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To cite this version:

Ludovic Jason, Luc Davenne. On the influence of the steel-concrete bond model for the simulation of reinforced concrete structures using damage mechanics. The Third International Conference on Damage Mechanics (ICDM 3), Jul 2018, Shanghai, China. cea-02339323

HAL Id: cea-02339323
https://hal-cea.archives-ouvertes.fr/cea-02339323
Submitted on 14 Dec 2019

ON THE INFLUENCE OF THE STEEL-CONCRETE BOND MODEL FOR THE SIMULATION OF REINFORCED CONCRETE STRUCTURES USING DAMAGE MECHANICS

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ABSTRACT: When dealing with the simulation of reinforced concrete structures, the bond model between steel and concrete can become a key point if crack properties are studied. The interface between both materials is indeed partly responsible for stress transfer and consequently for the crack spacing and openings. In the context of finite element simulations using damage mechanics for concrete, the influence of the relation between steel and concrete is evaluated by comparing two solutions: a perfect relation (same displacement at the interface) and a recently developed bond model (sliding allowed). The approaches are compared on a reinforced concrete tie, a bending beam and a shearing wall. The interest of including a fine description of the steel-concrete bond rather than a simple perfect relation between materials, regarding the simulation of local properties (crack openings especially) depends on the type of applications (loadings) and on the expected crack pattern (and/or distribution of steel).

Key words: concrete, steel-concrete bond, structures, damage, simulation

1. Introduction

Steel is widely used in civil engineering applications to strengthen concrete in tension. These so-called reinforced concrete structures, which present a more ductile behavior compared to plain concrete, may nevertheless be subjected to cracking. In this case, when a crack initiates, stresses in concrete drop to zero and the loading is totally supported by the reinforcements. They are then responsible for stress transfer around the crack from steel to concrete. This progressive redistribution, which can be easily demonstrated in the case of a reinforced concrete tie, is directly influenced by the bond properties [1]. That is why the influence of the steel-concrete bond has to be carefully studied, especially when the crack properties, which are directly related to this stress distribution, play a key role in the structural functions (failure mode, tightness…).

Experimentally, steel-concrete bond is generally described following three different steps [2]: a perfect "chemical" bond (no slip), then a gradual degradation of concrete around the steel ribs, followed by crack propagation (associated with a steel-concrete slip), and finally a total degradation of the interface with only a residual friction.

Finite element simulations remain the most convenient way to represent the expected behavior of a reinforced concrete structure. In this context, many solutions exist to model the steel – concrete bond: spring elements [3], zero thickness joint elements [4] or enriched description [5] among others. If these solutions propose a fine description of the bond mechanisms, they also require an increase in computational cost which is rarely compatible with structural simulations (increase of the number of degrees of freedom for enriched solutions, explicit meshing of the steel – concrete interface for joint elements…). Especially, they do not allow the use of 1D truss elements for steel, which is the most common way to model reinforcements in structural applications.

In this context, simpler models are then needed. The simplest solution is based on a strong hypothesis: steel, modeled using truss elements, and concrete keep the same displacements at the interface. This equality can be imposed using for example additional cinematic relations. If this approach is easily applicable, and frequently applied, at low additional computational cost, it is only able to represent the first step of the bond mechanism (perfect chemical bond).

To investigate the influence of this simplification, computations using this so-called "perfect relation" will be compared to simulations including a newly developed bond model [6]. Also based on a 1D truss element description of the reinforcement, it introduces the sliding between steel and concrete and enables to describe the complete evolution of the interface through a bond stress – bond slip law [7].

After a brief description of the "perfect relation" hypothesis and of the proposed bond model, three applications will be considered: a reinforced concrete tie [6], a four point bending beam [8] and a shearing wall [9]. In every case, the limits, if any, of the perfect relation hypothesis will be discussed.

2. Perfect relation and bond model

In this part, the perfect relation hypothesis and the newly developed bond model are going to be briefly presented. In the proposed simulations, concrete and steel are respectively meshed using 3D and 1D truss elements (fig. 1). The bond model (or the perfect relation) is then used to relate both materials, using either additional cinematic relations or a specific finite element.

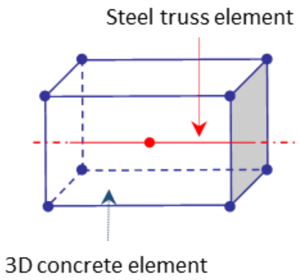

Fig.1 General presentation of the simulation using steel 1D truss and concrete 3D concrete elements.

2.1. Perfect relation

In the case of a perfect relation between steel and concrete, additional cinematic relations are used to impose the same displacements between steel nodes and surrounding concrete. The displacement u_i of the steel node i located at x_i simply writes:

$$u_i = \sum_j u_j N_j(x_i) \tag{1}$$

where u_j are the nodal displacements of the concrete element in which i is located and N_j is the associated interpolation function.

2.2. *Steel-concrete bond model*

To take into account the interfacial behavior between steel and concrete in a more appropriate manner, a new interface element has been developed in [6], based on the previous work in [10]. It is a zero thickness four node element which relates each steel truss element with an associated superimposed segment, perfectly bonded to the surrounding concrete (fig. 2).

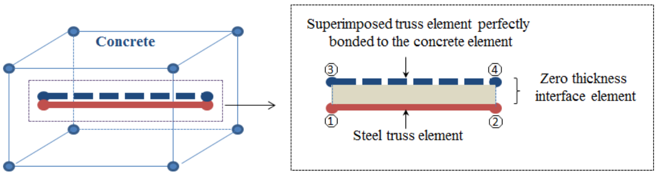

Fig.2 Principle of the zero thickness element for steel-concrete bond [6]

Each node of the interface element has three degrees of freedom (nodal displacements). A relation between the generalized slip in the local direct frame and the nodal displacements is written in the following form:

$$\{\delta(p)\} = \{\delta_t(p) \quad \delta_{n_1}(p) \quad \delta_{n_2}(p)\}^T = \overline{\overline{B}}(p)\{u\} \tag{2}$$

with

$$\overline{\overline{B}}(p) = \left[\overline{\overline{B}}_1(p) \quad \overline{\overline{B}}_2(p) \quad -\overline{\overline{B}}_1(p) \quad -\overline{\overline{B}}_2(p)\right] \tag{3}$$

and

$$\begin{aligned} \overline{\overline{B}}_1(p) &= 0.5(1-p)\overline{\overline{I}}_3 \\ \overline{\overline{B}}_2(p) &= 0.5(1+p)\overline{\overline{I}}_3 \end{aligned} \tag{4}$$

where $\overline{\overline{I}}_3 = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix}$ and $-1 \le p \le 1$

Constitutive laws are then defined between the bond stress $\{\sigma(p)\} = \begin{Bmatrix} \sigma_t(p) \\ \sigma_{n_1}(p) \\ \sigma_{n_2}(p) \end{Bmatrix}$ and the bond slip $\{\delta(p)\}$. In the tangential direction, the tangential stress σ_t is computed from the tangential slip:

$$\sigma_t(p) = f(\delta_t(p)) \tag{5}$$

In this contribution, the bond law follows the recommendations from [7].

In the normal directions, for sake of simplicity, a linear relation is assumed between the stresses σ_{n1} and σ_{n2} and the corresponding normal slips:

$$\begin{Bmatrix} \sigma_{n_1}(p) \\ \sigma_{n_2}(p) \end{Bmatrix} = k_n \begin{Bmatrix} \delta_{n_1}(p) \\ \delta_{n_2}(p) \end{Bmatrix} \tag{6}$$

The value of the normal stiffness is chosen high enough to be representative of a perfect bond ($k_n = 10^{15}$ Pa.m-1 in the following).

The nodal internal force vector in the local reference frame is then obtained by integration along the length of the element. Additional information can be found in [6].

3. Simulation of a reinforced concrete tie

In this section, the reinforced concrete tie, experimentally tested in [11], is simulated (fig.3). In this situation, loading is directly applied to steel at the ends of the reinforcement. The interface between both materials is thus directly involved in the stress transfer from steel to concrete.

In the simulation, the steel bar is represented using an elastic-plastic model with linear hardening. Concrete is modelled using a damage constitutive law developed in [12] and implemented in the finite element code Cast3M [13].

Fig. 4 illustrates the comparison between the bond model, the perfect relation and the experiment in terms of “global” results (steel stress as a function of the mean concrete strain). Both simulations are able to reproduce the experimental results, even if the perfect relation only simulates the envelope behavior (no partial unloading when a new crack initiates).

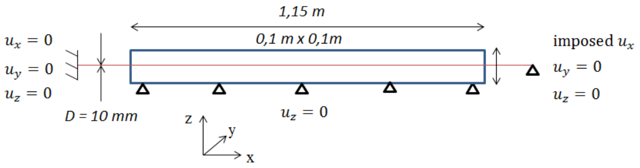

Fig.3 Geometry, boundary conditions and loading of the reinforced concrete tie [11].

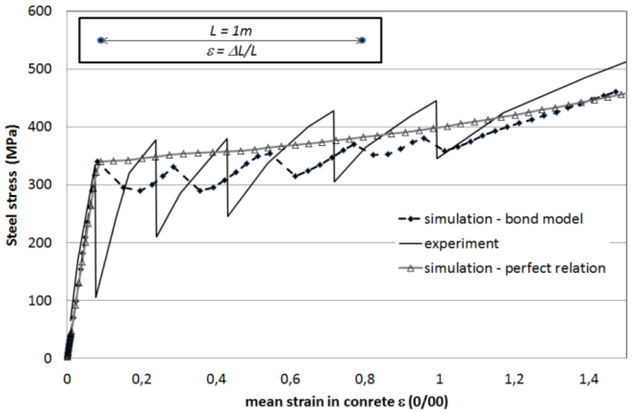

Fig.4. Evolution of the applied stress at the end of the steel reinforcement as a function of the mean strain in concrete in the reinforced concrete tie

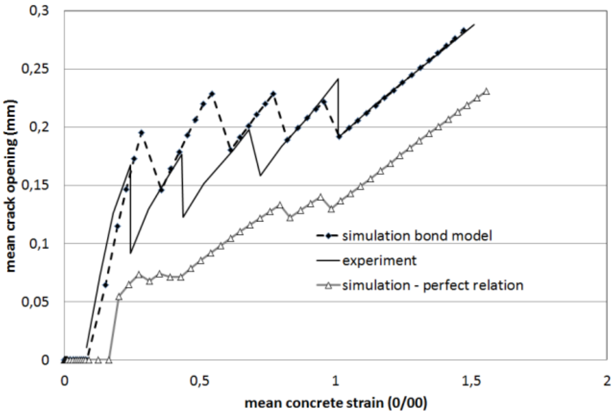

Fig. 5. Evolution of the mean crack opening as a function of the mean concrete strain

Fig.5 presents the evolution of the mean crack opening as a function of the mean concrete strain. In the case of this local quantity (crack opening), the perfect relation is unable to reproduce the experimental results, contrary to the bond model. For a given concrete mean strain, the perfect relation simulates a larger number of cracks and consequently a smaller mean crack opening. This is due to the transfer length, along which stress is transferred from steel to concrete (Fig. 6). It is significantly smaller in the case of the perfect relation, entailing a potential larger number of cracks [10]. This difference in transfer length is particularly significant in this case, as the differential slip around each crack reaches value up to 200 µm at the end of the loading. This value corresponds to a heavy degradation of the interface properties, which cannot be reproduced by the perfect relation.

The bond model, through a more representative transfer length, also enables the results to be less influenced by the initial distribution of the concrete material properties. For example, Fig. 7 presents three different initial distributions of the damage threshold used for three different simulations. Table 1 illustrates the results in terms of crack spacing, using the bond model and the perfect relation. In every case, using the bond model results in a constant number of cracks, in agreement with the experimental one. On the contrary, with the perfect relation, the number of simulated cracks, ranging from 6 to 8, is dependent on the initial distribution of the material properties.

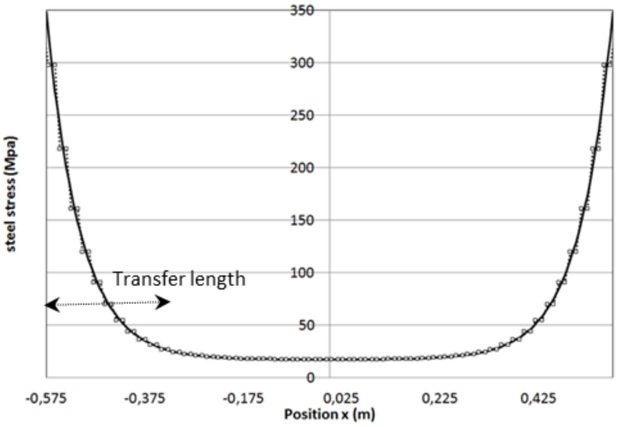

Fig.6 Distribution of the steel stress along the tie using the steel-concrete bond model.

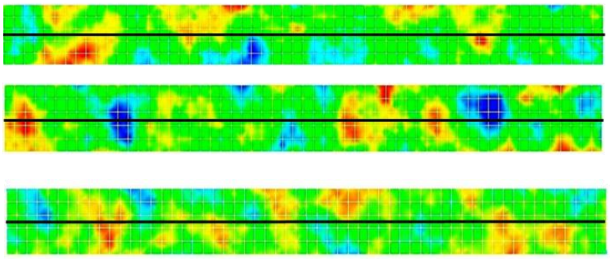

Fig. 7 Initial distribution of the damage threshold along the reinforced tie (cross section near the reinforcement). Red values correspond to the maximum ones.

Table 1. Crack spacing using the different distributions of the damage threshold (end of the loading)

	Bond model		Perfect relation	
	Number of cracks	Mean crack opening (mm)	Number of cracks	Mean crack opening (mm)
First distribution	5	0.283	6	0.219
Second distribution	5	0.283	8	0.164
Third distribution	5	0.283	7	0.209

As a conclusion, in a case where the interface is directly involved in the transfer of the loading and with a significant slip, considering a perfect relation for the bond between steel and concrete is not sufficient to capture appropriate local results (crack properties).

4. Simulation of a bending beam

In this section, the four-point bending beam, experimentally tested in [8], is simulated (Fig. 8). The structure is instrumented using digital image correlation on one lateral face in order to follow the initiation and propagation of the cracks during the loading (Fig. 9).

In this structure, even if the loading creates tensile stresses which are responsible for crack initiation inside concrete, the situation is different from the previous case. First, the loading is directly applied to concrete. Then, the expected slip is much smaller than the one obtained in the reinforced concrete tie.

For the simulations, the elastic-plastic and damage constitutive laws are once again chosen for steel and concrete respectively, with adapted parameters to reproduce the material properties (tensile and compressive strengths for example).

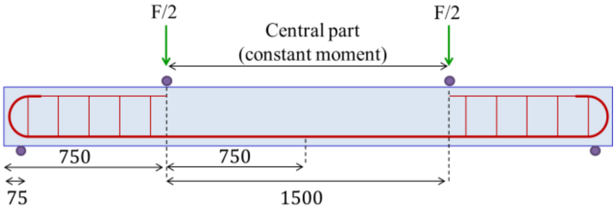

Fig.8 Four point bending beam [8]

Fig. 9 Experimental device with digital image correlation

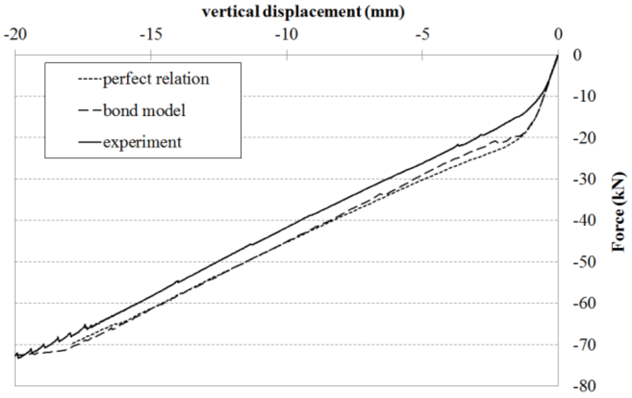

Fig. 10 Force – deflection curve for the four-point bending beam

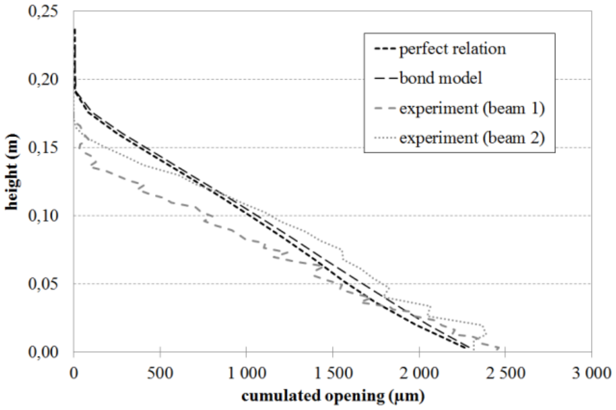

Fig. 11 Evolution of the total crack opening as a function of the height. End of loading (stabilized cracking phase).

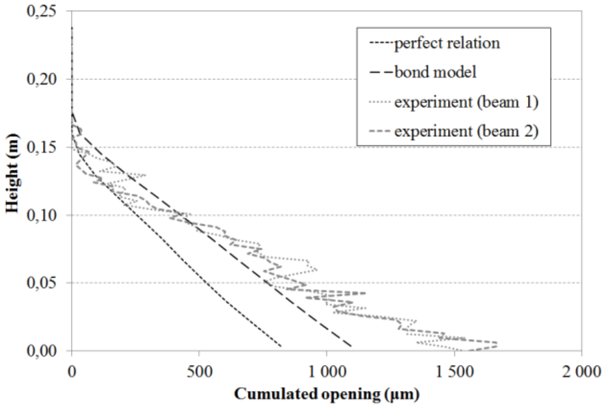

Fig. 12 Evolution of the total crack opening as a function of the height. Active cracking phase.

Fig. 10 provides the load-deflection curve for the two simulations and a comparison with the experiment. As for the reinforced concrete tie, both simulations are able to reproduce the experimental response.

Fig. 11 illustrates the evolution of the total crack opening in the height of the beam (sum of each individual crack opening obtained by digital image correlation at a given height) at the end of the loading. At this step, the cracks are stabilized, which means that no further crack appears until failure. In this case, the perfect relation and the bond model give similar results, even for this local quantity, and are in agreement with the experimental values. This may be explained by the configuration of the test, with especially anchorages at the end of the reinforcements that prevent a significant slip to occur. The maximum value of the simulated slip is equal to 50 µm with the bond model. It is not sufficient to entail a significant degradation of the interface and thus a significant difference compared to the perfect relation. The degradation of the concrete around the interface has in this situation a greater impact than the degradation of the interface itself.

Some improvements can nevertheless be obtained during the active cracking phase (beginning of the loading). In this situation, the bond model provides better results, still in agreement with the experiment.

As a conclusion, in the case where loading is applied to concrete, steel reinforcements are anchored and cracks are fully localized, the perfect relation gives acceptable results as far as global behavior and stabilized cracking phase are concerned.

4. Simulation of a shearing wall

In this section, the behavior of the shearing wall experimentally tested in the French national program CEOS.FR is simulated [14]. The dimensions and the reinforcement frame are given in Fig. 13 and Fig. 14 respectively. An increasing horizontal displacement is applied on the top of the wall. Four prestressed bars are used in the lower concrete beam to relate the structure to the ground (Fig. 15). For the simulation, an elastic behaviour is considered in the bottom and top heavily reinforced beams.

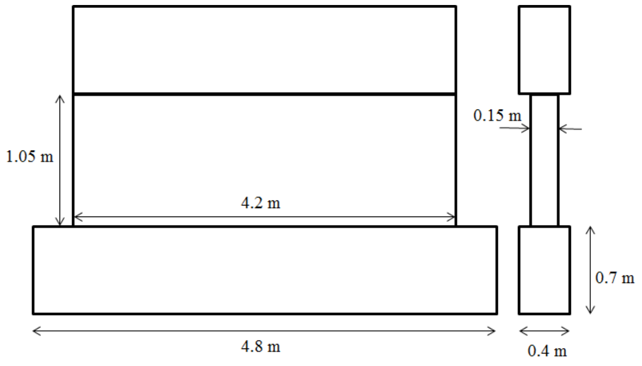

Fig.13 Dimensions of the shearing wall.

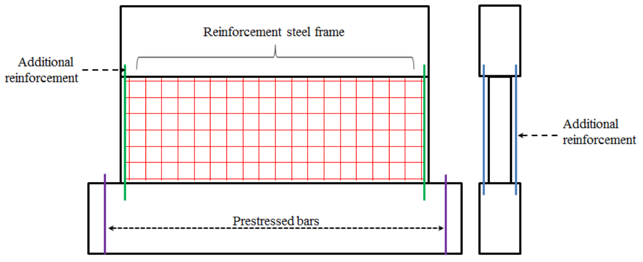

Fig. 14 Reinforcement frames for the shearing wall

Fig. 15 Principle of the experimental device for the loading and the boundary conditions

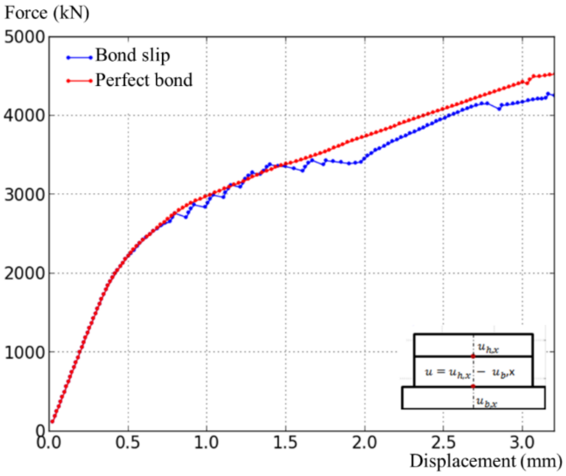

Fig. 16. Evolution of the force as a function of a relative horizontal displacement computed in the middle of the wall

For the steel reinforcements and the center part of the concrete wall, the same models as previously mentioned are considered.

Compared to the previous applications, in this case, loading is applied directly to concrete (different from the reinforced tie but similar to the bending beam), the reinforcements are well anchored (same as for the beam) but the expected crack pattern is a homogenized distribution rather than fully localized cracks (different from the beam).

Fig. 16 gives the evolution of the resulting force as a function of a relative horizontal displacement computed at the middle of the concrete wall. Once again for this type of global results, the two simulations give similar results (in agreement with the experiment, see [9] for further details).

Considering the simulated strain fields, a post-processing method for damage mechanics is used to obtain information on the crack properties [9].

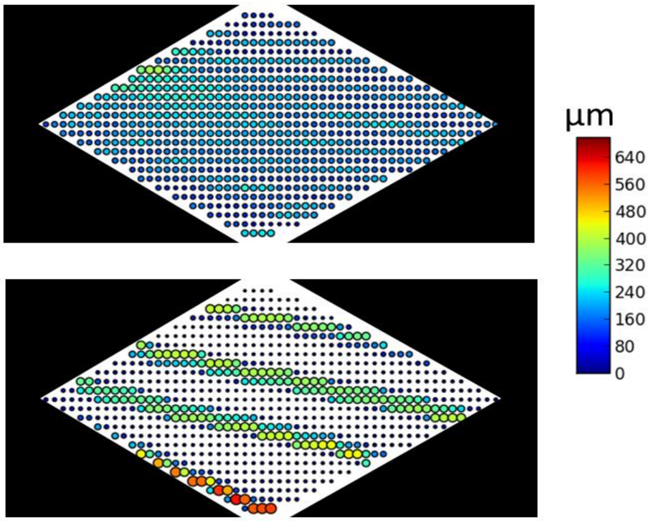

Fig. 17 Distribution of the “opening displacement”, representative of the crack path. Simulation using the perfect relation (top of the figure) and steel-concrete bond model (bottom of the figure). The simulated zone corresponds to the experimentally instrumented one (digital image correlation).

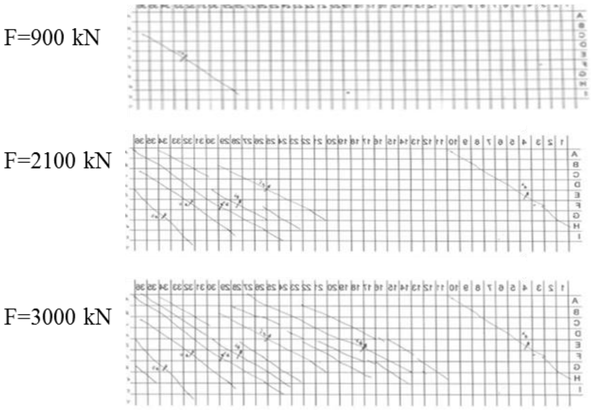

Fig. 18 Experimental crack patterns for different values of the resulitng applied force

The distributions of the so-called “opening displacement” are obtained (fig. 17). “Opening displacements” are computed from the relative concrete displacements and are representative of the crack path [9]. For the simulation using the perfect relation, the crack path is quasi-uniform with no particular localization of the mechanical degradation (homogenization). The distributed passive reinforcement frame holds back the displacements, due to a high stiffness at the interface (simulation of a permanent chemical bond) and prevents the localization of the cracks. On the contrary, the steel-concrete bond enables the degradation of the interface and a localization of the cracks, which in better agreement with the experiment (Fig. 18).

In this case, including the steel-concrete bond model in the simulation provides a better description of the mechanical degradation compared to a perfect relation.

5. Conclusions

Two solutions have been compared to take into account the steel-concrete bond in reinforced concrete structures in the context of finite element simulations using damage mechanics. Based on a perfect relation between the two materials (same displacements) or on the use of a newly developed bond model, they are both adapted to 1D truss element mesh for steel and do not suppose significant additional computational cost.

On applications for which loading is directly applied to steel and for which the interface is directly responsible for the stress transfer in concrete (reinforced concrete tie for example), including a representative steel-concrete bond model is necessary if a correct simulation of the crack properties needs to be achieved.

On applications for which loading is applied on concrete and for which crack is highly localized (reinforced bending beam for example), the perfect bond relation may be sufficient if the stabilized cracking phase is only studied. The perfect relation and the newly developed steel-concrete bond model give indeed almost similar results. This is due to a slight degradation of the bond, resulting in small slips at the interface.

Finally, if the loading is applied to concrete and if the crack pattern is more regular (shearing wall for example), the perfect relation is responsible for a quasi-uniform crack distribution with no particular localization. The distributed passive reinforcement frame holds back the displacements, due to a high stiffness at the interface. On the contrary, the developed steel-concrete bond model enables a better localization, similar to the experimental behavior.

As a conclusion, the interest of including a fine description of the steel-concrete bond rather than a simple perfect relation between materials will depend on the type of applications (loadings) and on the expected crack pattern (and/or distribution of steel). Whatever the case, solutions exist to correctly capture the crack properties, even for structural applications.

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