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# Punching behaviour of column-slab connection strengthened with non-prestressed or prestressed FRP plates

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ABSTRACT

Fracture resistance of concrete column-slab connections, with low reinforcement ratios and strengthened by carbon fibre reinforced polymer (CFRP) plates, is often governed by debonding failure modes. In this paper, a three-dimensional finite element model is conducted to predict the punching behaviour of concrete column-slab connections which incorporate bonded non-prestressed and prestressed CFRP plates to the tension surface of the slab. The adopted interfacial behaviour, between the FRP materials and concrete, takes into account the effect of mixed mode behaviour on the displacement fields at the critical diagonal cracks (CDC). Results are then presented in terms of the ultimate load capacities, load-strain relationships in the FRP and internal steel reinforcement, load-interfacial stress relationships, and stress distributions in the FRP reinforcement. It can be stated that by adopting the mixed mode behaviour at the interface between the fibres and concrete, the onset of debonding is effectively captured by the FE analysis, which is very difficult in a laboratory test.

## 1. Introduction

Existing concrete flat slabs may need to be strengthened due to insufficient punching shear capacity, as a result of changing the building use, new openings in a slab, design or construction errors. Several investigations have been conducted on strengthening the column-slab connection by different techniques to delay or prevent punching shear failure. These techniques, so far, can be categorized into two main approaches; direct shear strengthening and flexural shear strengthening. In the former approach, FRP composites are threaded through the thickness of the slab; in a similar method to the application of shear studs as a transverse reinforcement, to carry the diagonal tension forces [1,2]. However, in the second approach FRP laminates are bonded; as flexural reinforcement, to the tension surface of the slab [3–5]. This last one is easy to implement and efficient, especially for slabs with low-reinforcement ratios.

The punching behaviour of RC slabs strengthened with externally bonded reinforcement (EBR) during the new service life is relatively complicated because of the structural complexity and the progress of both loads and cracks. To attain the ultimate loading capacity of such strengthened slabs, the composite action has to be maintained. That is the EBR contributes to the flexural strength, which indirectly enhance the punching capacity of the strengthened slab [6]. Yet, the CDC induced interfacial debonding, is considered one of the most common failure modes responsible for loss of composite action of RC slabs strengthened with EBR. This mode of failure occurs; near the column area, when the shear capacity of the section is exceeded prior to the load level reaching the flexural strength. The development of such diagonal shear crack in flat slabs is associated with both horizontal and vertical openings [7]. Hence, the change in the displacement fields at the interface level; around the crack mouth, affects the bonding behaviour [8,9].

Two-main analysis approaches can be adopted in analysing FRP-RC composite sections; the classical analysis theories and the numerical analysis. The first approach, which is adopted in most of the available codes [10–13] for FRP strengthening applications, involves solutions of 'simple' design equations based on closed form solutions [14]. These formulae are usually accompanied with some provisions to control deflection and cracking. The applicability of this method is limited to certain geometries and loading configurations. Thus, it is not applicable to non-conventional slab design in some cases; for example, a slab strengthened with EBR or a slab with internal openings. An acceptable approach to overcome such shortcomings is to impose some modifications to the steel RC design practice for the design of RC slabs strengthened with EBR. The main assumptions adopted in this approach are; (a) no slip between the external FRP and the concrete substrate, (b)

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premature separation and shear failure of the FRP is not admissible and (c) tensile strength of the adhesive can be ignored (i.e. bond line is thin).

The finite element analysis (FEA) may, therefore, provide a more reliable and accurate method to predict the behaviour of slabs strengthened with EBR in both the serviceability and ultimate limit state. It enables accounting for different debonding failure modes if the bond interface is modelled properly. Generally, two approaches are available for modelling the bond interface in FRP-strengthened reinforced concrete structures. The first one is the meso-scale FEA, in which the FRP nodes are directly connected to the adjacent concrete nodes. In this approach a fixed angle crack model (FACM) is employed in conjunction with a very fine finite element mesh with element sizes being one order smaller than the thickness of the facture layer of concrete. This method was recently developed by Lu et al. [15] and was able to give accurate predictive results for the FRP-concrete bonded joints. In the second approach, which is more suitable for large structures and 3D FEA, a layer of interface elements between the FRP and the concrete is employed. In this case, the debonding is simulated as failure of the interface elements rather than a failure of the very fine finite element mesh adjacent to the FRP plate in the first approach.

Experimental investigation on column-slab connections, with low reinforcement ratio, and strengthened with non-prestressed or prestressed CFRP plates have been conducted, of which results have been published before by the authors [16]. In this study, a three-dimensional finite element model is developed to provide a further insight into the punching behaviour of such column-slab connections. The interfacial debonding behaviour between the FRP and concrete is modelled, considering the change in the displacement fields due to the development of the CDC. Results are presented herein in terms of the load deflection relationships, failure modes, ultimate load capacities, stresses in the steel reinforcements and CFRP plates, and interfacial slip and stress distributions. The numerical predictions are compared with test data and good agreement has been obtained.

## 2. Summary of experimental study

Five specimens were designed and fabricated to simulate interior column-slab connections. The test specimens were $1800 \times 1800 \times 150$ mm square slabs with a stub column, $250 \times 250 \times 142$ mm at the centre of the slab that was monolithically cast with the slab, as shown in Fig. 1. The characteristic compressive strength of concrete was 35 MPa. All specimens were reinforced with 8 No. 12 mm diameter bars in each direction giving a reinforcement ratio of 0.33%. The tensile test of the reinforcing bars indicated that the measured yield and the ultimate strengths were 579 and 650 MPa, respectively. Four slabs out of five were strengthened with non-prestressed and prestressed CFRP plates bonded to the tension surface of concrete substrate. The ultimate tensile strength and the modulus of elasticity of the plates in the fibre direction were 2970 and 172,000 MPa. For the prestressed slabs, different values of pre-stressing forces were introduced to the CFRP plates to examine the effect of the prestressing ratio on the punching behaviour of such connections. The prestressing forces were limited by the creep rupture strength of the FRP plate which is 55% of its ultimate tensile strength; according to ACI Committee 440 [11]. The prestressing device shown in Fig. 2 was developed to apply the prestressing forces. To simulate precracked slabs due to ageing or overloading conditions a notch of 12 mm depth was introduced for all specimens at the critical section; two times the slab depth *d* from the column face, according to Eurocode 2 [17]. Table 1 presents a brief description of test specimens, while the subscript numbers denote the effective residual prestressing ratio.

All specimens were loaded concentrically, and were simply supported along all four edges with the corner free to lift, as shown in Fig. 1. Equally spaced linear variable deferential transformers (LVDTs) were placed on the top surface of one eighth of the specimen to measure the deflection profiles of the specimen during the application of load, as shown in Fig. 3. High-precision LVDTs were used to measure the crack mouth opening displacement during the test. Strain gauges were mounted along each direction to measure the strains of the FRP sheets and steel bars to provide additional strain information. Further details of the experimental programme can be found in Ref. [16].

## 3. Finite element analysis

The specimens under investigation represent the region of negative bending around an interior column-slab connection up to the points of contraflexure. This loading condition has two planes of symmetry; XZ and YZ, as shown in Fig. 2. Accordingly, only a quarter of the slab was modelled.

In this study, finite element software [18] was used to model the concrete slabs, adopting a discrete modelling approach. A typical slab model is shown in Fig. 4. Eight-node isoparametric brick elements were adopted to model the concrete. To represent the internal steel reinforcement, two-node linear displacement truss element was used. Perfect bond between the grid reinforcement and the surrounding concrete material was assumed. This was considered reasonable because ribbed and bent bars were used in the tests. A conventional layered shell element was chosen to model the FRP reinforcement. This is based on the assumption that the loading configuration is most likely to produce in-plane stresses in FRP, so that the out-of-plane normal stress component is zero. Consequently, FRP could be modelled as an anisotropic homogeneous material under plane stress.

The FRP-concrete interface was modelled using a two-node connector element. The connectivity of this element could be simplified since it combines three springs working orthogonally. Two springs out of three were assumed to be working in the local first and second directions (the first and second shear direction), while the third one was applied in the out-of-plane direction (normal direction) between the concrete and FRP plates. The four-node rigid surface element was chosen to model the supports. Each node has three degrees of freedom. All elements were connected to a reference point where the boundary conditions were applied.

To verify the model, mesh sensitivity was investigated against both the reference slab ($RS_0$) and the strengthened slab ($RS\text{-}F_0$) in order to cover the range of reinforcement ratio. Different mesh sizes of 75 mm, 50 mm, 30 mm and 25 mm were considered in the sensitivity analysis so that the tension stiffening overshadows softening of concrete. The mesh was said to be converged when an increase in the mesh density had a negligible effect on the results obtained, while the load-midspan deflection response was the reference parameter in determining the appropriate mesh size. The convergence study implied that the 50 mm mesh size converged to both the 30 mm and 25 mm mesh sizes. On the other hand the 75 mm mesh appeared to have unstable behaviour after cracking and to experience numerical problems. Therefore, it was decided to adopt the 50 mm average mesh size for the analysis.

### 3.1. Material modelling

#### 3.1.1. Concrete

A damaged plasticity model was used for the analysis of the concrete slabs [18]. The model makes use of the yield function of Lubliner et al. [19], with the modifications proposed by Lee and Fenves [20] to account for different evolution of strength under tension and compression. Structural aspects of the rebar-concrete interaction, like bond-slip and dowel action, are indirectly considered by introducing some "tension stiffening" into the concrete softening behaviour to simulate load transfer across cracks through the rebar.

Fig. 5 shows the uniaxial stress-strain relationship of concrete adopted in this study. For concrete under compression, the material model specified in Eurocode 2, Part 1.1 [17], was modified to include the initial linear elastic response up to 40% of the mean compressive