The Effect of Number and Distribution of Shear Connectors on the Behavior of Composite Girders

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ABSTRACT

Composite construction is one of the methods of construction which is widely used in civil engineering projects. The scope of the present research is to study the behavior of composite girders consisting of a high-strength concrete deck slab integrated with two steel I-beams by headed steel stud connectors. The research was conducted by experimental and theoretical investigations. In the experimental work, four models were constructed as test samples. The test samples were designed to be tested and failed in flexure. The dimensions of deck slab for each sample were (1000×2000×70mm: width×length×thickness), while the dimensions of each I-beam were (142×2000mm: depth×length). In the theoretical work, the test samples were modeled numerically and analyzed using finite element method. The numerical models were made in three dimensions by software package (ANSYS 12.1, 2009). Verification of the numerical results was conducted by comparison with experimental results. It is concluded that the redistribution of shear connectors keeping the same number by minimizing the spacing at the end zones of the span and maximizing it at the mid-span zone enhances the behavior of the composite girder by increasing ultimate load and decreasing deflection and end-slip, and this improvement may reach (8%).

KEYWORDS: Steel-concrete composite construction, Composite girders, Stud shear connectors, Finite element analysis.

INTRODUCTION

Composite construction is integrating two materials by proper connectors to produce a member with new section (composite section) with the advantage of good specifications of the two materials.

Composite girders can be constructed from a concrete deck slab and steel I-beam connected together by steel headed studs. Such construction was used in the present work. The main advantages of such construction are: (1) avoiding high depth of girders and then excessive self-weight especially for the case of long span; (2) increasing the stiffness of the member by: a. resisting the tensile stresses by steel I-beam to avoid weak resistance of concrete in the tension zone of the girders. b. resisting compressive stresses by concrete deck slab to avoid the problem of buckling of steel in the compression zone of the girders.

Still there is a main point which controls the performance of the composite girders, which is the composite action. Composite action is the degree of the connection (or bond) between the concrete deck slab and steel I-beam. The structural performance for the composite section depends on the achievement of
composite action. The degree of composite action is mainly affected by mechanical and geometrical properties of shear connectors. The degree of the composite action is ranging between the case of zero bond when there is no shear connectors between the integrated materials and the case of full bond when there is enough number of shear connectors. In the case of full bond, one can assume that there will be no relative slip between concrete slab and steel beam and the two components will act as one unit. Complete connection is not preferable in composite section since the non-deformable connectors may cause crushing in concrete (Al-Thebhawi, 2005; Kadhim, 2007; Zeinul-Abideen, 2010).

The main role of shear connectors is to resist longitudinal slip along the contact surface and consequently resist shear forces, in addition to resisting the vertical splitting forces which try to separate the composite materials. Fig. (1) shows some types of shear connectors. Stud headed shear connector is the most popular type of connecting device to be used in composite construction. The reason is that tee and channel connectors resist shear forces in one direction only while the headed stud can resist shear forces similarly in any direction within the plane of slip surface. In addition, there are reasons related to availability, simplicity in welding, obstruction reinforcements and cost.

![Figure (1): Some types of shear connectors (Johnson, 1994; Nethercot, 2004; Cosenza and Zandonini, 1999; Al-Darzi and Al-Juboory, 2013)](attachment:image.png)

Lam and El-Lobody (2001) developed a finite element model to simulate the structural behavior of headed stud shear connector in steel-concrete composite beam. Nonlinear three-dimensional finite
element analysis using software package (ABAQUS) was developed to investigate the load-slip behavior of the headed stud in a push-off test.

Bachachi (2007) carried out a theoretical investigation to predict load-deflection behavior of composite beams consisting of a reinforced concrete slab and a steel beam with shear connectors under static loads. The composite beams were modeled by using software package (ANSYS 9) as a nonlinear three-dimensional finite element model.

Chang et al. (2005) carried out an experimental investigation to predict the static and fatigue behavior of large stud shear connectors for steel-concrete composite bridges. Large shear stud can be an excellent alternative in high shear areas of steel-concrete composite bridges.

Barth and Wu (2006) carried out an experimental investigation by fabricating two composite steel girders from high-performance concrete and one four-span continuous composite steel bridge tested to failure and used to validate a proposed FEA model.

**Experimental Work**

The experimental work was conducted through testing four composite girder samples, each consisting of a concrete deck slab and two steel I-beams connected together by steel headed studs. Fig.(2) shows the dimensions of the test sample. Dimensions of the concrete deck slab are (1000×2000×70mm: width×length×thickness), reinforced with a double layer of mesh wire of (5.6 mm) diameter and opening size of (150 mm). The dimensions of steel I-beam are (142×2000mm: depth×length). Headed steel studs (shear connectors) were welded to the top flange of each I-beam. The diameter of used shear connectors is (9.8 mm) and the overall length is (40 mm). Fig. (3) shows the typical cross-section of the tested samples in the investigation.

![Composite steel-concrete sample](image1)

Figure (2): Composite steel-concrete sample (dimensions in millimeters)

![Cross-section of a typical sample](image2)

Figure (3): Cross-section of a typical sample (dimensions in millimeters)
The control sample contains (38) shear connectors, (19) shear connectors for each I-steel beam. The shear connectors were distributed uniformly along the longitudinal axis of each I-steel beam by a spacing of (100 mm) center to center of headed stud shear connector.

The test samples are named as (BG1, control sample), (BG2), (BG3) and (BG4). All the samples are loaded monotonically by two line loads, applied at a distance of (320 mm) from the mid-span for each line. The test samples were simply supported during the test. Fig.(4) shows distribution and number of shear connectors for the test samples. Table (1) shows details of each sample; namely number of shear connectors and their distribution.
Table 1. Details of test samples

<table>
<thead>
<tr>
<th>Name of Sample</th>
<th>Shear Connectors for each I-Steel Beam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
</tr>
<tr>
<td>BG1</td>
<td>19</td>
</tr>
<tr>
<td>BG2</td>
<td>14</td>
</tr>
<tr>
<td>BG3</td>
<td>19</td>
</tr>
<tr>
<td>BG4</td>
<td>19</td>
</tr>
</tbody>
</table>

Properties of Materials

Steel Beams

Steel beams of I-section shape were used in constructing the tested samples. The type of steel beam is (European IPE 140). The material of the steel beam was tested according to (ASTM A370). Table (2) shows the properties of the steel beam.

<table>
<thead>
<tr>
<th>Sectional Area</th>
<th>I&lt;sub&gt;s&lt;/sub&gt;</th>
<th>Steel Grade</th>
<th>Yielding Stress σ&lt;sub&gt;y&lt;/sub&gt; (MPa)</th>
<th>Tensile Strength σ&lt;sub&gt;u&lt;/sub&gt; (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1645 mm&lt;sup&gt;2&lt;/sup&gt;</td>
<td>5.28 x10&lt;sup&gt;6&lt;/sup&gt; mm&lt;sup&gt;4&lt;/sup&gt;</td>
<td>ST 37</td>
<td>344</td>
<td>494</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth</th>
<th>Flange Width</th>
<th>Flange Thickness</th>
<th>Web Thickness</th>
<th>Radius of Curvature</th>
<th>Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>142 mm</td>
<td>72 mm</td>
<td>6.5 mm</td>
<td>5.5 mm</td>
<td>7 mm</td>
<td>13 kg/m</td>
</tr>
</tbody>
</table>

Headed Steel Stud Shear Connectors

The stud shear connectors were used after cutting the threaded part from the original bolts of type (M.H 8.8). Overall length (40 mm) and diameter (9.8 mm), as well as head diameter (17 mm) and height of head (6.3 mm) were used in the construction of test samples. The yielding stress for studs is (σ<sub>y</sub>=240MPa), and the tensile strength is (σ<sub>u</sub>=1010MPa).

Steel Reinforcement of Concrete

The type of steel reinforcement used in this study was (MD 25). Two layers of mesh steel wire reinforcement (BRC) of deformed bars with a diameter of (5.6 mm) and a spacing of (150 mm c/c) in both directions were used. The area of steel given by such reinforcement is (170 mm<sup>2</sup>/m) for one layer. The yielding stress is (σ<sub>y</sub>=462MPa), and the tensile strength is (σ<sub>u</sub>=507MPa).

Concrete

The materials (fine aggregate, coarse aggregate, cement and water) used in preparing the concrete were tested according to the standard specifications. Mixing of concrete was carried out using electrical tilting drum mixer.

EXPERIMENTAL RESULTS

The obtained results from the experimental testing of the present study are:

1. Deflections at:
   a. Central point of the sample which lies at the bottom face of the concrete slab. The symbol of the deflection at this point is (DCC).
   b. Mid-span point of one of the steel beams which lies at the bottom face of the bottom flange of the steel beam. The symbol used for this point is (DCS).
   c. Points beneath the two line loads at the bottom face of the bottom flange of the steel beam. The symbols used for these points are (D1) and (D2).
2. Slip at ends between concrete slab and one of the steel beams. The symbols of these slips are (S1) and (S2).

All the results were recorded at each stage of loading. The value of load was obtained from the analog reader of the test machine. The experimental data were obtained by using dial gauges for deflection and slip, demec discs and extensometer for normal strain.

Table (3) shows the ultimate load recorded for each sample and the load of the first crack formed in the concrete slab and the ratio between the two loads. In addition, the values of compressive strength of concrete of each sample are listed. Sample (BG1) is the control sample, it failed under an ultimate load of (P_u=405 kN). There is a slight reduction in the value of the ultimate load by a ratio of (1.5 %) for the sample (BG2), (P_u=399 kN). This reduction occurred due to the decrease of the number of shear connectors from (19) in (BG1) to (14) in (BG2) in each of the two steel beams, and this decrease was obtained by omitting the shear connectors in the middle third of the sample span. The reduction in the value of the ultimate load is slight because the omitted shear connectors lie in the region of zero shear force.

Table 3. Ultimate load and first crack load

<table>
<thead>
<tr>
<th>Sample</th>
<th>First Crack Load P_cr (kN)</th>
<th>Ultimate Load P_u (kN)</th>
<th>P_cr / P_u (%)</th>
<th>Compressive Strength (f_{cu})</th>
<th>Notes (variable parameter)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BG1</td>
<td>185</td>
<td>405</td>
<td>46</td>
<td>55</td>
<td>Control Beam</td>
</tr>
<tr>
<td>BG2</td>
<td>135</td>
<td>399</td>
<td>34</td>
<td>69</td>
<td>Number of Shear Connectors</td>
</tr>
<tr>
<td>BG3</td>
<td>185</td>
<td>436</td>
<td>42</td>
<td>75</td>
<td>Distribution of Shear Connectors 75-100 mm</td>
</tr>
<tr>
<td>BG4</td>
<td>135</td>
<td>425</td>
<td>32</td>
<td>57</td>
<td>60-120-180 mm</td>
</tr>
</tbody>
</table>

Figure (5): Crack patterns at failure for BG1-control sample

The results of ultimate loads show that (P_u) values for the samples (BG3) and (BG4) are (436 kN) and (425 kN), respectively. The increase percentages of the ultimate loads for samples (BG3) and (BG4) are (8%)
and (5%) respectively. The reason of this increase is related to the increase of shear strength capacity at the ends of the sample span, through minimizing the spacing of shear connectors, which are the regions of higher shear force value along the sample span. However, such increase is limited to a small amount, and this may be due to the reduction of shear strength at the neighborhood of critical sections due to shear connectors' spacing increase. The compressive strength values for the samples (BG2) and (BG3) are higher than that for the control sample (BG1). This causes an increase in the ultimate load. For this reason, modified numerical models with compressive strength (55 MPa) were built. The modified numerical models for (BG2) and (BG3) show that the ultimate load for (BG2) is (389 kN) and for (BG3) is (413 kN). Notice that the compressive strength for sample (BG4) is approximately equal to that for control sample (BG1).

Fig.(5) to Fig.(8) show the state and the crack pattern of the samples after test.

Figure (6): Crack patterns at failure for BG2-sample without shear connectors at mid-third of span (region of zero shear)

Figure (7): Crack patterns at failure for BG3-unequal spacing of shear connectors (75-150) mm
The recorded results of deflection for all the tested samples are listed in Table (4) with the corresponding ultimate loads.

Table 4. Maximum deflections

<table>
<thead>
<tr>
<th>Sample</th>
<th>BG1</th>
<th>BG2</th>
<th>BG3</th>
<th>BG4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Load $P_u$ (kN)</td>
<td>405</td>
<td>399</td>
<td>436</td>
<td>425</td>
</tr>
<tr>
<td>Deflection (mm)</td>
<td>DCS</td>
<td>24.19</td>
<td>41.83</td>
<td>44.02</td>
</tr>
<tr>
<td></td>
<td>DCC</td>
<td>26.62</td>
<td>39.20</td>
<td>38.34</td>
</tr>
<tr>
<td></td>
<td>D1</td>
<td>25.87</td>
<td>35.41</td>
<td>40.50</td>
</tr>
<tr>
<td></td>
<td>D2</td>
<td>--</td>
<td>34.20</td>
<td>37.15</td>
</tr>
</tbody>
</table>

Fig. (9) shows the relation between the applied load and the vertical deflection for each tested sample.

Reviewing the previous curves and tables of deflection indicates that the maximum deflection always occurs at mid-span either in the concrete deck or in the steel beams. The difference in deflection between concrete deck and steel beam is not large. This difference may be due to flexural effect in transverse direction (short direction) of the concrete deck slab. The relative vertical displacements at ultimate load between concrete slab and steel beam at mid span are (0.0001mm for BG1, 0.2471 mm for BG2, 0.0mm for BG3 and 0.0001mm for BG4). Fig. (10) and Fig. (11) show a comparison between the control sample (BG1) and the other test samples for a deflection of the steel beams at mid-span point (DCS) which is almost the maximum deflection.

Omitting shear connectors in mid-third span increases deflection and reduces the stiffness of the girder represented by the slope of load-deflection curve. The increasing trend of percentage of the deflection is increased with loading increase. The maximum deflection corresponding to the ultimate load of samples (BG1 and BG2) was (24.19 and 41.83 mm), respectively. Thus, there is an increase in deflection of about (73%); although there is a little decrease in ultimate load (about 1.5%).
Figure (9): Load-deflection curve of test samples

BG1

BG2

BG3

BG4

Figure (10): Effect of number of shear connectors on load-deflection curve (deflection at mid-span point of steel beam)

Figure (11): Effect of distribution of shear connectors on load-deflection curve (deflection at mid-span point of steel beam)
Changing distribution of shear connectors in the samples (BG3 and BG4) indicates that there is a slight effect on the stiffness of the girders. Deflections of sample (BG3) are close to those for the control sample (BG1) at the same increments of loading, while deflections of sample (BG4) are a little bigger than those for sample (BG1). The maximum deflections of samples (BG3 and BG4) at ultimate load are (44.02 and 27.07 mm), respectively.

End slips at the ends of one of the steel beams were recorded for each load increment. End slip readings are denoted as (S1) and (S2). Fig.(12) shows the load versus average slip of (S1 and S2) for all tested samples.

The comparison of end slip values shown in Fig.(12) indicates that the end slip results of samples (BG3) and (BG4) are less than that for control sample (BG1). This means that the redistribution of shear connectors included in samples (BG3) and (BG4) enhanced the slip behavior. Omitting the shear connectors in the middle third of span for sample (BG2) increased the end slip. At the advanced stages of loading (second half of loading), the rate of increasing slip in sample (BG2), with the absence of shear connectors at mid-third span) is greater than those for the other samples. Redistribution of shear connectors in samples (BG3 and BG4) increases the slope of load-slip curve; i.e., the slip stiffness of the girder was increased in these samples, especially in sample (BG3). The reason behind that is increasing the number of shear connectors at the ends which are the zones of higher shear force.

**Finite Element Modeling**

Finite element analysis, as used in structural engineering, determines the overall behavior of a structure by dividing it into a number of single elements, each of which has well-defined mechanical and physical properties. Modeling of the constitutive material properties is an important aspect of any finite element analysis. The constitutive model should correctly describe the behavior of the material under uniaxial and multiaxial states of loading. Finite element modeling and analysis were carried out to simulate the behavior of the four tested composite steel-concrete girders from linear through non-linear response and up
The choice of the proper element type is very important in the finite element analysis. The chosen element type depends upon the geometry of the structure and the number of independent space coordinates necessary to describe the problem. Composite members are made of different materials; i.e., steel, concrete, shear connectors and reinforcing bars, which are brought together to constitute a composite system. Each component of composite member should be modeled by the proper element type and then each type of element should be provided by the properties according to the material of that component. In the present study, three-dimensional model was used to analyze composite girders consisting of a concrete deck slab and two I-steel beams integrated by steel studs shear connectors. The concrete slab was divided in its length, width and depth into brick elements (SOLID65). Element type (SHELL43) (or 4-node plastic small strain shell) was used to model steel I-beam. Reinforcement of concrete and stud connectors were modeled by element type (LINK8). Element (COMBIN39) was used, in this study, to simulate the behavior of the shear connectors in resisting the tangential forces between the concrete slab and the I-steel beams. The contact between concrete slab and steel beams produces normal forces and tangential forces acting on the plane of contact. This action was modeled by using 3-D point-to-point contact element called (CONTAC52).

Studying the effect of shear connectors’ number and distribution faces a difficulty in the simulation of the connectivity between stud elements with concrete and steel beam elements. If the bond between concrete slab and steel beams is fully bond (which can be achieved by using an excessive number of studs), this difficulty will be solved by connecting directly the neighboring concrete elements and steel beam elements through concerted nodes. Thus, a need for using more types of elements appears to represent the bond action between concrete slab and steel beams.

The required time of analysis is about (60-120 sec).

Figure (13): Geometry of the numerical model

**NUMERICAL RESULTS**

The numerical results of ultimate loads, vertical deflection and horizontal slip are concerned to compare them with those of experimental work. This comparison was conducted to verify the numerical model. The numerical results are presented without any calibration.

Table (5-a) shows a comparison between experimental and numerical ultimate loads for the
study samples. Table (5-b) shows a comparison between numerical samples (with different compressive strengths of concrete slab as the experimental samples) and modified numerical samples (with the same compressive strength (55 MPa) as the control sample).

In general, the ultimate loads predicted by the numerical analysis are greater than those of experimental testing.

**Table (5-a). Comparison of load and deflection at service and ultimate stages for the tested samples**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Service Load</th>
<th>Deflection (DCS) at Service Load (mm)</th>
<th>Ultimate Load $P_u$ (kN)</th>
<th>Deflection (DCS) at Ultimate Load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BG1</td>
<td>263</td>
<td>278</td>
<td>5.4</td>
<td>6.35</td>
</tr>
<tr>
<td>BG2</td>
<td>259</td>
<td>260</td>
<td>0.3</td>
<td>7.88</td>
</tr>
<tr>
<td>BG3</td>
<td>283</td>
<td>299</td>
<td>5.6</td>
<td>7.49</td>
</tr>
<tr>
<td>BG4</td>
<td>276</td>
<td>287</td>
<td>3.8</td>
<td>7.61</td>
</tr>
</tbody>
</table>

The percentage of difference for the ultimate loads is between (0.3-5.6) % for all the samples as shown in Table (5-a). The deflection in numerical models is in general smaller than that in experimental samples, and the percentages of variation are between (12.6-27.0)% at the ultimate load and (10.8-15.9)% at the service load (65% of ultimate load). The exception is that the numerical deflection of sample (BG1) is a little greater than that in the experimental sample. The percentage variation for sample (BG1) is (6.4%) at ultimate load and (4.8%) at service load. The percentage of variation in deflection for sample (BG1) is very small; it is almost equal zero. Hence, in general, the numerical models are stiffer.

**Table (5-b). Comparison of load and deflection at service and ultimate stages for the numerical samples and modified numerical samples**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Service Load</th>
<th>Deflection at Service Load (mm)</th>
<th>Ultimate Load $P_u$ (kN)</th>
<th>Deflection at Ultimate Load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BG1</td>
<td>278</td>
<td>278</td>
<td>0</td>
<td>6.66</td>
</tr>
<tr>
<td>BG2</td>
<td>253</td>
<td>260</td>
<td>3</td>
<td>6.91</td>
</tr>
<tr>
<td>BG3</td>
<td>281</td>
<td>299</td>
<td>4</td>
<td>6.38</td>
</tr>
<tr>
<td>BG4</td>
<td>287</td>
<td>287</td>
<td>0</td>
<td>6.40</td>
</tr>
</tbody>
</table>

Figs. (14-17) show a comparison between experimental and numerical results for deflection and slip.
The Effect of Number...

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Figure (14): Load-deflection and load-slip curves of sample (BG1)

Figure (15): Load-deflection and load-slip curves of sample (BG2)

Figure (16): Load-deflection and load-slip curves of sample (BG3)

Figure (17): Load-deflection and load-slip curves of sample (BG4)
This comparison shows in general that the numerical models are stiffer, and the numerical analysis gives a smaller value for the deflection and a greater value for ultimate load with a little difference in the ultimate load values. The results of slip show that there is a deviation in results in the middle third of loading stages and that there is a good convergence at approximately the last third of loading stages.

This may be caused by the following: 1. The concrete of experimental samples is not perfectly homogeneous as assumed in the numerical models. 2. Micro-cracks which may have occurred in concrete due to shrinkage reduce the stiffness in some degree. 3. Cutting and welding processes of the stud connectors produce initial stresses in the studs and the steel beam. 4. Perfect bond between concrete and steel or CFRP reinforcements is assumed in the finite element analysis, but in the experimental samples this bond is not perfect and there is a slip which causes a loss in composite action.

Experimental error of the instrumentation used for the measurements (analog load reader and dial gauges) may have occurred, but the experimental results are still in the same general trend.

Figure (18): General deformed configuration of numerical model

CONCLUSIONS

The general behavior during test process is similar for all tested samples. The first cracks are formed at about (32-46) % of ultimate load for the tested samples. This percentage is changed with varying the number and distribution of the shear connectors. The general trend in ultimate load values is to increase with the increase of shear connectors. Redistribution of shear connectors by increasing spacing in the mid-span zone and decreasing spacing at the ends of the span with keeping the same number of shear connectors improves the behavior of the tested samples through increasing the ultimate load value, decreasing deflection and end-slip. The adopted finite element modeling in general rather overestimates the ultimate load in comparison with the experimental results. The deviation is only in the range from (0.3%) to (5.4%).
REFERENCES


