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A NUMERICAL SIMULATION OF BOND FOR PULL-OUT TESTS: THE DIRECT PROBLEM

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ABSTRACT

The objective of this paper presents the results of subtask dealing with the bond behaviour study of the reinforcement systems under monotonic loading pull-out tests. This numerical method is based on the slip and the bond stress distributions through the anchored length of the bar in the concrete block. The work refers, especially to the implementation of reinforcing bars and bond-slip models between steel and concrete in the developed finite element program. For the application of the proposed method, three analytical expressions of bond-slip relationship are selected. The obtained results are presented and commented with the fundamental characteristics of plain concrete and reinforced concrete members. The bond models in contribution with concrete and reinforcing steel provide a relativity good representation of bond-zone system responses.

Keywords: Bond-slip relationship, pull-out test, embedment length, RC members, bond stress distribution, perfect model, cohesive model

1. INTRODUCTION

A reinforced concrete material is a composite material made up of two components with unequal mechanical behaviour and physical features. In general, the external load is already applied to concrete and the reinforcing bars receive its part of the load only from the surrounding concrete by bond. In composite structures, the bond between different components of reinforced concrete member has a primordial role and its negligence conducted to poor structural response. In the past, this complex phenomenon has led engineers to take empirical formulas for the design of reinforced concrete structures. For these reasons, the incorporation of bond is considerably carried out in recent works.

To calibrate the bond behaviour, experimental studies widely known as pull-out or push-out tests were performed (Tastani, 2002). Numerous researchers have already investigated the anchorage behaviour of reinforcing bar and a number of analytical bond-slip models have been developed. Ones of the most widely used bond stress-slip relationship is proposed by Ciampi et al. (1982) and Eligehausen et al. (1983). More, many researchers have

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developed analytical and numerical models yielding to stress-slip response and bond stress distribution of reinforced concrete members (Tepfers 1980, Clément 1987, Alsiwat 1992, Focacci 2000, kwak 2001, Ben Romdhane 2002, Sezen 2003, Redzuan 2004, Benholzar 2005, Dominguez 2005, Khalfallah 2005). The common object is to have a reliable analysis that takes into account the effective behaviour of reinforced concrete structures.

Although significant improvements in the experimental field as well as in the theoretical aspects of steel-concrete interface behaviour have been achieved, most of them assume perfect bond. Particular attention is paid to stress variation in steel and to the force transfer between two materials. In this context, the gradual deterioration of the interface will be considered between the extremes cases (khalfallah 2005): the perfect bond and the no bond cases in which reinforcing bar can slip.

To accomplish the task, the interface is considered as a separate material with a proper behaviour and the constitutive modelling is derived using an energetic approach. For more realistic simulations, non-linear laws with several branches which mark different stages in the overall behaviour during a progressive loading were taken up.

In this way, there are two distinct kinds of analytical models based on force equilibrium principle of pull-out problem (1) the full perfect model and (2) the cohesive (contact) interface model. The principal difference between these two approaches is that in the first case a perfect interface is assumed; no slip between reinforcing bar and surrounding concrete is allowed and displacement field is assumed as continuous at the interface region. The displacement continuity requirement is abandoned when the interface is considered to be cohesive. In this study, these two models are considered.

To reach the goal, the finite element method is well known as a robust tool in modelling fields of structural mechanics is used in reinforced concrete analysis. Primordial parameters have a great influence on the response of reinforced concrete tensioned members such as: the progressive cracking, the tension stiffening effect, the non-linear material properties and the bond effect at the concrete-steel interface which can be used to improve the members' response.

This article is a partly task of a group research work on non-linear behaviour of reinforced concrete structures: modelling and analysis. The computational system has for base a finite element program which has already conceived for this object, such that to till now, the commercial programs don't take into account the behaviour of reinforced concrete structures. The work refers to the implementation of different representations of reinforcing bars and to quantify the contribution of bond between concrete and steel.

2. REINFORCEMENT FINITE ELEMENT MODELS

In finite element modelling of reinforced concrete structures, there are three different alternative representations of reinforcement: smeared, embedded and discrete reinforcement models. The first one is rarely used and therefore it depends on the nature of used structure. The discrete and embedded representations are be formulated and introduced in the developed program.

2.1 Discrete reinforcement representation

The discrete modelling of steel reinforcement is the first approach used in finite element analysis of reinforced concrete structures (Ngo and Scordelis 1967). The discrete representation of reinforcement uses one dimensional truss elements and it is the only way for accounting for bond slip and dowel action effects, Figure 1.

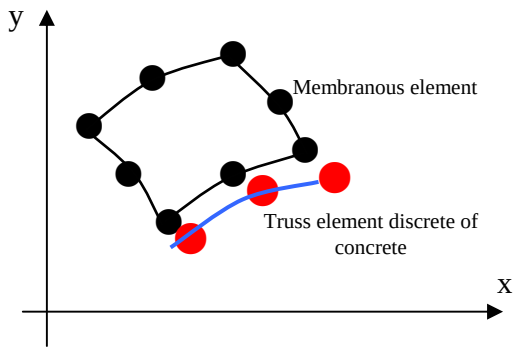


Figure 1. Discrete representation of steel bars

A significant advantage of discrete representation is that it can account for possible displacement of the reinforcement with respect to the surrounding concrete. The bond effects are usually related with this representation and the bond-link or cohesive models can be used to connect the steel and concrete nodes in order to considerate this effect. The main disadvantage is that the finite element mesh patterns are only restricted by the location of reinforcement and consequently the increase of the number of concrete elements and the degrees of freedom. In this way, Lagrange or Serindipity isoparametric concrete elements are used and a line three node truss elements is used to represent the steel and the compatibility between concrete and steel must be guaranteed.

2.2 Embedded reinforcement representation

In this representation, the reinforcement bar is considered as an axial member incorporated in the concrete element such that its displacements are consistent with membranous concrete elements and bond lost can be considered, Figure 2.

In this scope, many works have been presented different formulations for this model, Balakrisha [4], Allwood [2] and Philips [13]. Embedded models allow for an independent choose of concrete mesh. So the same number of nodes and degrees of freedom are used for both concrete and steel. The disadvantage of this procedure is that additional degrees of freedom increase the computational and numerical treatment.

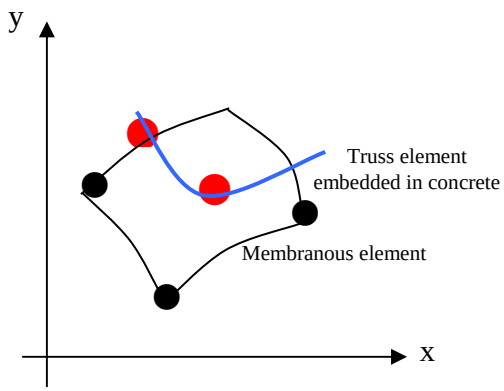


Figure 2. Embedded representation steel bars

3. FINITE ELEMENT MODELS FOR BOND

In this strategy, two distinct procedures of the bond lost were studied: (1) the bond-link model also known as shear-lag model and (2) the cohesive model as presented bellow. These elements are associated with the discrete reinforcement model, which has the advantage of representing different material properties more precisely. Afterwards other bond conditions at different nodes can be easily represented.

To describe the bond behaviour between concrete and steel, the vertical and horizontal relative displacement between concrete and steel in the local coordinates can be considered. The same type of isoparametric elements and it has, at the unloaded stage, no physical dimension in the transverse direction. It uses linear, quadratic or cubic interpolation functions corresponding to the number of nodes per element. In linear analysis, the vertical relative displacements are too small compared to the horizontal displacement. That means that the concrete and steel common nodes have the same degrees of freedom in local vertical axis while they have different degrees of freedom in horizontal axis.

3.1 Perfect bond analysis

This element can be conceived as two orthogonal springs (Ngo and Scordelis [16]) that connects the concrete nodes to its same of steel nodes, Figure 3. For each spring an appropriate stress-strain relation is defined. In linear analysis, it is assumed that the steel nodes are perfectly bonded to those of concrete and no slip is then assumed. This means that concrete and steel having the same nodes and per consequent the same degrees of freedom.

The stiffness of the steel elements is directly added to the corresponding degrees of freedom in the global stiffness matrix.

The bond effect is assumed as an interaction between reinforcing bars and surrounding concrete. When the change of stresses in concrete and steel occurs, the effect of bond begins and becomes more pronounced at the end anchorages of reinforcing bars and in the vicinity of cracks. To evaluate the bond stress-slip relationships, many studies have been published taken the perfect bond in consideration.

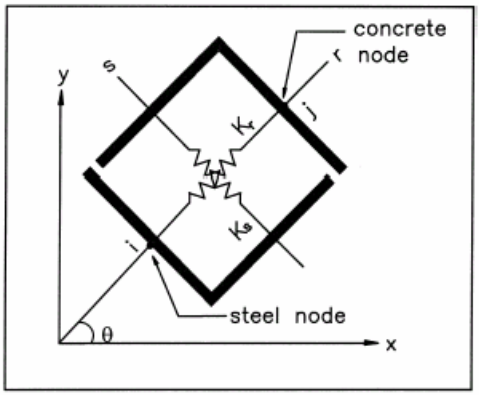


Figure 3. Bond-link model

3.2 Cohesive bond analysis

The behaviour of the concrete-steel interface must be described from stress-strain laws. Many constitutive relationships are presented in the literature. In this element, the behaviour of the interface continuum between concrete and steel is described by a proper law which considers the specific properties of the bond. The contact element provides a continuous connection between reinforcing bars and concrete (de Groot, [8]), (Keuser, [12]) who showed the bond-link element cannot represent adequately the stiffness of bond.

3.3 Analysis without bond

In this case, the stiffness matrices of the steel elements are computed in local axis at the nodes of non bond. The concrete element stiffness matrices are calculated in the global axis and they are transformed steel local axes at common nodes. In y-direction, concrete and steel have the same degree of freedom but having different degree of freedom in x-direction at common nodes.

4. LOCAL BOND SLIP RELATIONSHIP

In the following, it is assumed that the bond characteristics of reinforcing bar are analytically described by a local relationship of bond $\tau=\tau(s)$, in which τ is the shear stress acting on the contact surface between bars and concrete and s is the slip; that is the relative displacement between those of steel bar and concrete. Once the relation $\tau=\tau(s)$ is known and using equilibrium equation and compatibility relations, the second differential equation governing the slip can be defined as:

$$\frac{d^2 s}{dx^2}-\frac{\pi D}{E_s A_s}\tau(s)=0 \tag{1}$$

which D is the diameter, A_s is the cross sectional area, E_s is the Young's modulus of the reinforcing bars and s(x) is the slip between concrete and steel abscissa x.

Using the Eq. (1), important phenomena can be observed such as: the anchorage length evaluation, the determination of the tension stiffening effect and cracks spacing and opening. These problems can be solved once the boundary conditions of the specific problems are specified and this observation reinforces the importance of a consistent local bond-slip relationship.

4.1 Analytical expressions for bond-slip relationship

Two alternative basic hypothesises have been used in the past: in one bond stress is considered to be linear function (Ngo and Scordelis, [16]), while in the other it is considered to be a non-linear relationship between bond stress and slip. Analytical expressions for the local $\tau=\tau(s)$ relationships have already been developed. In this work, two $\tau=\tau(s)$ relationships are selected as examples to simulate the bond-slip behaviour between reinforcing bar and surrounding concrete.

a) The relationship established by Eligehausen is expressed by the following non-linear function as:

$$\tau(s) = \tau_0 (\frac{s}{s_0})^{0.4} \ , \ \ 0 \le s \le s_1 \tag{2a}$$

$$\tau(s) = \tau_0 \ , \ s_1 \le s \le s_2 \tag{2b}$$

$$\tau(s) = \tau_0 - (\tau_0 - \tau_f)\frac{s - s_2}{s_3 - s_2} \ , \ s_2 \le s \le s_3 \tag{2c}$$

$$\tau(s) = \tau_f \ , \ s \ge s_3 \tag{2d}$$

With s_1= 1 mm, s_2= 3 mm, s_3= 10.5 mm $\tau_f = 5.00MPa$ and $\tau_1 = 13.50MPa$, Figure 4.

b) The simple bi-linear bond stress-slip model is selected and the parameters of the model are derived from the experiment data corresponding to material features of each specimen, Figure 5.

To represent the bond-slip effect, the ultimate bond stress is considered the same for the used relationships.

$$\tau_1(s) = E_{b1} s \ \ s \le s_1 \tag{3a}$$

$$\tau_2(s) = \tau_1 + E_{b2}(s - s_1) \ s_1 \le s \le s_2 \tag{3b}$$

s_1= 2 mm, s_2= 10.5 mm, $\tau_f = 10.55MPa$ and $\tau_1 = 13.50MPa$

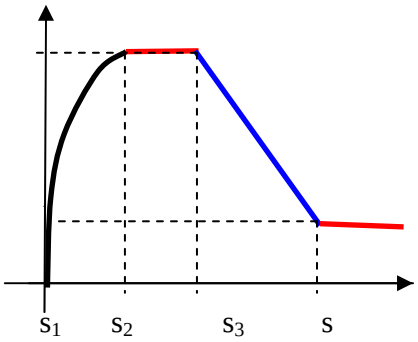


Figure 4. Exponential bond-slip model

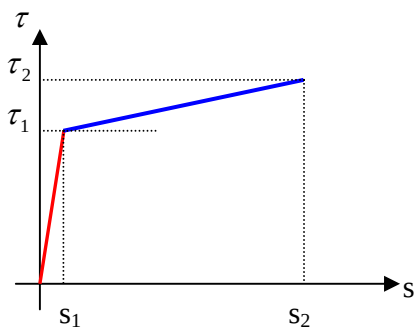


Figure 5. Bi-linear bond stress-slip

The local $\tau=\tau(s)$ relationship must be introduced in Eq. (1) for the solution of the structural problem and it can be proved that when the slips are small, the analytical expressions are the same laws and having the same structural responses.

4.2 Interpretation of pull out tests

In this section, a method to calibrate a given local bond slip; $\tau=\tau(s)$ relationships of pull-out test results, is described. It consists of the numerical solution of Eq. (1). Useful relationship involving the pull-out tests between the loaded end and the zero-slip point can be determined by using an energy approach. The external forces work acting along the transfer length:

$$W_{ext}=\pi D\int_0^x\left(\int_0^{s(x)}\tau(s)ds\right)dx \tag{4}$$

The elastic energy in the bar along x of length is:

$$W_{\text{int}} = \frac{1}{2}\int_v \{\sigma\}^t \{\varepsilon\} dv \tag{5}$$

From Eqs. (4) and (5) for the one-dimensional problems, it's possible to obtain:

$$\pi D \int_0^x \left(\int_0^{s(x)} \tau(s) ds \right) dx = \frac{A_s}{2E_s} \int_0^x \sigma^2(x) dx \tag{6}$$

and it leads to;

$$\int_0^x \left\{ \pi D \int_0^{s(x)} \tau(s) ds - \frac{A_s}{2E_s} \sigma^2(x) \right\} dx = 0 \tag{7}$$

The Eq. (7) must be satisfied for each value for x ($0 \le x \le l_{an}$: which l_{an} is the transfer length), the above relation can be expressed as:

$$\pi D \int_0^{s(x)} \tau(s) ds - \frac{A_s}{2E_s} \sigma^2(x) = 0 \tag{8}$$

For the particular value of x= l_{an}, which corresponds to the anchorage length,

$$\pi D \int_0^{s_{an}} \tau(s) ds - \frac{N^{*2}}{2E_s A_s} = 0 \tag{9}$$

The corresponding total load is generated along the transfer length is expressed by:

$$N^* = \pi D \sqrt{\frac{1}{2} D E_s \int_0^{s_{an}} \tau(s) ds} \tag{10}$$

which s_{an} is the slip at the loaded end.

When a local relation of bond is chosen, the Eq. (10) represents the applied load-slip relation.

4.3 Determination of anchorage length

Consider a bar of pull-out test and application of the proposed method. The local bond-slip relationship is described by the relations by the relations (2) and (3) with its parameters. Let D =12 mm is the diameter, A_s $\frac{\pi D^2}{4}$ =113.04 mm² is the cross area and E_s=210^5 MPa is the longitudinal Young's modulus of the bar. The bar is embedded in concrete block, Figure 6,

the minimum anchorage length, l_{an}, needed to obtain no-slip at the free end when the nominal load $N^{*=}$ 10 KN, is applied to the bar, is chosen.

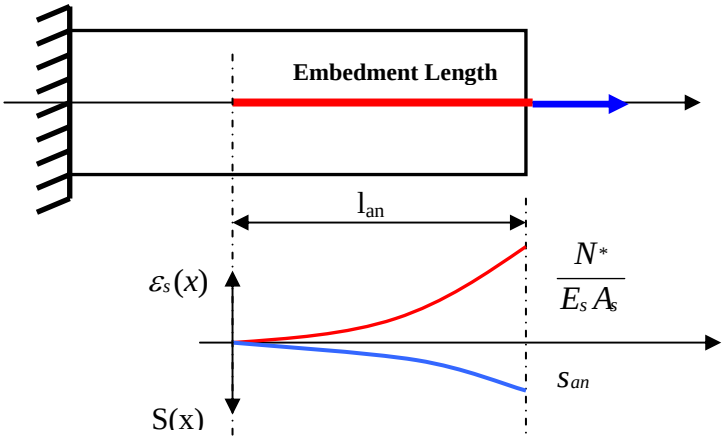


Figure 6. The pull-out test: strain and slip repartitions

For the same problem, it is necessary to compute the minimum anchorage length l_{an} and the corresponding load N^*. Therefore to resolve the problem, it must be providing two equations to be established.

The first step consists of determining the maximum pull-out load N^*, for each relationship of bond $\tau=\tau(s)$, the general solution s(x) of the differential Eq. (1) can be expressed as:

$$s(x) = g(x) \tag{11}$$

where g(x) is the function solution.

Using the boundary conditions for both ends of the reinforcing bar to determining the integration constants:

Table 1. Materials characteristics

Material properties	**Values**
Concrete compressive strength	35 MPa
Concrete tensile strength	3.5 MPa
Concrete E modulus	20909 MPa
Poisson's coefficient	0.30
Steel E modulus	210.000 MPa

Table 2. Boundary conditions

free end : x = 0	loaded end : x = l_{an}
$\varepsilon_s(x=0)=0$	$\varepsilon_s(x=l_{an})=\dfrac{N^*}{A_s E_s}$
S(x = 0) = 0	s(x = l_{an}) = s_{an}
$\dfrac{ds(x=0)}{dx}=0$	$\dfrac{ds(x=l_{an})}{dx}=\varepsilon_s(l_{an})$

The numerical solution s(x) of Eq. (1) with boundary conditions (Table 2) corresponding to either local bond-slip relationship (2) or (3). The value of x to which N^* is needed anchorage length.

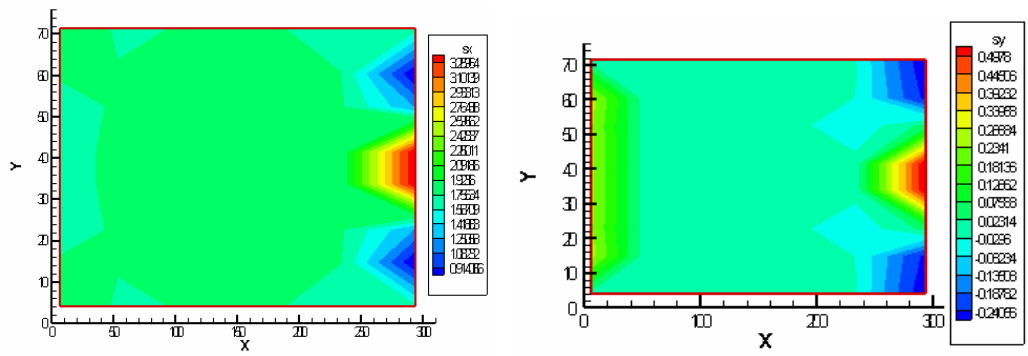
5. NUMERICAL EXAMPLES

In order to test the proposed demarche to calibrate the bond-slip effect, the response of anchored reinforcing bars under monotonic pull-out load is studied. The specimen is an anchored # 12 bar in a well confined block of concrete of 300×75.31×75.31mm^3 witch corresponds to anchorage length of 12.5 bar diameters.

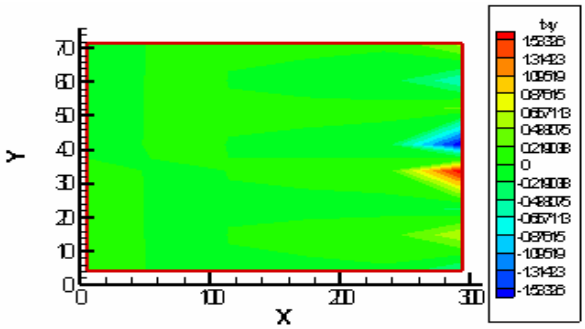
In studied cases, the concrete was modelled by eight-node Serendipity plane stress elements with 2×2 Gauss integration points and the reinforcement bar was modelled by three-node truss elements.

The material proprieties of materials are shown in Table 1 and one of bond are described in section 4 indifferently for the selected relationships.

To calibrate the bond slip effect between concrete and steel, three distinct models are selected, such as: (1) full perfect, (2) Eligehausen model and (3) bi-linear model. These models are introduced in finite element program and the collected results are analysed and discussed in next section.

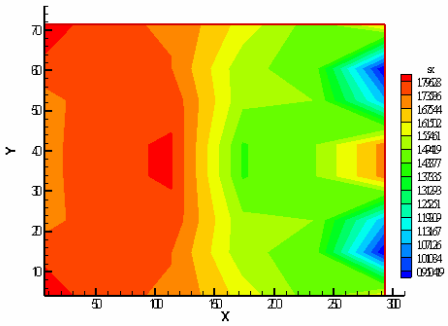


(a) (b)



(c)

Figure 7. Stress repartition in plain concrete, (a) σ_{xx}, (b) σ_{yy} and (c) τ_{xy}.



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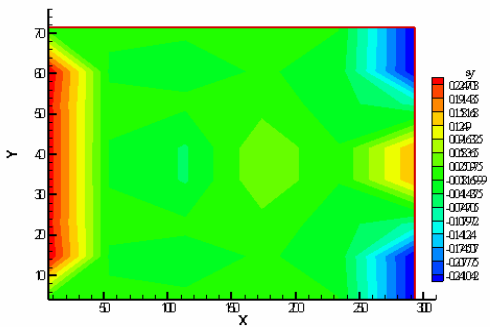

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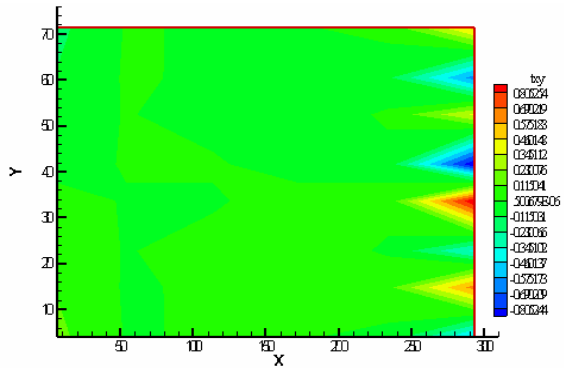
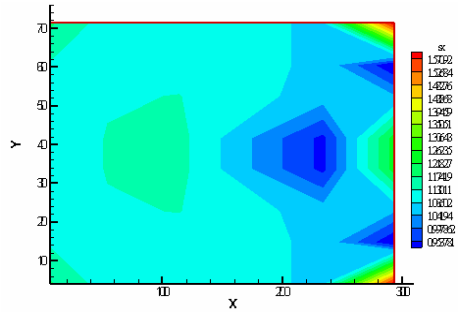


(c)

Figure 8. Stress repartition in reinforced concrete for perfect bond, (a) σ_{xx}, (b) σ_{yy} and (c) τ_{xy}.

The analysis of the distribution of normal and shear stresses along anchored bar where the full-perfect bond is assumed, in this case, the transmission of stresses is carried out in the integral way relative to the contact nodes, Figure 8.



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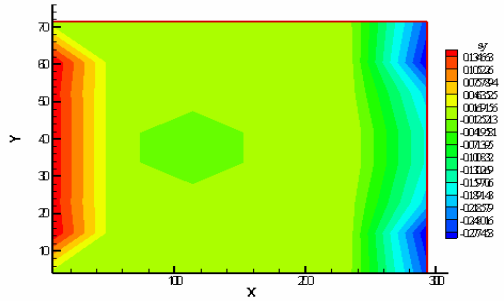

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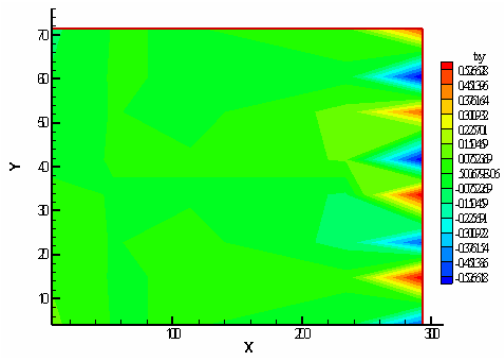


(c)

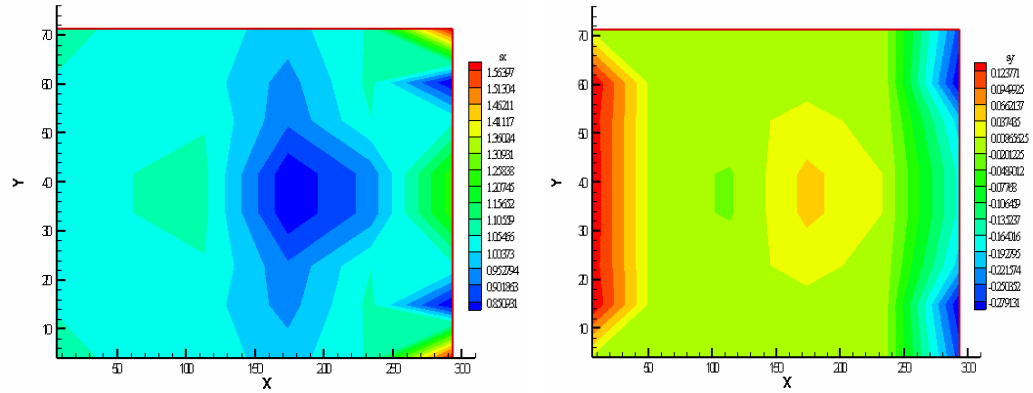
Figure 9. Stress repartition in reinforced concrete with Eligehausen model, (a) σ_{xx} , (b) σ_{yy} and (c) τ_{xy}

The above Figures (8-10) showed the concentration and the dissipation of stresses in the interface zone for full-perfect model, exponential model and bi-linear model respectively. When the degradation of bond starts in certain nodes (localized stresses), there will be a total re-reduction of the stresses between reinforcing bars and concrete by the intervention of bond.

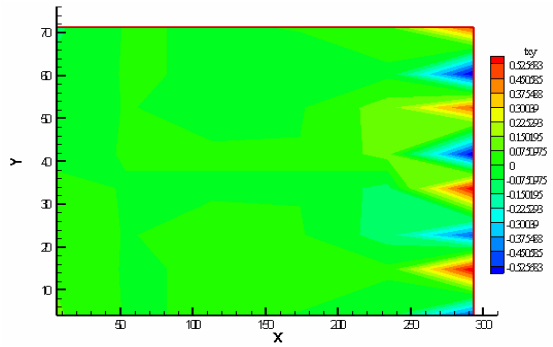
The comparison between distinct bond models, it is possible to record in presence of the interaction, the stress stresses are much concentrated in loaded end and they propagated inside of the structure. Thereafter the damage continues proportionally to the degradation of the bond characteristic and growth of concrete stresses.

From obtained results, it can be noted that the incorporation of the interface in a numerical simulations makes possible to reproduce in a more satisfactory way the rational observations, due to the transmission stresses between concrete and reinforcing bars.

According to the normal and shear stress curves (Figures 8-10), it is possible to appreciate how the connection influences the transmission of the efforts from steel bar towards the concrete and vice versa. From the Figures, it can be seen that analytical results of two kinds of elements (linkage and contact) are almost identical but the full-perfect model under-estimates the bond strength.

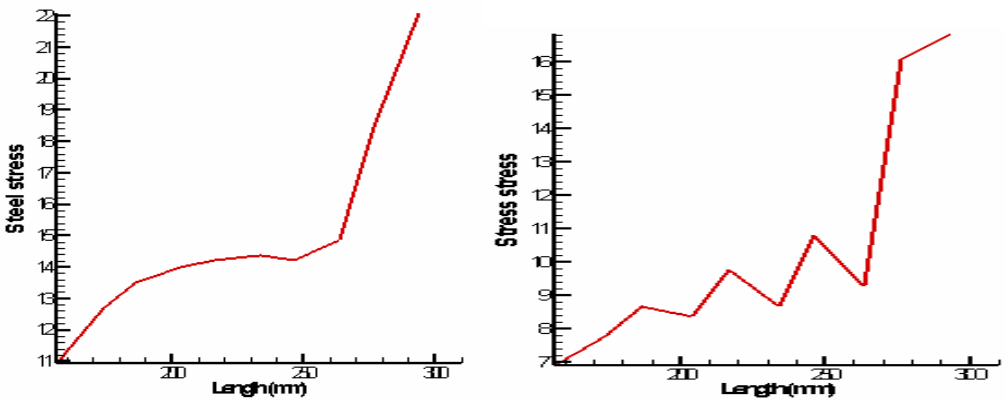


(a) (b)

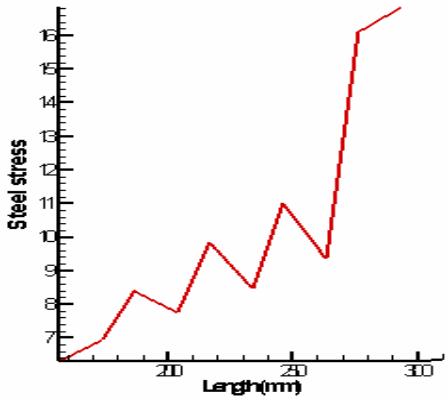


(c)

Figure 10. Stress repartition in reinforced concrete with bi-linear law of bond, (a) σ_{xx} , (b) σ_{yy} and (c) τ_{xy}



(a) Perfect model (b) Exponential law



(c) Bi-linear law

Figure 11. Steel stress along the anchorage length

The Figure 11 shows the steel stress repartitions along the reinforcing bar; in this case, the predicted steel load computed the full-perfect model is over-estimated the steel load and it is 31 % higher than ones calculated by contact models.

6. CONCLUSIONS

The paper studied the calibration of bond-slip behaviour under monotonic loading. It presents the analysis and the incorporation for perfect bond and contact bond models for use in finite element analysis. These formulations were conducted to the following conclusions: (1) The improvement of finite element models of composite material, it is necessary to use not only the constitutive laws of concrete and steel but also one of the interface, (2) The stress distribution in the steel bar of pull-out tests may principally be influenced by the properties of the interface, (3) The transfer length depends of the bond stress relationship, (4) It appears that the full bond model under-estimates the strength bond and analytical response computed using contact element match slightly better than those calculated using linkage element.

Although this study is restricted to 2-D problems, for the efficiency and reliability of the proposed demarche, the bond-slip can easily be adopted to 3-D problems and added to 3-D finite element programs.

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