Original Article

A case study of flexural performance of reinforced concrete beams bonded with steel plates with different thickness

Rafid Saeed Atea

Al-Furat Al-Awsat Technical University/Najaf Technical Institute, Iraq

ABSTRACT

This study investigates the flexural behavior of four reinforced concrete beams. The beams were composed by adding steel plates which have different thickness (2, 3, and 5 mm) in the tension zone to invent out the consequence of the altered plate thicknesses on the flexural behavior on these beams, and the consequence of using typical concrete. The first beam is made of normal concrete (non composite beam) additionally, the other beams are prepared using usual concrete (composite beams by plates). The connection between the concrete and steel plate was by using shear connector, to gain the effective connection between the concrete and steel plate. The study consists of two parts: the first part is an experimental work through casting and testing beams, while in the second part, an analysis has been conducted to the tested specimens by using a three dimensional nonlinear finite element method by ANSYS program (Version 18.1). The increase of ultimate strength for plated beam compared with unplated beam (73%, 86% and 161%) with increase the thickness steel plate (2, 3 and 5) respectively. Concrete strain, crack width and numbers of cracks decrease with increasing the thickness of steel plate.

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1. Introduction

Composite construction has been extensively castoff for building structures over the previous (50 years). Primarily, recognized for beams in buildings, composite components now also hold columns and shear walls, and are regularly active in high-rise structures, remaining to their high axial load capacity and stiffness. The preceding insufficient years have seen the enhancement of the complete composite frame, where the benefits of steel and concrete are combined to offer structural systems of excessive strength and stiffness. This type involves a steel plate forms the sofit of the beam and this performances in combination with the reinforced concrete. Steel plate will effort as enduring formwork and as extra reinforcement to internal reinforcement. A small or large steel plate is collective with reinforced concrete for consistent this new type of composite beam, namely, composite reinforced concrete beam. Shear connectors are welded to the plate to ensure composite
action of the steel plate and the reinforced concrete. Important advantages of composite reinforced concrete beam are greater stiffness. This decreases the deflection of the member, as associated with non-composite structure. Subedi and Balgin [1], approved out an investigational effort containing of four beams, one of which was used as the control. The other three were visibly reinforced with steel plate of (2 mm, 4 mm and 6 mm) thickness on both sides of the web by bolts of (16 mm) in diameter. The collection of a particular thickness of plate was ruled by ease of handling. Al-Ghareib [2] exhibited investigations on twelve (175 mm × 275 mm × 3000 mm) reinforced concrete beams reinforced with altered reinforcement ratios (0.95, 1.43, 2.37, and 3.56%) and maintained by steel plate. The experiment was to explore and revision the flexural behavior of beams, twelve beams were alienated into three groups; each one consists of four beams. The first group consists of beams with normal strength concrete (BN) of nominal compressive strength of (20MPa), the second group is with high strength concrete of nominal compressive strength of (70MPa) and the third group is made of hybrid strength concrete of nominal compressive strength of (70MPa) at the upper third of the section and nominal compressive strength of (20MPa) at the lower two thirds. Hwang et al. [3] presented an experimental and analytical study concerning the seismic retrofitted reinforced concrete frames containing partition walls using the CFRP laminate. The test result showed that the use of CFRP laminate with passable end anchorage was fairly active in refining the shear strength of partition walls. Lei et al. [4] investigated the experimental research and numerical simulation of RC beams strengthened with bonded steel plates, the experimental program was supported by a three-dimensioned finite analysis using ABAQUS. At the end of experiments and finite analysis, it was concluded that the investing strengthening technique can significantly improve the load-carrying capacity and the phenomenon of stress concentration at the end of interface, as well as the damage at interface, can be well simulated with cohesive element provided by ABAQUS. Abd El-Raouf et al. [5] studied the flexural Strength and Toughness of Austenitic Stainless Steel Reinforced High- or White Cast Iron Composite, I- and T-sections, and volume fractions of austenitic stainless steel (310 SS) were examined under three-point bending test. The dimensions of casted beams used for bending test were (50 × 100 × 500 mm3). Carbon and alloying elements diffusion enhanced the metallurgical bond across the interface of casted beams. Carbon diffusion from high-Cr WCI into 310 SS resulted in the formation of Cr-carbides in 310 SS near the interface and Ni diffusion from 310 SS into high-Cr WCI led to the formation of austenite within a network of M7C3 eutectic carbides in high-Cr WCI near the interface. Inserting 310 SS plates into high-Cr WCI beams resulted in a significant improvement in their toughness. All specimens of this metal matrix composite failed in a ductile mode with higher plastic deformation prior to failure. The high-Cr WCI specimen reinforced with I-section of 310 SS revealed higher toughness compared to that with T-section at the same volume fraction. The presence of the upper flange increased the reinforcement efficiency for delaying the crack growth. The rest of researches are concerned with reinforced concrete strengthened by involuntarily committed steel plates. Where several types of connectors were driven into the concrete through the plates, in order to provide sufficient bonds between them and to develop the composite behavior. In this approach, the common mode of failure was flexural with full strength being organized. From above, it can be established that the second approach is the more effective one, and need refining. It has not yet been established visibly the theoretical background to describe and develop the composite action and the prediction criteria for deflection of the strengthened (composite) beam, which were an important marvels for serviceability state.

### 2. Experimental program and tests set-up

In this effort, the consequence of steel plates with different thickness on the behavior of beams and normal concrete is considered. The materials which cast-off were involving of cement, sand, gravel and water. The mixing process used in this study was outlined by Emborg [6], and modified by Al-Jabri [7]. The concrete combination design is allowing to the offered and adapted (ACI 211.1) method using the EFNARC [8]. After mixing, concrete is poured into lightly oiled molds in three layers and well compacted by using

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Cement (kg/m3)</th>
<th>Water (kg/m3)</th>
<th>Sand (kg/m3)</th>
<th>Gravel (kg/m3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC</td>
<td>350</td>
<td>181</td>
<td>700</td>
<td>1155</td>
</tr>
</tbody>
</table>

NC, normal concrete.

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Cube 150 × 150</th>
<th>Cylinder 150 × 300</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_c$ (MPa)</td>
<td>$f_t$ (MPa)</td>
</tr>
<tr>
<td>NC</td>
<td>37</td>
<td>32</td>
</tr>
</tbody>
</table>

$f_c$, compressive strength, tested according to ASTM C39-01. The average of three the specimens at age 28 days are taken.

Ec, Static Modulus of Elasticity, tested according to ASTM C469-02a.

Fr, modulus of rupture, tested according to ASTM C78-02.
2.1. Steel plate

Tensile tests are conducted on several specimens, at least three specimens, prepared from the steel plates, which are used in fabricating the composite beams. Material properties obtained from the coupon tests for steel plates, static yield stress and ultimate strength, are summarized in Table 3.

Details of push-out tests are given later in the following chapter, while the results of tensile tests are given in Table 4.

In Table 4, \( T_u \) is the ultimate tensile force of the steel bolt (stud connector) obtained from tensile test. Yield and ultimate tensile strength is calculated by dividing \( T_u \) on the area of steel bolt (stud connector) based on inner diameter.

2.2. Details of the beams

The sections of beam are designed according to ACI 318M-2008, and the dimensions of beam are \( b = 150 \text{ mm} \), \( h = 250 \text{ mm} \) with length of 1600 mm. Table 5 and Figs. 1 and 2 show all beams test details and flexural reinforcement.

3. Results and discussions

To explore the flexural performance of the beams, one control beam with normal strength concrete and the second plated beam were established.

### Table 3 - Specifications and test results of steel plates-average value.

<table>
<thead>
<tr>
<th>Plate thickness (mm)</th>
<th>Yield stress (N/mm²)</th>
<th>Ultimate stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>208</td>
<td>344</td>
</tr>
<tr>
<td>3</td>
<td>218</td>
<td>355</td>
</tr>
<tr>
<td>5</td>
<td>254</td>
<td>373</td>
</tr>
</tbody>
</table>

### Table 5 - The details of beams.

<table>
<thead>
<tr>
<th>Beam no.</th>
<th>Dimension of plate</th>
<th>Studs no.</th>
<th>Distance between studs</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>RSP1</td>
<td>1400 × 150 × 2</td>
<td>10</td>
<td>60</td>
</tr>
<tr>
<td>RSP2</td>
<td>1400 × 150 × 3</td>
<td>10</td>
<td>60</td>
</tr>
<tr>
<td>RSP3</td>
<td>1400 × 150 × 5</td>
<td>10</td>
<td>60</td>
</tr>
</tbody>
</table>

### Table 4 - Specification and test results of threaded bolt-average values.

<table>
<thead>
<tr>
<th>Steel specimens</th>
<th>Measured diameter (mm)</th>
<th>( T_u ) (kN)</th>
<th>( Q_u ) (kN)</th>
<th>Ultimate shear strength (N/mm²)</th>
<th>Ultimate tensile stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inner</td>
<td>Outer</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel bolt (stud)</td>
<td>8.51</td>
<td>9.6</td>
<td>35</td>
<td>23</td>
<td>356</td>
</tr>
</tbody>
</table>

(i.e. effective area). \( Q_u \) is the ultimate shear force of steel bolt from direct shear test.

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Fig. 1 – Details of all beam.
3.1. General behavior

The experiment consequences are given in Table 6. All beams of this group were intended to fail in flexure.

At about (25–36%) of the ultimate load, more cracks developed at the bottom of the beam which advanced toward the main cracks and often joined them. One or more cracks propagated faster than the others. As estimated, the main cracks for all test beams initiated at the middle zone and (RC, RSP1 and RSP2) presented ductile flexural failure, just (RSP3) showed shear failure. The performance of the control beams was generally similar up to failure.

3.2. Ultimate strength

The noted ultimate loads of the established beams are obtainable in Table 7.

For the tested beams (RSP1, RSP2 and RSP3), which have plates in tension flanges only, the increases in strength were (73%, 86% and 161%) respectively. This improvement is due to different thicknesses of plates which means increase in strength of beams. This intention confirms that the ultimate flexural strength is controlled mainly by the resistance of plates, which is increased with increasing steel plate thickness (Fig. 3).

3.3. Deflections

Load-deflection curves of the established beams at mid span at all stages of loading up to failure were constructed and shown in Fig. 4.

3.4. Concrete crack width

The crack width of the major flexural crack at the level of tensile reinforcement was measured by means of crack deflection

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**Table 6 – Test results of (RC, RSP1, RSP2, RSP3).**

<table>
<thead>
<tr>
<th>Beam no.</th>
<th>Load(kN)</th>
<th>$P_{cr}$</th>
<th>$P_{u}$</th>
<th>$P_{cr}/P_{u}$</th>
<th>$P_{cr}/P_{u}$</th>
<th>Mode of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>25</td>
<td>70</td>
<td>1.15</td>
<td>35.71</td>
<td>Flexure (tensile failure)</td>
<td></td>
</tr>
<tr>
<td>RSP1</td>
<td>33</td>
<td>121</td>
<td>1.3</td>
<td>27.27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RSP2</td>
<td>43</td>
<td>130</td>
<td>1.74</td>
<td>33.03</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RSP3</td>
<td>64</td>
<td>183</td>
<td>2.6</td>
<td>34.97</td>
<td>Shear</td>
<td></td>
</tr>
</tbody>
</table>

$(P_{cr})$, first crack loading for reference beam (RC) = 25 kN.

---

**Table 7 – Ultimate load of tested beams.**

<table>
<thead>
<tr>
<th>Beam no.</th>
<th>Experimental $P_{u}$ (kN)</th>
<th>$P_{cr}/P_{u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC</td>
<td>70</td>
<td>1.08</td>
</tr>
<tr>
<td>RSP1</td>
<td>121</td>
<td>1.73</td>
</tr>
<tr>
<td>RSP2</td>
<td>130</td>
<td>1.86</td>
</tr>
<tr>
<td>RSP3</td>
<td>183</td>
<td>2.61</td>
</tr>
</tbody>
</table>

$(P_{u})$, ultimate loads of reference beams (RC) = 70 kN.
Fig. 4 – Load – deflection curve between the unplated beam and plated beams.

Fig. 5 – Comparison of load – crack width curves for beams.

distribution at mid span of all beams designed to fail in flexure is shown in Fig. 7:

Figs. 8–16 shows the numerical strain distribution of beams. From this figures it can be noticed that the maximum strain occurred along the load path where the inclined crack has been occurred. From the inspections of the strain result, it gives good agreement with experimental result.

4. Conclusions

Based on the results of this study, the following conclusions are as shown: The number of shear connector obligatory in composite reinforced concrete is very much reduced associated with the number required in normal composite construction of comparable strength, where large interaction area donates in moving shear by friction which influences the number of shear connectors. The concrete beam strengthened with epoxy bonded plates. The communual approach of failure for these beams is a early failure which considered by ripping off the plate organized with the concrete cover to which it may be devoted. This was owing to the faintness constructed in the concrete cover as the concrete section has been loaded for the first time. It is value observing that no parting between the plate and the concrete as the epoxy layer is strong sufficient. Also, mutual problem, of this technique is the corrosion of the steel plate surfaces in the long term consequence which will create parting. Many trials were carried out to support the epoxy bonds by extra bolts provided at the plate ends, with extra little enhancement to the performance. For the beams (RSP1, RSP2 and RSP3), increases in strength were (73%, 86% and 161%) respectively. This improvement is due to different thicknesses of plates, which means increase in strength of beams. Increasing the thickness of plates for beams leads to decrease the deflection and width of cracks and numbers of cracks. Increasing the thickness will increase the stiffness of the beams hence the deformations will be reduced. Strain distribution at mid span of all beams designed to fail in flexure is shown in Fig. 7:

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Fig. 7 – Concrete load-strain curve for beams.

Fig. 8 – Normal strain distribution on X-direction for RC beam.
Fig. 9 – Normal strain distribution on X-direction for RSP1 beam.

Fig. 10 – Normal strain distribution on X-direction for RSP2 beam.
Fig. 11 – Normal strain distribution on X-direction for RSP3 beam.

Fig. 12 – Normal strain distribution on Y-direction for RC beam.
Fig. 13 – Normal strain distribution on Z-direction for RC beam.

Fig. 14 – Normal strain distribution on Y-direction for RSP1 beam.

Conflicts of interest

The author declares no conflicts of interest.

REFERENCES


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