Composite Structures 92 (2010) 1391–1398

Contents lists available at ScienceDirect

Composite Structures

journal homepage: www.elsevier.com/locate/compstruct

The effect of CFRP and CFRP/concrete interface models when modelling retrofitted RC beams with FEM

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ARTICLE INFO

Article history:
Available online 14 November 2009

Keywords:
Carbon fibre reinforced plastic (CFRP)
Strengthening
Laminate
Cohesive model
Reinforced concrete beam
Finite element analysis (FEA)

ABSTRACT

Concrete structures retrofitted with fibre reinforced plastic (FRP) applications have become widespread in the last decade due to the economic benefit from it. This paper presents a finite element analysis which is validated against laboratory tests of eight beams. All beams had the same rectangular cross-section geometry and were loaded under four point bending, but differed in the length of the carbon fibre reinforced plastic (CFRP) plate. The commercial numerical analysis tool Abaqus was used, and different material models were evaluated with respect to their ability to describe the behaviour of the beams. Linear elastic isotropic and orthotropic models were used for the CFRP and a perfect bond model and a cohesive bond model was used for the concrete–CFRP interface. A plastic damage model was used for the concrete. The analyses results show good agreement with the experimental data regarding load–displacement response, crack pattern and debonding failure mode when the cohesive bond model is used. The perfect bond model failed to capture the softening behaviour of the beams. There is no significant difference between the elastic isotropic and orthotropic models for the CFRP.

1. Introduction

Upgrading of reinforced concrete structures may be required for many different reasons. The concrete may have become structurally inadequate for example, due to deterioration of materials, poor initial design and/or construction, lack of maintenance, upgrading of design loads or accident events such as earthquakes. In recent years, the development of strong epoxy glue has led to a technique which has great potential in the field of upgrading structures. Basically the technique involves gluing steel or FRP plates to the surface of the concrete. The plates then act compositely with the concrete and help to carry the loads.

The use of FRP to repair and rehabilitate damaged steel and concrete structures has become increasingly attractive due to the well-known good mechanical properties of this material, with particular reference to its very high strength to density ratio. Other advantages are corrosion resistance, reduced maintenance costs and faster installation time compared to conventional materials.

The application of CFRP as external reinforcement to strengthen concrete beams has received much attention from researchers [1–5], but only very few studies have focused on structural members strengthened after preloading [6,7]. The behaviour of structures which have been preloaded until cracking initiates deserves more attention, since this corresponds to the real-life use of CFRP retrofitting.

Researchers have observed new types of failures that can reduce the performance of CFRP when used in retrofitting structures [8]. These failures are often brittle, and include debonding of concrete layers, delamination of CFRP and shear collapse. Brittle debonding has particularly been observed at laminate ends, due to high concentration of shear stresses at discontinuities, where shear cracks in the concrete are likely to develop [9]. Thus, it is necessary to study and understand the behaviour of CFRP strengthened reinforced concrete members, including those failures.

Several researchers have simulated the behaviour of the concrete–CFRP interface through using a very fine mesh to simulate the adhesive layer defined as a linear elastic material [10]. However, they have not used any failure criterion for the adhesive layer. Most researchers who have studied the behaviour of retrofitted structures have, however, not considered the effect of the interfacial behaviour at all [11–13].

In this paper, we use the finite element method to model the behaviour of beams strengthened with CFRP. For validation, the study was carried out using a series of beams that had been experimentally tested for flexural behaviour and reported by Obaidat [14]. Two different models for the CFRP and two different models for the concrete–CFRP interface are investigated. The models are used for analysing beams with different lengths of CFRP applied.

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doi:10.1016/j.compstruct.2009.11.008

2. Experimental work

Experimental data was obtained from previous work by Obaidat [14]. Eight identical RC beams were loaded with a four point bending configuration with a span of 1560 mm, and distance between loads of 520 mm. All beams were 300-mm high, 150-mm wide, and 1960-mm long. The longitudinal reinforcement consisted of two ϕ 12 for tension and two ϕ 10 for compression. Shear reinforcement was sufficiently provided and consisted of ϕ 8 c/c 100 mm, as seen in Fig. 1.

Two control beams were loaded to failure and the other beams were loaded until cracks appeared, then retrofitted using different lengths of CFRP, see Fig. 2. The CFRP was adhered to the bottom surface of the beams with their fibre direction oriented in the axial direction of the beam. Each CFRP plate was 1.2 mm thick and 50 mm wide. Finally the beams were retested, while the deflection and load were monitored.

A comparison of load–deflection curves of retrofitted beams and control beams is presented in Fig. 3. The experimental results showed that the retrofitting using CFRP increased the strength of the beam and the effect increased with the length of the CFRP plate. All retrofitted beams failed due to debonding of the CFRP.

3. Finite element analysis

Finite element failure analysis was performed to model the nonlinear behaviour of the beams. The FEM package Abaqus/standard [15] was used for the analysis.

3.1. Material properties and constitutive models

3.1.1. Concrete

A plastic damage model was used to model the concrete behaviour. This model assumes that the main two failure modes are tensile cracking and compressive crushing [15]. Under uni-axial tension the stress–strain response follows a linear elastic relationship until the value of the failure stress is reached. The failure stress corresponds to the onset of micro-cracking in the concrete material. Beyond the failure stress the formation of micro-cracks is represented with a softening stress–strain response. Hence, the elastic parameters required to establish the first part of the relation are elastic modulus, E_c, and tensile strength, f_{ct}, Fig. 4a. The compressive strength, f'_c, was in the experimental work measured to be 30 MPa. E_c and f_{ct} were then calculated by [16]:

$$E_c = 4700\sqrt{f'_c} = 26,000 \text{ MPa} \tag{1}$$

$$f_{ct} = 0.33\sqrt{f'_c} = 1.81 \text{ MPa} \tag{2}$$

where f'_c, is given in MPa.

To specify the post-peak tension failure behaviour of concrete the fracture energy method was used. The fracture energy for mode I, G_f, is the area under the softening curve and was assumed equal to 90 J/m^2, see Fig. 4b.

The stress–strain relationship proposed by Saenz [17] was used to construct the uni-axial compressive stress–strain curve for concrete:

$$\sigma_c = \frac{E_c \varepsilon_c}{1 + (R + R_E - 2)\left(\frac{\varepsilon_c}{\varepsilon_0}\right) - (2R - 1)\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2 + R\left(\frac{\varepsilon_c}{\varepsilon_0}\right)^3} \tag{3}$$

where

$$R = \frac{R_E(R_\sigma - 1)}{(R_\varepsilon - 1)^2} - \frac{1}{R_\varepsilon}, \quad R_E = \frac{E_c}{E_0}, \quad E_0 = \frac{f'_c}{\varepsilon_0} \tag{4}$$

and, $\varepsilon_0 = 0.0025$, $R_\varepsilon = 4$, $R_\sigma = 4$ as reported in [18]. The stress–strain relationship in compression for concrete is represented in Fig. 5.

Poisson's ratio for concrete was assumed to be 0.2.

3.1.2. Steel reinforcement

The steel was assumed to be an elastic–perfectly plastic material and identical in tension and compression as shown in Fig. 6.

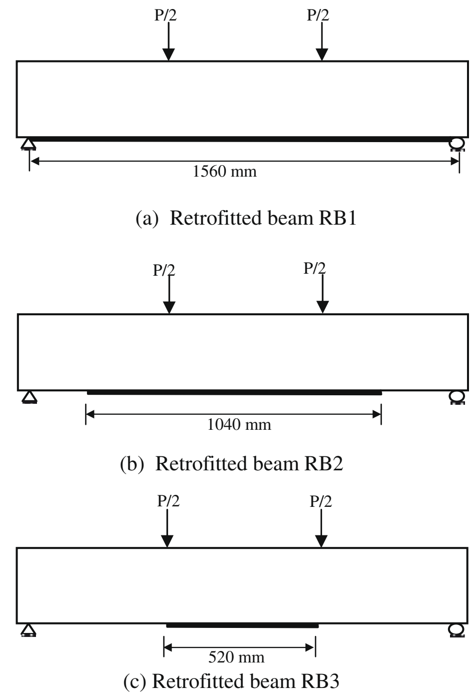

Fig. 2. Length of CFRP laminates in test series RB1, RB2 and RB3.

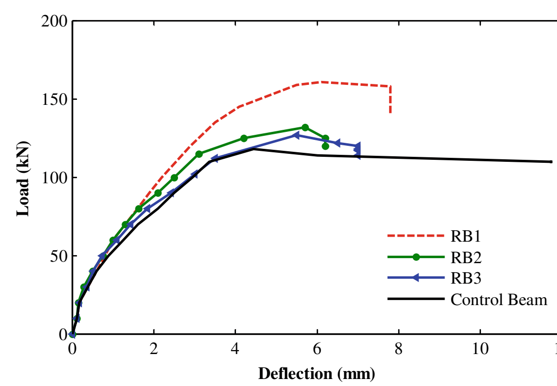

Fig. 3. Load versus mid-span deflection for un-strengthened and strengthened beams.

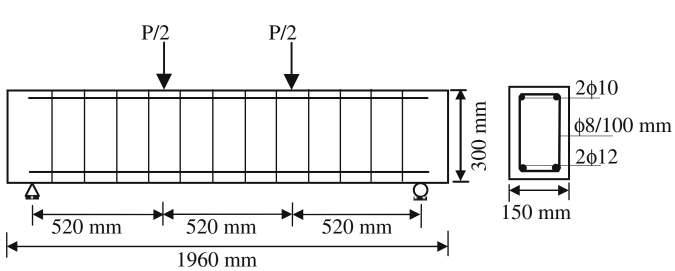

Fig. 1. Geometry, reinforcement and load of the tested beams.