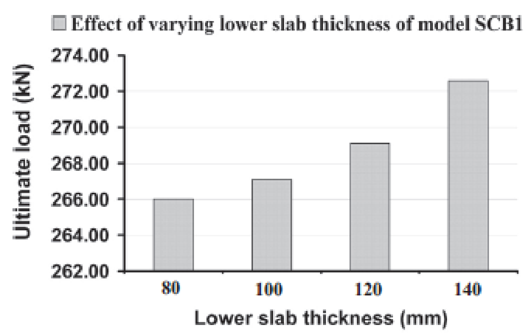
(b) Maximum deflection as a function of the lower slab length of model SCB1

Figure 22 Fracture characteristics as a function varying lower slab length of model SCB1.

consecutive slab thicknesses (e.g. 100 mm and 120 mm) reached non-remarkable value of approximately 0.85% for all models. Fig. 23.b compares the results obtained for the maximum deflection values, taking into account the same model and the proposed lower slab thicknesses as mentioned above. It has to be observed that the case of the beam model including lower slab thickness of 80 mm had the maximum value of the maximum deflection, whereas the case of the lower slab thickness of 140 mm had the minimum one. The decrease in the maximum deflection values for two consecutive slab thicknesses (e.g. 100 mm and 120 mm) reached a slightly remarkable average value of approximately 3.5% for all models.

6.5. The influence of varying the head studs arrangement

In order to complete the parametric study, the effect of changing the arrangement of the head studs on the characteristics of the collapse stage for the continuous double steelconcrete composite beam is discussed. Three cases of head studs arrangement were proposed. The first case is when the studs were fully arranged along the whole length of the upper and the lower interface slab-steel beam surfaces. The second case is when the studs were arranged with staggered shape, while the third case is when the studs were completely removed. This study was applied to the model SCB1, as a case study. Fig. 24 indicates the effect of varying the studs arrangement on the fracture characteristics of the proposed model.



(a) Ultimate load as a function of the lower slab thickness of model SCB1

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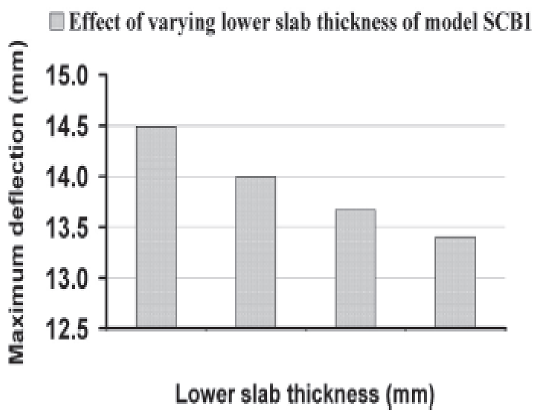
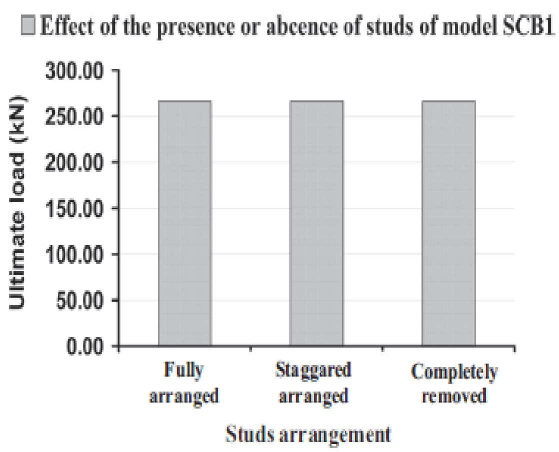
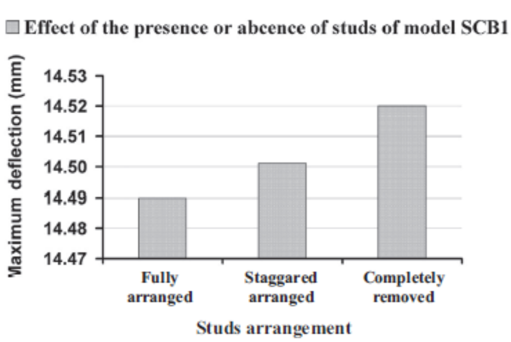


(b) Maximum deflections as a function of the lower slab thickness of model SCB1

Figure 23 Fracture characteristics as a function varying lower slab thickness of model SCB1.



(a) Ultimate load as a function of the studs arrangement of model SCB1



(b) Maximum deflection as a function of the studs arrangement of model SCB1

Figure 24 Fracture characteristics as a function varying studs arrangement of model SCB1.

Fig. 24.a presents a comparison of the results of the ultimate load values. This study was applied to the model SCB1 with the same cases of the studs arrangement as mentioned above. It has to be noted that the change of the shape of the studs arrangement has no influence on values of the ultimate load.

Fig. 24.b compares the results obtained for the maximum deflection values. It has to be observed that the beam model including fully studs arrangement had the minimum value of the maximum deflection, whereas the case of the completely removed head studs had the maximum one. The increase in the maximum deflection values for two consecutive studs arrangement (e.g. fully and staggered arrangement) reached a very slightly remarkable value of approximately 0.08% for all models.

6.6. The influence of varying the head studs diameter

The effect of changing the value of the diameter of the head studs on the characteristics of the collapse stage for the continuous double steel-concrete composite beam is examined as a part of this study. Four cases of head studs diameter of values 13 mm, 16 mm, 19 mm, and 22 mm were suggested and implemented to the model SCB1, as a case study. Fig. 25 clarifies the effect of varying the studs diameter on the fracture characteristics of the proposed model.

Fig. 25.a presents a comparison of the results of the ultimate load values. This study was applied to the model SCB1 with the same values of the head studs diameters as stated above. It has to be noted that the change of the head studs diameter has no influence on values of the ultimate load.

Fig. 25.b compares the results obtained for the maximum deflection values. It has to be concluded that the beam model containing 13 mm head studs diameter had the minimum value of the maximum deflection, whereas the case of the studs diameter of 22 mm had the maximum one. The increase in the maximum deflection values for two consecutive studs diameter (e.g. 16 mm and 19 mm) attained a very slightly notable average value of approximately 0.05% for all models.

7. Conclusions

This paper investigates the behavior of the continuous steelconcret composite beam taking into account the existence of the double composite action and the head stud shear connectors. Based on the finite element numerical study and the experimentally available results, the following main conclusions can be extrapolated:

1. A numerical proposed model based on the finite element theory can be used to examine the geometrical and mechanical characteristics in steel-concrete composite

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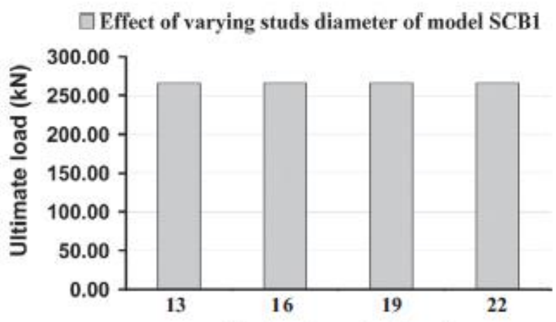
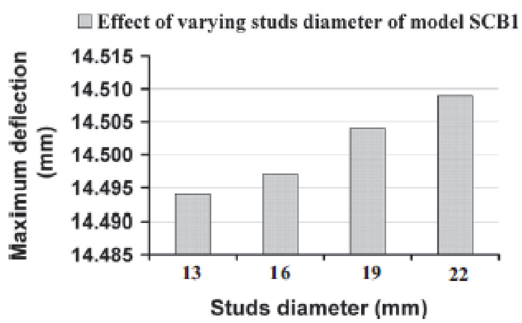


(a) Ultimate load as a function of the studs diameter of model SCB1



(b) Maximum deflection as a function of the studs diameter of model SCB1

Figure 25 Fracture characteristics as a function varying studs diameter of model SCB1.

beam with double composite action, resulting in a good agreement when comparing to available full-scale test data.

2. The comparison of the strength capacity values obtained by the proposed and experimental models leads to a good agreeable between them. An average increase in the proposed strength capacity values of approximately 29% compared to the experimentally available data was concluded for all proposed models. However, a softer performance of the validation figures (load – deflection curves) is observed for the developed models than that of the experimental results. This is mainly due to the varying range of the William-Warnke constants values, which must be chosen carefully by a sensitivity analysis in order to encounter the Ansys convergence problems as mentioned above.

3. An increase in the proposed interface steel-concrete slip values of approximately 38% compared to the experimentally available data was observed, leading to slightly non-agreeable results.

4. An increase in the proposed interface steel-concrete slip strain values of approximately 49% and 55% compared to the experimentally available data was observed for both the upper and the lower slabs respectively, leading to somewhat non-agreeing values between them. This is due to the difference in values of friction (shear slip) at the slab-steel beam interface between the analytical and experimental approaches, because of the presence of the contact elements for simulating this friction. This means that the shear slip has a significant contribution to composite beam deformation, which cannot be negligible.

5. Parametric studies were carried out to look at the impact of removing the lower slab, the effect of varying steel beam height and the lower slab length and thickness, and the effect of changing the head studs arrangement and diameter. These studies were performed to investigate the effect of these parameters on the strength and the deflection capacity of the steel-concrete composite beams having double composite action.

6. The presence of the lower slab increases the proposed strength capacity values by an average amount 0.08% for all experienced composite models, leading to a minor effect on the strength capacity. Moreover, the proposed values of the maximum deflection decrease by a significant average amount of 20% for all beam models when removing the lower slab.

7. In comparison with the five suggested cases of steel beam height involved in the parametric study, it can be observed that the more increase the steel beam height is the bigger the ultimate load values are.

8. Moreover, this study showed that the smaller the lower slab length or thickness is the smaller the ultimate load values and the bigger the maximum deflection values are.

9. It can be noted that the change of the shape of the studs arrangement has no influence on the values of the ultimate load. In addition, the beam model including fully studs arrangement had a minimum value of the maximum deflection, whereas the case of the completely removed head studs had the maximum one.

10. In comparison with the five head stud diameters suggested in this study, one can concluded that the change of this parameter has no effect on the values of the ultimate load. In addition, it has to be noted that the smaller the head studs diameter is the smaller the maximum deflection values are.

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