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NUMERICAL MODELLING OF THE SHEAR STRENGTH BETWEEN CONCRETE LAYERS

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Summary. Some of the most used strengthening techniques for reinforced concrete structures include the increase of existing cross-sections. The monolithic behaviour of the strengthened elements depends basically on the interface between the substrate and the new concrete layer. A complete numerical model capable of dealing with the composite response of these RC elements is still missing. Such a model mainly depends on the interface behaviour, namely shear friction and dowel action. A numerical model is calibrated using experimental data carried out to assess the longitudinal shear strength between two concrete layers with different values of added reinforcement. A parametric study is developed to identify the influence of the following parameters on the interface behaviour: elastic stiffness; cohesion; internal friction angle; fracture energy; dilatancy; steel constitutive law and the bond-slip relation between connectors and concrete. The role of each parameter is clarified and the most relevant conclusions are presented.

1 INTRODUCTION

Strengthening of reinforced concrete structures is frequently achieved by increasing the area of existing cross-sections. Several experimental studies were already carried out to identify and analyse the most important parameters for the bond strength of old-to-new concrete interfaces[1]. In spite of the wide range of experimental work developed, a complete numerical model for the composite response of these RC elements is still missing. In this work, a numerical model is introduced to deal with several parameters controlling the interface behaviour for push-off tests.

2 NUMERICAL MODEL

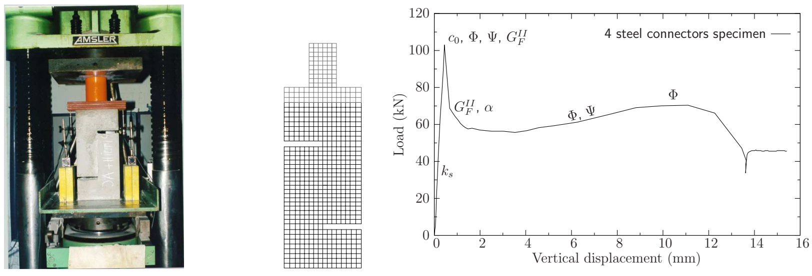
The tested push-off specimens are composed by two “L” shaped halves enveloped by a $254 \times 546 \times 127$ mm^3 prism – see Fig 1(a). The interface surface was prepared by sandblasting and crossed by a variable number of S400 steel connectors, ranging from 0 to 6. A detailed description of the experimental set-up can be found in Júlio[1].

Concrete behaviour is assumed as linear elastic under tensile stresses and elastic-perfectly plastic under compression, limited by an average compressive strength[1] of 43 MPa. Concrete Young’s modulus and remaining material properties which were not experimentally obtained, are computed according to Eurocode 2[2] guidelines. For the interface, a plasticity model with a Mohr-Coulomb friction law yield surface[3] is chosen. The steel connectors follow a multilinear constitutive law adjusted to the experimental tests and the MC 90[4] proposal for the bond stress-slip relation is applied.

The numerical model is composed by plane stress bilinear finite elements for concrete. At the structural joint, four-node zero-thickness interface elements are used to connect the bilinear elements from each side of the joint. Crossing the interface, linear truss elements are adopted to simulate the connectors. These were subsequently connected to the concrete bilinear elements using zero-thickness finite elements with the above mentioned steel-concrete bond law. Loading, boundary conditions and mesh (see Fig. 1(b)) were defined after a preliminary study.

3 PARAMETRIC STUDY

Following a preliminary study, each material parameter role at each stage could be put into evidence (see Fig 1(c)), and a strategy was settled to evaluate them: *i*) the elastic stiffness, k_s, is first evaluated to match the experimentally observed value; *ii*) the internal friction angle, ϕ, ensures that the experimental average residual stress is obtained. A sensitivity analysis with respect to the dilatancy, ψ, follows, in order to approximate the experimental hardening modulus; next, once these first parameters are evaluated, *iii*) peak load is used to adjust both the cohesion, c_0, and mode-II fracture energy, G_F^{II}; *iv*) finally, the softening part of the diagram and minimum value after peak load are approximated by varying G_F^{II} and the bond-slip shape parameter α defined in MC 90[4].



(a) (b) (c)

Figure 1: (a) Experimental set-up; (b) adopted mesh; and (c) most relevant parameters at each stage of a load vs. displacement curve for a push-off specimen.

From the parametric study undertaken only the main observations are hereafter mentioned.

Approximation of the experimental stiffness leads to the adoption of a value for the shear stiffness of 18 N/mm^3 for interface elements inserted along the structural joint.

According to the material models adopted, failure of the push-off specimen is due to both the yielding of the steel connectors and to the horizontal restraining force at the top and bottom steel plates. A value of $\tan(\phi) = 0.95$ provides the best approximation for the maximum residual load for all experimental curves. For constant α, the rate at which the residual load is reached is controlled exclusively by the dilatancy. A value of $\tan(\psi) = 0.180$ gives rise to a good approach of experimental hardening – see Fig. 2(a).

Cohesion is defined by studying the results regarding specimens without steel connectors. However, the peak load seems to depend on both cohesion and fracture energy. Therefore, stages *iii*) and *iv*) are interrelated. G_F^{II} is settled equal to 3.0 N/mm in order to reproduce the experimental softening – Fig. 2(b), thus allowing to define c_0 as 3.8 N/mm^2.

The parameter responsible for the bond stress-slip shape α has major influence on how fast steel connectors start to be tensioned, rather than on the rate as in the case of dilatancy – see Figs. 2(a) and 2(c). The decreasing of α towards zero implies constant stress along the steel connector and faster stress increase. As a consequence, the steel connectors are tensioned earlier and hardening is shifted towards the left of the diagram, decreasing the concavity around the minimum value.

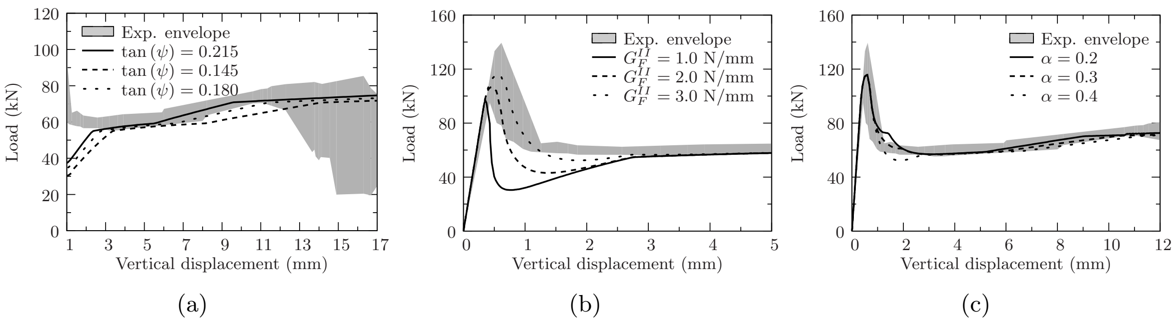


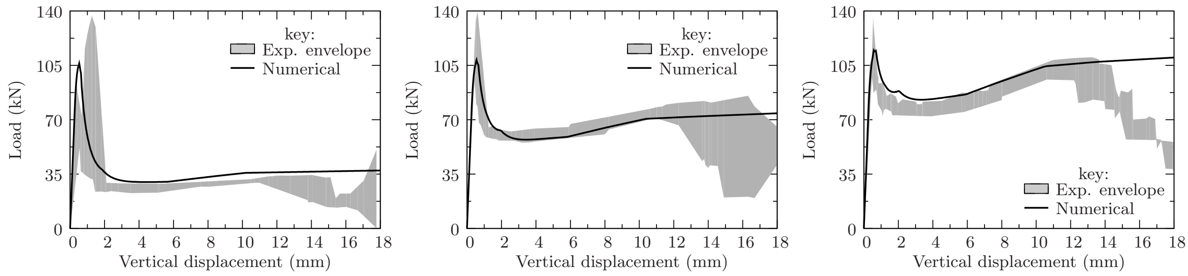
Figure 2: Load vs. displacement curves for 4 steel connectors and varying: (a) dilatancy; (b) fracture energy; and (c) α with $G_F^{II} = 3$ N/mm.

4 RESULTS

According to the parametric study overview in Section 3, the following parameters were adopted: $k_s = 18$ N/mm^3; $\tan(\phi) = 0.95$; $\tan(\psi) = 0.18$; $c_0 = 3.8$ N/mm^2; $G_F^{II} = 3.0$ N/mm; and $\alpha = 0.3$. Fig. 3 presents the load vs. displacement curves for all analysed cases. A good agreement is found for all experimental data concerning all studied situations. Moreover, the debonding stress and the maximum average stress after peak load present differences smaller than 4% and 7%,respectively, between numerical and experimental results.

5 CONCLUSIONS

Although there is still some uncertainty with respect to the present analysis, as general conclusions it was shown that it is possible to: i) fairly simulate all experimental situations



(a) (b) (c)

Figure 3: Load vs. displacement curves for (a) 2 steel connectors; (b) 4 steel connectors; and (c) 6 steel connectors.

and ii) to assess the role of each parameter in the global structural response. In particular, it should be stressed that: iii) the correct modelling of the initial linear elastic branch of the experimental tests is not possible, unless an initial stiffness parameter significantly below the one corresponding to a monolithic behaviour is adopted. Moreover, iv) the internal friction angle is found crucial for the residual strength of the specimen, which is certainly related to the sandblast treatment of the interface[1]; v) the dilatancy defines the rate at which the steel connectors are tensioned, thus the shape of the hardening branch of the experimental data; vi) peak load is controlled by means of shear strength and fracture energy; and, finally, vii) the softening part of the diagram and minimum load depend on fracture energy and bond stress-slip shape.

The dowel action and crushing of concrete were not simulated but probably these phenomena must also be implemented to better simulate the behaviour between peak and minimum load. Last but not least, it should also be stressed that the discretization of the steel connectors could lead to an increased residual strength, but it would also lead to shear failure, instead of the tensile failure which was the experimentally observed mechanism[1]. This is a consequence of the limitations inherent to the truss elements adopted. Further experimental and numerical research is being carried out addressing this fundamental issue.

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