Smeared crack models of RC beams with externally bonded CFRP plates

P. Fanning, O. Kelly

Abstract Concrete, a complex mix of variously sized aggregates, sand, water, additives and cement binder, is one of the more common engineering materials used for the design and construction of structures and bridges. Concrete is characterized by good compressive strength properties but it demands the use of internal reinforcement, generally in the form of round steel bars, to carry tensile stresses. The strength of the resulting element is dependent on the amount and distribution of steel reinforcement included during construction. It is not however possible to include additional internal reinforcement after construction in the event of the applied loading being increased and therefore consideration must be given to strengthening the structure externally, demolishing it, or confining it to specific usage, for example a maximum weight restriction on a bridge. In circumstances where restricted usage is not practicable structural strengthening is generally more favourable than demolition and replacement. Research in the area of strengthening of existing bridge beams is currently topical in the European Union given recent EU directives, aimed at encouraging free trade and movement of goods and services, which require all bridges to take 40 tonne vehicles.

This paper describes the numerical modelling procedures employed, using smeared crack models available in ANSYS V5.4, to capture the load-deformation response and modes of failure, of reinforced concrete beams which have been strengthened, using carbon fibre reinforced polymer (CFRP) composite material plates. Experimental verifications of these simulations have also been performed and are discussed in the present paper.

1 Introduction

Strengthening of beams in flexure can be achieved by bonding or anchoring plates, traditionally steel plates, to the tensile face of a reinforced concrete member. The shear strength can be enhanced with plates bonded or anchored to the beam sides in the shear spans. Steel plates, although effective in providing enhanced strength, are difficult to manipulate on site, cannot be delivered to site in lengths greater than about six metres and on exposed concrete surfaces they are particularly prone to corrosion problems.

In recent years significant effort has been dedicated to the use of fibre reinforced polymer composite plates in strengthening applications (Meier et al., 1992; Meier, 1996; Emmons et al., 1998). The plate bonding techniques and the strengthened solutions associated with CFRP composite material plates are recognised, in certain circumstances, to be superior to those achieved using steel. In particular the CFRP composite material combines a high specific strength with a low specific density and is available in continuous rolls of almost any length. The greater ease of manipulation and handling results in reduced temporary works requirements, shorter project durations and lower labour costs. Transportation difficulties, which generally restrict the length of steel plates to no more than 6 m, and as a result demand that complicated lap joints be fabricated for even medium length beams, are overcome. Both the fibre and matrix of the CFRP composite material are inert, consequently unlike steel plate bonded systems corrosion does not pose a threat to the long term durability of the strengthened system.

Although this material has recently been used successfully in strengthening schemes in Europe (Midwinter, 1997; Neubauer and Rostary, 1997; Barboni et al., 1997) and worldwide (Nanni, 1995; Sen and Shahawy, 1994) there are many design issues associated with CFRP materials used in plate bonding schemes which remain to be resolved. Hutchinson and Rahimi (1993) and Saadatmanesh and Ehsani (1991) have shown that the CFRP post-strengthening can lead to a catastrophic brittle failure in the form of plate peeling. This is consistent with the work of Quantrill et al. (1995) and Garden et al. (1993). Plate peeling can be overcome by using external anchorage (Spadea et al., 1998), which extend into the compression zone of the beams. Unanchored plate solutions remain attractive as anchorage adds significantly to both project costs and complexity. Raoof and Zhang (1997) and Zhang et al. (1995) have sought to predict upper and lower bounds for plate peel off loads based on upper and lower
bound flexural crack densities proposed by Watstein and Parsons (1943). Although attractive the range within which plate peel-off is predicted to occur is large and the lower bound solution can be very conservative. Non-linear models for reinforced concrete systems have been evaluated by Kotsovos and Spiliopoulos (1998) and Hofsetter and Mang (1996). Smear crack models are available in most commercial finite element software. The suitability of these models in capturing the load-deflection response and modes of failure of externally strengthened beams was examined by monotonically loading ten instrumented reinforced concrete beams to failure, eight of which were strengthened in flexure with different plate configurations, under laboratory conditions and comparing numerical and test data.

2 Laboratory programme: Test beams and strengthening system

The mode of failure associated with plate-peel off in shown in Fig. 1. Typically the plate separates from the reinforced concrete beam at its termination point and unzips along its bonded length bringing with it the concrete beneath the tension reinforcement. Three metre long beams, 155 mm wide by 240 mm deep, were selected for the test programme to negate any effects of scale in this sensitive region.

A cross section through the beam, Fig. 2, illustrates the internal reinforcement. Three 12 mm diameter steel bars are included in the tension zone with two 12 mm steel bars as compression steel. Ten shear links, formed from 6 mm mild steel bars, are provided at 125 mm centres in the shear spans. Two unplated beams formed a control set, (B1 and B2), and four different plate lengths were used in subsequent pairs of beams, (B3 to B10). Each of the beams was simply supported with a clear span of 2.8 m and loaded symmetrically and monotonically, under displacement control, in four point bending, with point loads 0.3 m either side of the mid-span location, to failure.

The Sika Carbdur System, comprising CFRP composite material plates and a two-part epoxy resin adhesive, provided the external reinforcement on the strengthened beams. The Carbdur S plates, with a longitudinal modulus of elasticity of 155 GPa, a lower bound tensile strength of 2400 MPa with an associated strain to failure of 1.4%, were used in these tests. These plates, consisting of unidirectional carbon fibres embedded in an epoxy matrix, were 1.2 mm thick, 120 mm wide and manufactured by a pultrusion process. Prior to application of the plates the beam surfaces were ground to expose the aggregate and the Carbdur S plates were degreased to provide a clean bond surface. Sikadur-30 adhesive, the thickness of which was controlled by 3 mm diameter granular beads, was applied to the strengthening plates and the ground concrete surfaces and a period of three days was allowed elapse prior to testing.

The test programme was designed to assess the strength enhancement offered by anchored plates and also the relative reduction in ultimate load carrying capacity of these beams due to plate peel-off. Beams B3 and B4 had plates bonded along their full length. These plates were thus continuous under the support locations with the support reactions anchoring the plates to prevent plate peel-off. Beams B5 and B6 had 2.03 m long plates centred on the mid-span resulting in 65% of the shear spans being plated, and beams B7 and B8 had 1.876 m plates, (resulting in 58% of the shear spans being plated), similarly arranged. The shortest applied were 1.7 m long, and placed centrally on beams B9 and B10.

3 Experimental test results

The mid-span deflections for the 3.0 m beams, control and strengthened, are plotted in Fig. 3 with the failure loads and mode of failure documented in Table 1. The unstrengthened beams, B1 and B2, failed at 68.3 and 67.9 kN respectively.

Fig. 1. Plate peel-off

Fig. 2. Reinforcement details for test beams