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Finite element implementation of a steel-concrete bond law for nonlinear analysis of beam-column joints subjected to earthquake type loading

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Abstract. Realistic steel-concrete bond/slip relationships proposed in the literature are usually uniaxial. They are based on phenomenological theories of deformation and degradation mechanisms, and various pull-out tests. These relationships are usually implemented using different analytical methods for solving the differential equations of bond along the anchored portion, for particular situations. This paper justifies the concepts, and points out the assumptions underlying the construction and use of uniaxial bond laws. A finite element implementation is proposed using 2-D membrane elements. An application example on an interior beam-column joint illustrates the possibilities of this approach.

Key words: finite element; nonlinear analysis; reinforced concrete; beam-column joints, steel-concrete bond; cyclic behavior.

1. Introduction

Today, simplified finite element models are capable of describing very faithfully the behavior of lineic reinforced concrete elements while being operational on standard small size computers (Fardis 1991). Their main application deals with structures of the frame type, that is, composed of elements which definitely verify the basic assumptions for their formulation. However, the connections between these elements violate the beam theory, and cannot be considered as infinitely rigid: reinforced concrete joints are the center of important distortions, of slippage between steel and concrete, and their contribution to energy dissipation and global flexibility may become important. In that manner, the reliability of the beam models are significant only if their semi-rigid connections are correctly modeled, and this is the framework of the present study. It aims more particularly to propose a modeling for the joint, which is reliable enough to serve as a reference for the construction for a global component based model for the connections of reinforced concrete elements (Fleury 1996-a).

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1.1. Beam/column joints

The different behaviors of a typical interior beam/column connection along diverse loading paths have been described in (Viwathanatepa *et al.* 1979-a, Del Toro Rivera 1988, Paulay 1989, Wong *et al.* 1990, Leon 1990, Kobayashi 1992, Fleury 1996-b). From this phenomenology, the following aspects must be acknowledged for modeling:

(a) Four deformation mechanisms coexist, that can be observed at a semi local level (Fig. 1):

- Yielding of the main flexural steels at the beam/column interface which extends both in the connection and the beam;
- Slippage of steel in regard to its concrete sheath provoking the opening of the beam/column interface and which is mainly responsible for the pinching of the hysteresis loops;
- Distortion of the connection under diagonal cracking;
- Distortion of reinforcement at the interface: dowel action.

These modes of deformation not only coexist, but the mechanisms which generate them are interdependent: the steel/concrete bond depends on the importance of the shear cracking of the central part, on the state of the steel - elastic or plastic -, on the dowel action; the dowel action depends on the opening of the interface crack which is influenced by the yielding of steel.

Their relative contribution to global deformation evolves during the course of time and adds up to the other modes of deformation of the structure: bending and distortion of beams and columns. The fixed-end rotation may contribute by itself up to 50% to the inter story drift. The steel distortion at the interface does not participate much to that displacement, but the action of the reinforcement on the concrete, the dowel action, modifies in particular the bond in the interface area which is, however, already significantly deteriorated when this phenomenon becomes important.

(b) It is difficult to isolate the connection as having its own behavior law: First of all, the boundary conditions to which it is subjected vary during loading, thus modifying the stress

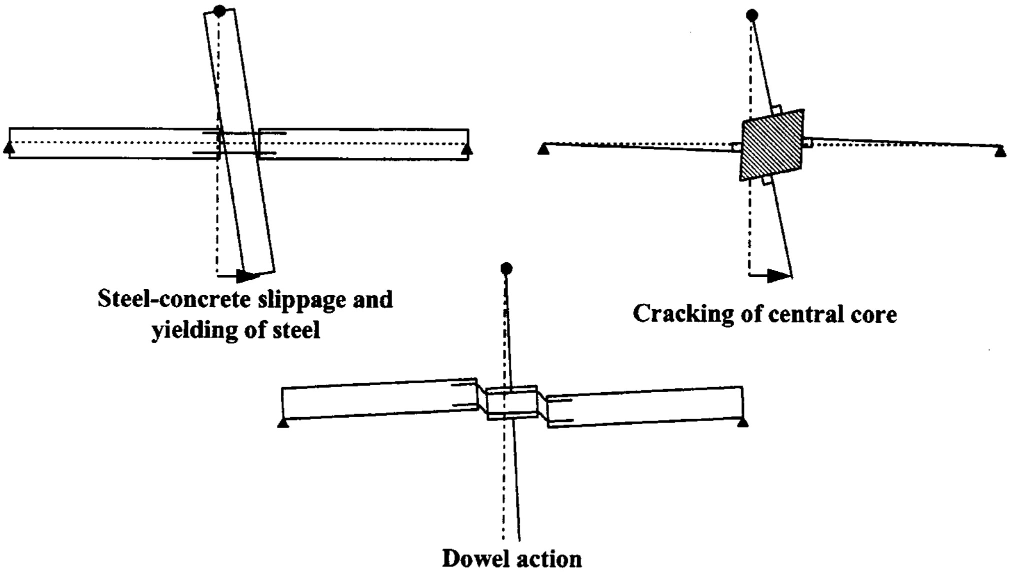


Fig. 1 Different deformation modes of the connection

distributions and the global behavior. Furthermore, these boundary conditions may also be the object of a behavior law and they depend on the state of the neighboring beams and columns. Finally, the moment/curvature laws of the beam sections on both sides of the connection are not independent one of the other, as soon as the bond, subjected to push-pull, deteriorates.

1.2. Consequences for modeling

The relative variations of the generalized stresses loading the connection are not known beforehand and depend on the seismic forces distribution and on the sequence of appearance of the 'plastic hinges'. In that manner, the space of the generalized stresses as a function of which the internal connection global law could be formulated, has twelve dimensions: moments, shear and normal forces in the four connected elements. The likely assumption of a nil normal force in the beams and equal column shear force cuts this range by three dimensions. If the connection is located on the lower floors of a high rise building, it is possible to neglect the normal force variation on both sides of the joint. The range is then reduced at best to eight dimensions. Furthermore, the inventory of the parameters influencing the behavior shows that at a global level, their number is large and that their influences are not independent (Fleury 1996-b) (the influences of one parameter depends on the value of several others a priority in a non-linear manner). Confronted to such complexity, the number of experiments to calibrate a global law is very limited. An alternative consists of going one degree down the observation scale and consider the elementary mechanisms.

The shear behavior of the central core requires at least a cyclic biaxial concrete law. Besides the connection thickness, the third dimension contributes by the degree of lateral confinement brought by the hoops. This confinement can be important if cracks within the loading plane form, or if the stresses are close to the compressive strength.

As far as the steel/concrete slippage mechanism is concerned, its modeling requires particular attention owing to its specificity. It is at the same time imperative to be able to describe this phenomenon which greatly participates to the inter story drift, and which also plays a part in the introducing of forces into the connection. The following is therefore dedicated to the study and to the modeling of the bond.

2. Uniaxial approach to bond modeling

In order to justify the choices leading to the model described here, a number of phenomenological aspects must be addressed first. Sound theories have already been proposed to describe the mechanisms of steel-concrete slippage and bond degradation (Tassios 1979, Viwathatatepa 1979-a). Starting from these theories, a number of influencing parameters can be identified, amongst which: height and spacing of the ribs, concrete mechanical characteristics under the in-situ conditions of the 3-D stress state and strain rate, casting direction, specimen scale.

Beyond these material and geometry parameters influencing the bond behavior, it is the influence of the actual situation, or confinement within the column/beam connection that complicates the identification of its mechanical characteristics. Because this last parameter is probably the most difficult to properly take into account, its influence is described now in more detail.

2.1. Confinement

2.1.1. Radial stresses from external forces

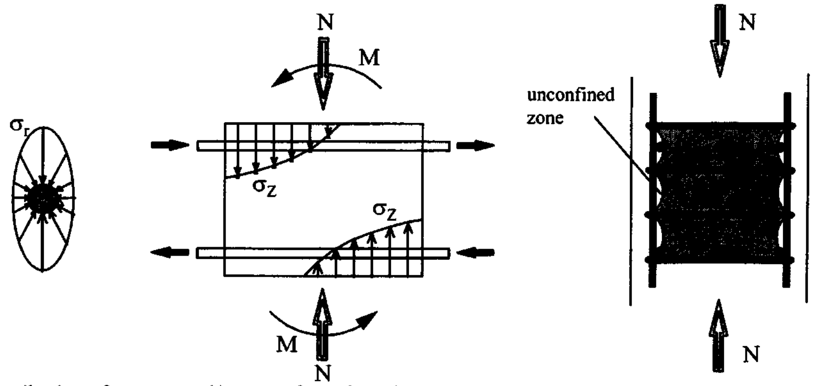
The compression stresses are exerted mainly along the direction of the column axis but also along all the other directions radial to the reinforcement, since concrete strain due to the Poisson effect is partially blocked by the hoops. Compression generated by the action of longitudinal and lateral beams contributes also to the confinement. For a given constant stress brought by the column, the importance and distribution of radial stress around the bar depend on the bar position and of the position along the bar. This is illustrated in Fig. 2(b) and (c). The state of stress is not axisymmetric anymore (Fig. 2a) and the problem becomes actually triaxial. There cannot be much negative confinement due to the fact that tension is limited by the resistance of concrete under tensile stress.

2.1.2. State of the steel

The Poisson effect within compressed steel creates an internal pressure on the concrete sheath and acts as a confinement. Conversely, steel under tensile stress tends to shrink, and the available area opposed to the concrete keys, as well as friction, decreases. These phenomena are even more marked if the steel yields. A radial compression stress generates of course a positive action on the bond mechanisms: not only the concrete resistance happens to increase, it also restricts conical, radial, and cylindrical cracks. The friction coefficient increases, both between steel and concrete and between the two concrete surfaces. Therefore, failure occurs subsequently to a more pronounced crushing of the struts to which the concrete cones are assimilated, leading to a more ductile behavior. The struts are more inclined with respect to the bar axis, and do not emerge, thus, as rapidly out of the concrete block front face. This delays the forming of a 'tension cone'.

2.1.3. Effect of open surfaces vicinity

When the cone cracks reach the cracked beam/joint interface, a concrete block breaks out, and



(a) Distribution of radial stresses (b) Forces from the columns (c) Confined areas under normal stress

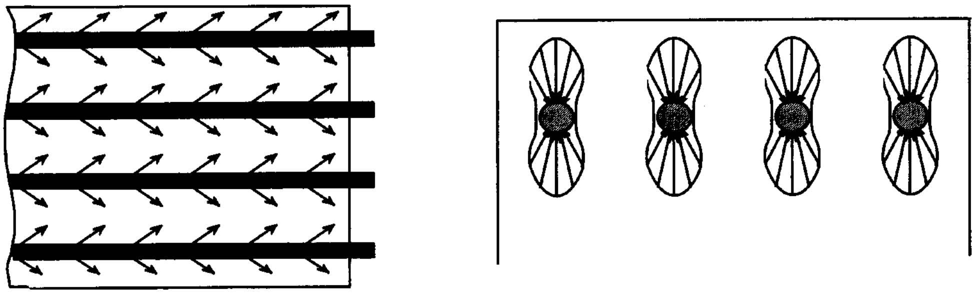
Fig. 2 Radial stresses on bond and confinement within a beam/column joint the bond can be considered as nil over that distance. The cover thickness plays also an important part, and its effect can be compared to that of a nearby parallel bar subjected to the same displacement: upon the open concrete surface parallel to the rebar, there exist no forces capable to balance those coming from the steel. The extreme no cover condition may happen if the cover has been destroyed as the result of a high compression or of a high level of shear cycling in the connection. Similarly, between two parallel bars, the forces from the steel are opposed along the radial direction but add up along the tangential direction, and find no reaction, so that in both cases, the bond cannot develop within a horizontal plane (Fig. 3). The condition is closer to a planar loading within a vertical plane, longitudinal to the bars. In both cases, radial cracking is facilitated. This crack acts in the same manner as a negative confinement or as a shrinking of the steel: The lug/concrete contact surface is smaller, the friction lower, and the concrete stiffness drops along the radial direction. Within the beam/column joint, the phenomenon is quite slowed down by the confinement and occurs essentially when the steel is in tension. Depending on the location of the vertical bars of the column crossing the connection, these may limit the phenomenon.

2.1.4. Dowel action

Another situation occurring at the beam/joint interface in a real situation is the dowel action of the reinforcement onto the concrete. When the interface is almost totally cracked, the bars alone pick up the shear force. This dowel action reduces the bond resistance, especially in the beam, over an area extending from two to three times the bar diameter (K. Kobayashi 1992).

2.1.5. Shape of anchorage

In the exterior connections, the flexural steel must be bent toward the inside of the joint in order to be sufficiently anchored. This generates a complex triaxial state of stress which modifies the bond mechanisms. The hook is responsible for local compressive stresses in concrete inside the curved part. Furthermore, to undergo any slippage, the bar has to straighten up, and thus works under bending. It is not anymore a bond problem but rather a dowel type anchorage problem.



Forces exerted by the steel within the horizontal plane

Hypothetical circumferential distribution of bond

Fig. 3 Influence of the vicinity of a rebar or of an open surface parallel to the bars

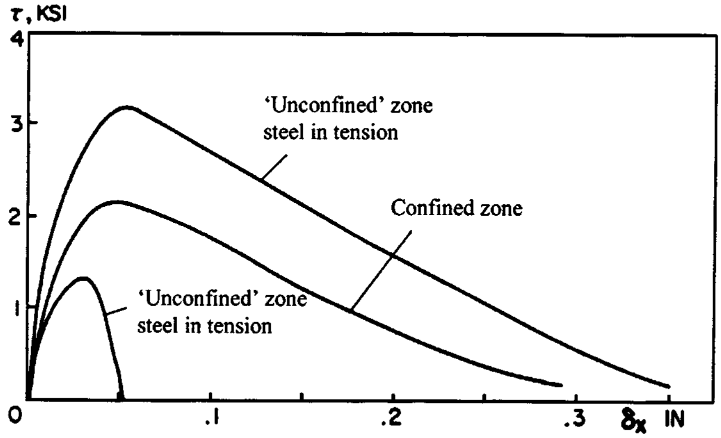


Fig. 4 Typical *t-d* curves for different regions (taken from Eligehausen *et al.* 1979)

2.1.6. Consequences on bond-slip curves

It has been underlined that the mechanisms that contribute to bond depend largely on the situation in which the bond develops. Indeed, the problem is truly tri-dimensional, and strongly depends on the topology. In that manner, the most brittle regions concerning bond are the ones wherein the tension in steel is maximum. These areas cumulate the following drawbacks: no confinement, open surface vicinity, and dowel action. On the contrary, areas where the steel is highly compressed are tri-compression areas, which greatly improves the bond resistance and ductility (Fig. 4).

2.2. Modeling concepts

2.2.1. Need for a specific model

Bond develops essentially by the action of the steel lugs against the concrete keys, this generates diverse types of cracks which do not have in the real conditions of a beam-column joint an axially symmetrical character. On a theoretical standpoint, the best and most general model would consist in a 3-D cyclic concrete model associated to a cyclic steel model implemented within a refined 3-D mesh describing all the steel lugs. This solution is not realistic yet, because of the size of the problem created. Besides, as far as is known, there lacks convincing results of 3-D concrete models applied to a complex structure under general cyclic loading. It has to be reminded that such a model should allow the prediction of cracking in any three directions, the concrete crushing, the unilateral aspect of crack closing, and the shear transfer degradation between cracks according to the level and number of cycles.

The first simplification consists in reducing the problem size by assuming its equivalence with a smooth bar perfectly bonded to the concrete sheath. This would allow to by-pass the meshing of the lugs and to consider bigger elements, since stress distribution would then be much more homogeneous. This approach has been used in particular by Viwathanatepa *et al.* (1979-b), Clent (1987) and more recently by Pijaudier-Cabot *et al.* (1991). In all cases, monotonic, in-plane, or axially symmetrical calculations are performed in a domain of low level loading. They came up with interesting results but which are still unsatisfactory for prediction purposes (Fleury 1996-b). Furthermore, these methods are still unusable in three dimensions and under cyclic loading of high amplitude.

Another idea consists in formulating a specific model for this concrete zone wherein the different bond mechanisms develop.

2.2.2. Modeling scales and types of behavior laws

Around the ribbed steel, the stress gradients are high and degradations are complex. From a certain distance from the rebar, the action of the interior part of the cylinder upon the exterior part becomes much more homogeneous. The limit parting these two volumes depends on many parameters, it varies along the steel length and with the load, and is difficult to define.

The specific model must describe the behavior of the material here called 'transition concrete', which is between the rebar and the 'homogeneous' concrete here called 'transition concrete'. It must give a good representation of the bond/slip relationship, which has been shown as being a function of the radial stress. The behavior description space can therefore be reduced to a plane, and the laws be expressed in two dimensions.

Although it is possible to integrate these laws along the direction radial to the reinforcement, it is usual to consider the behavior of the transition concrete as averaged upon the radial direction. Given the difficulty met to define the transition surface between the transition concrete and the 'structural' concrete, the representation of this material as being concentrated within a very thin layer is preferred. This allows to consider the law no more as a completely local law, but as one which integrates the radial dimension in its mechanical characteristics. These are, hence, semi-local laws: global along the radial direction, local along the longitudinal direction. Furthermore, this option allows to use more directly the results of pull-out tests which yield average quantities.

2.2.3. Taking the confinement into account

For an axisymmetric situation or a 3-D modeling, laws expressed in the τ-s space (bond-radial stress space) are theoretically more satisfactory. The effect of a variable confinement is explicitly dealt with. However, using these laws entails for the modeling of the structure to be able to account for the radial stresses in a realistic way. This is not always easy, even in a tri-dimensional approach, as then all secondary transverse reinforcement must be individually modeled.

Besides the problems linked to the implementation of a cyclic 3-D concrete law, the beam/column joint when loaded within its plane essentially develops a planar state of stress, even though a confinement exists in the column due to the presence of hoops. In order to capitalize on that assumption which allows to drastically simplify the problem, the bond model must also be designed accordingly, in particular, the impossibility to access the radial stresses and their distribution around the steel perimeter will have to be taken into account. Therefore, bond and radial stresses have to be considered as averaged on the circumference. Two approaches may then be considered:

First approach: It consists in building relations between the in-plane stresses and the average σ_r radial stress on a steel or a steel layer for different steels layouts in the lateral direction: this would constitute a true 'confinement model'. The hoops percentage, their plastic behavior, the number of bars in a bed, the cover thickness would all need to be taken into account. The transition concrete law must be then bi-axial, and must include a stress pre-processing to access σ_r.

By doing so, two major problems arise: first, building the cyclic 'confinement model' seems difficult, and would require a great number of tests or of three dimensional analyses. As far as is known, such an attempt has not been tried yet. Secondly, the existing bi-axial bond models which, for most of them, are expressed within the framework of classical elasto-plasticity, are only valid under monotonic loading.

Second approach: It consists in mapping the average confinement and constructing uni-axial laws whose mechanical characteristics depend, between others, on the position within the structure. The confinement evolution is then considered as being a function of the evolution of slippage and, therefore, can be implicitly accounted for by the law. Assumption has to be made that to a given slippage history corresponds a single confinement history, already taken into account by the characteristics of the law. This is only valid for a generic load history which is the one encountered in a beam-column joint. The model is then specialized, and the bond mechanical characteristics will be valid exclusively for a beam-column joint implementation. This approach has the advantage of simplifying the expression of the behavior law which is then uni-axial in terms of bond-slip, and of allowing to rely on the numerous tests carried out which reproduce this situation. Yet, similarly to the first approach, characteristics determination is a function of many parameters which determine the confinement such as the axial force coming from the columns, the hoops percentage, the number of bars within a bed, the position in the connection, on top of other mechanical and geometric characteristics.

The bond model searched for must be compatible with the joint bi-axial modeling, and may be exclusively specialized for that type of problem. It must describe imperatively the cyclic behavior. For these reasons, the only existing type of law depicting the cyclic behavior has been chosen, that is a uni-axial bond/slip law taking implicitly into account the variable confinement distribution, and whose characteristics are expressed accordingly to all the parameters which influence directly or not the behavior.

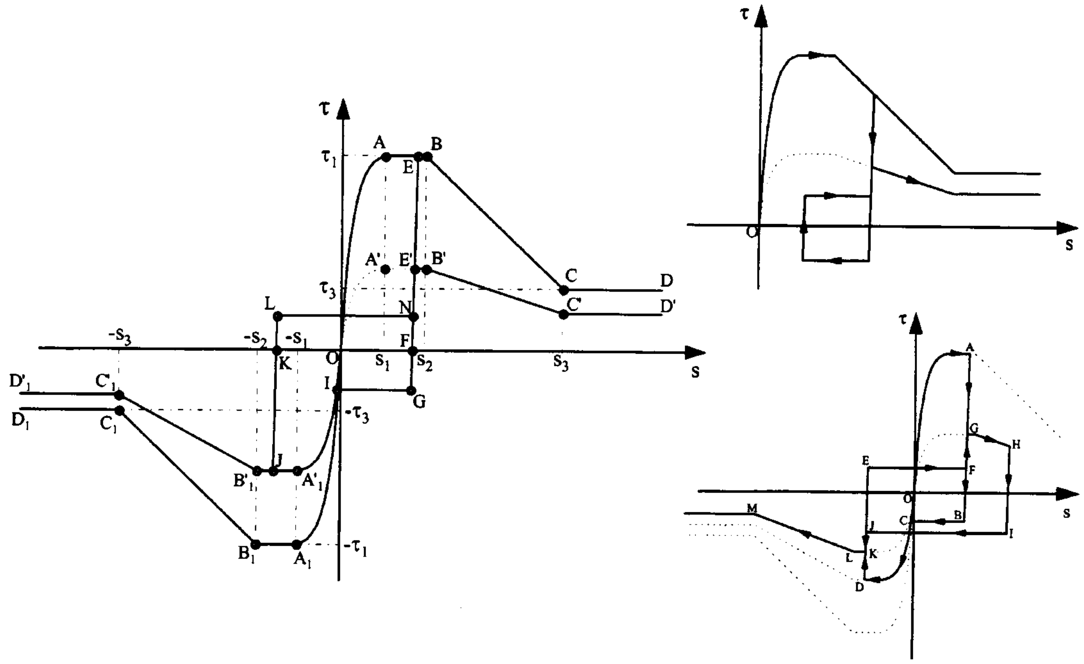
2.3. Description of the bond model used

The model adopted and presented herein is a slightly modified version of the one proposed by Eligehausen *et al.* (1983).

2.3.1. Reference model (Eligehausen et al. 1983)- confined regions

Fig. 5 shows a typical cycle. The curve OABCD or $OA_1B_1C_1D_1$ describes the behavior under a monotonic load, and is entirely defined by the values of s_1, s_2, s_3, τ_1, τ_3, and α. It is composed of four segments:

1) The non linear ascending branch, of equation : $\tau=\tau_1.(s/s_1)^{\alpha}$.
2) It is followed by a plateau for $s \in [s_1 , s_2]$, defined by $\tau=\tau_1$.
3) The descending branch is linear up to point C of coordinates $\{s_3, \tau_3\}$.
4) The last segment defines the friction plateau, thus $\tau=\tau_3$. Unloading from this envelope curve is elastic with a high stiffness, until reaching the friction stress (path EFG). A decrease of slip is achieved under constant bond, up to the intersection with the OA'_1 curve (path GI). If slippage keeps increasing in the negative direction, a curve similar to that for monotonic loading is followed, however with reduced τ values (path IA'_1J). Unloading from Point J occurs again in a perfect elasto-plastic manner with a friction threshold until the previous unloading straight line (path JKLN) is reached. This unloading line is now followed during



Typical cycle Examples of loading paths

Fig. 5 Schematic description of bond law

reloading until it intersects the reduced curve envelope ($OA'_1B'_1C'_1D'_1$) of the positive direction: OA'B'C'D' which is then followed (Path NE'B'C').

If instead of reloading beyond point N, bond is subjected to cyclic loading between slip values of N and K, the bond/slip relationship is of the perfect elasto-plastic type, but with friction thresholds which deteriorate when the number of cycles increases. The loading paths examples of Fig. 5 illustrate several possibilities.

The reduced envelope curves are obtained from the monotonic curve by reducing the characteristic bond stresses τ_1 and τ_3, by a factor which is a function of a damage parameter d. For d=0 reloading reaches anew the monotonic curve envelope, for d=1 damage is complete, and bond is completely destroyed. The strong correlation between degradations of τ_1 and τ_3 observed experimentally led to adopt as damage index for τ_3 a function of that modifying τ_1. The damage parameter is computed solely from the dissipated energy during cycling. This energy is used, on the one hand to damage the material, and on the other hand to overcome friction which is partially transformed into heat. Therefore, only a fraction of the total energy dissipated is considered to compute d. The steel/concrete friction thresholds between unloading and reloading are also damaged according to a parameter based on the energy dissipated by friction alone.

2.3.2. Reference model (Eligehausen et al. 1983) - unconfined regions

The model as defined above is valid only in a well confined region where the low variation of radial stress does impact much the bond behavior. However, as was mentioned previously, confinement conditions vary from one location to the other within the beam/column joint. When the load is reversed, the regions that were the most confined correspond now to those of worst confinement, and *vice-versa*. The law becomes asymmetrical and is then characterized by five more parameters which define the initial curve envelope in the second direction.

In order to take into account the damage asymmetry, the computed energies corresponding to incursions in tension are multiplied by an amplification factor δ, calculated as a function of the ratio between the surface areas under monotonic curves respectively in compression and in tension. This definition of the amplification factor results in a reduced curve envelope in the negative direction at about 10% of the initial values when the first slip is imposed up to $s=s_3$, followed by a load reversal.

2.3.3. Modifications to the reference model

In the original law, δ is set as twice the ratio between the surface areas under monotonic curves respectively in compression (E_{opi}) and in tension (E_{opo}). In fact, symmetry is progressively restored when approaching the confined region; so that δ will vary continuously according to the variation of the envelope curves. Then, when reaching the confined regions of symmetric behavior, $E_{opi}=E_{opo}$, and $d=2$ instead of 1. It was thus decided to use an amplification factor given by $\delta=a.(E_{opi}/E_{opo})+b$, and b such that $\delta=1$ for $E_{opi}=E_{opo}$ (symmetry), and $\delta=2$ (E_{opi}/E_{opo}) for the extreme case of dissymetry.

Another modification was brought in to avoid having an infinite slope at the origin: the increasing part of the envelope curve is decomposed in: a first linear increase, according to the stiff unloading modulus, up to the intersection with the original curve, still defined by $\tau=\tau_1.(s/s_1)^\alpha$.

Finally, the computation of the updated frictional resistance for first unloading from a larger slip value than the peak slip in the previous cycles is slightly modified. Indeed, using the formulae given in Eligehausen *et al.* (83), for continuously increasing cycles, the ratio of frictional resistance over the residual resistance (τ_3) could be greater than 1 and increase indefinitely. This updated frictional resistance is computed here as the average between previous value and the value given for the first load reversal, taking the actual residual resistance in the corresponding expression.

3. Implementation of the bond model

3.1. Characteristics

A number of propositions are available for the definition of the law characteristic points according to the main problem parameters. Nevertheless, because the bond law is expressed only as a function of quantities which are supposed to be evenly distributed around the steel perimeter, this law depends strongly on the problem topology. Every test result is dependent on the very type of the test itself, and it is difficult to extrapolate the results (especially quantitative results) to too different structures (Fig. 6). This consideration greatly reduces the number of experiments usable for quantifying the model characteristics and their variations according to the parameters. The great number of parameters involved in the semi-local type of law chosen adds to the difficulty in finding suitable experiments. Experimental programs encompassing the whole range of the parameters variations are scarce, and the domains studied mostly pertain to axes, each

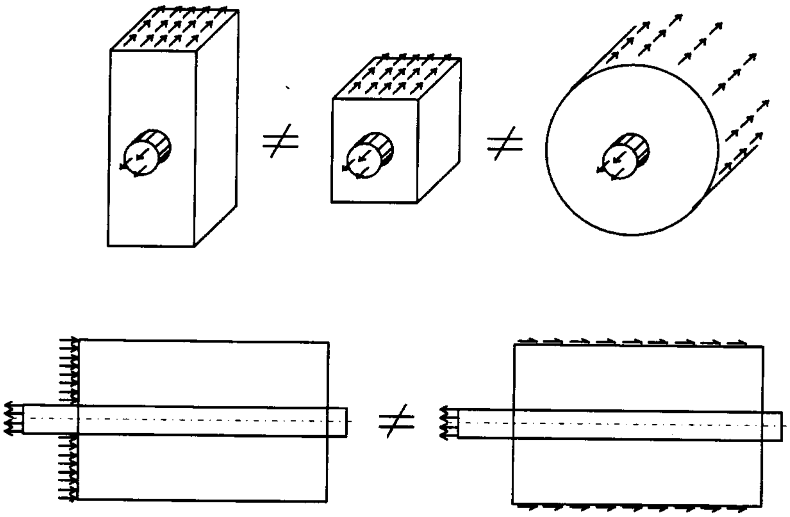


Fig. 6 Different boundary conditions related to the bond problem

parameter being studied in turn around the considered standard position. Moreover, some authors being interested in particular in only one or two quantities overlook to give information about the fixed parameters which still are needed to completely define the problem, such as the loading rate, the steel geometry, or even the concrete tensile resistance.

Nevertheless, in Fleury (1996-b) formulae are proposed for the determination of characteristics which rely on a number of propositions and on a number of tests when they are estimated to be appropriate to each particular situation in the beam-column connection: starting from a relatively generic value, the influence of each studied parameter corresponds to a coefficient generally independent from the other parameters. It is judged that gaps and wide approximations still remain, mostly for the unconfined regions located close to the connection/beam interface.

3.2. Finite element support

Different elements can support the selected law. Ngo and Scordelis (1967) used concentrated springs linking the steel and concrete displacements along the rebar axis. Goodman and John (1977) then Beer (1985) proposed zero thickness lineic elements, giving the relative displacements between the two faces at any curvilinear abcissa. Other authors (De Groot *et al.* 1981, Barzegar and Maddipudi, 1997) developped elements that include the steel strains. This can be seen as the pre-assemblage of two distinct elements sharing degrees of freedom. Recently, Monti *et al.* (1997) proposed a new element for reinforcing steel including bond slip, which uses force instead of displacements interpolation functions, yet implemented in a finite element program based on the stiffness method. This approach allows to satisfy equilibrium within the element in a strict sense, and would thus enable the use of large size elements, one for each zone of uniform bond characteristics. In this approach, compatibility of displacements with concrete is only obtained at the two end nodes connected to concrete. In fact, no application is shown in which the new element is integrated in a concrete mesh.

In this study, continuous thin elements are used to implement the bond law. Although it is not necessary to resort to a very continuous interpolation of the displacements along the radial direction, yet slippage distribution along the element must be approximated in a more precise way. For that reason, the six-node element (three along the longitudinal direction and two along the radial direction) is used by many authors. The QUA4 element, with a bilinear displacement interpolation, and the TRI3 constant strain element are two classical elements, existing in all the codes, and which can also be used as small thickness elements as a support for the selected bond law.

Having one of the dimension tending towards zero gives them similar properties: very low stiffness under longitudinal membrane action, and independence of the distortion and of the normal and longitudinal strains. When this thickness becomes very small, the stress along the privileged direction tends towards zero and the element can only work under shear stresses or membrane stresses in the radial direction. The thickness of the element can be considered either as a factor of regularisation or penalisation, or as a characteristic dimension (Sharma and Desai 1992). These three elements are thus differentiated by the shape of the stiffness matrix, strain continuity, and number of integration points.

Among these, the small thickness QUA4 element seems to be the most accurate, both for displacements and for strains, given the same total number of degrees of freedom (Fleury 1996-b). Among the three compared elements it is the one displaying the most integration points: four. It is therefore, the most demanding in terms of law calculation and integration. It is thus proposed that for that type of application, the small thickness QUA4 element be used, keeping two Gauss points only located on its longitudinal axis, which is enough owing to the small thickness.

3.3. Validation example

The implementation of the bond law has been validated in (Fleury 1996-b) on the Viwathanatepa, Popov and Bertero's tests (1979-b), which are well described, documented, and detailed. The structure consists of a bar embedded in a concrete block, representing a beam/column joint, either pulled only on one side, or pulled on one side and pushed on the other. Among the series of tests carried out on this device, tests including monotonic and cyclic loading, along with failure by steel yielding or bond deterioration were used for the validation.

Results of monotonic calculations are very satisfactory: Indeed, the initial stiffness, the beginning of yielding, the stiffness evolution and the ultimate resistance are correctly predicted. However, the examination of the strain, stress, and slippage distributions throughout the structure at certain time steps, leads to consider with caution the excellence of the global results obtained: it was concluded that the bond characteristics distribution and the assumption made that they do not vary in the course of loading are the main sources of approximations. The whole difficulty to predict, a priori, the degree of confinement at one particular point and its evolution, which is supposed to be implicitly integrated within the behavior law, is met again. The cyclic results are slightly less convincing, due to the use of an isotropic hardening model for steel.

Despite of these few reservations about the precision of the local results under high loads, they remain qualitatively good, and they are judged quantitatively satisfactory: given the scatter which does occur anyway on experimental results, a greater precision would be illusory. If the variation of the bond characteristics along the bar is difficult to apprehend, the good precision of the global results indicates that the values a priori chosen considering the problem data remain valid in an 'average' sense.

4. Application to a beam-column joint

The bond model presented in the previous part has been developed in order to allow the description of one of the most important phenomena of the interior beam/column joint behavior. The first purpose of this calculation is to put the model in the situation for which it has been designed and to evaluate its relevance.

4.1. Test description

The selected structure is the beam/column connection tested by Del Toro in 1988. The specimen, loading, and boundary conditions have been chosen to describe the situation of a connection within a ten story frame loaded by an earthquake. The reinforcement has been calculated according to the dispositions of ACI Code 318-83 to insure a ductile failure mode by beam flexure. During the experiment, the cracking of the central core and the steel/concrete slippage have been observed.

The schematic description of the specimen is presented in Fig. 7. The different elements are all 20 cm thick. The length of the beams and columns, measured from the connection center, are respectively 2 and 1 m. At their extremities, the beams are resting on simple supports while the column is articulated at its top. The beams flexural steel is asymmetrical because its determination takes into account constant vertical loads which are brought by the floors. The hoop spacing is constant over the whole length of the column, joint included, while in the beams, the hoops are twice as closer in the connection vicinity than at their ends.

The loading is done in three stages: 1 First, a 200 kN compression force is applied on the columns to simulate the action of the vertical loads of the upper floors. 2 To simulate the vertical loads of the considered floor on the beams, an initial flexion is then imposed. It is introduced by applying a vertical force upon the beams extremities and by arresting the vertical displacement on the support. 3 Finally, the seismic action is simulated by a horizontal force which is applied at the

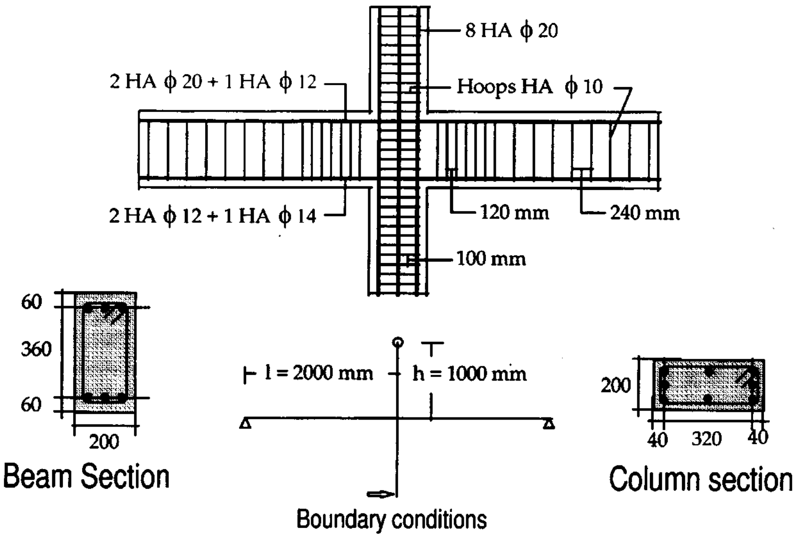


Fig. 7 Schematic description of Del Toro's connection

lower end of the beam, with a displacement control. Cycles with increasing amplitudes are imposed, each train of cycles consisting in five cycles of the same amplitude.

4.2. Modeling

The connection is modeled along its medium plane with membrane elements and plane stress assumption. The complete mesh is shown in Fig. 8. It is composed of 384 concrete elements, 32 bond elements, and 528 steel elements. The number of nodes is 475, which gives 950 degrees of freedom. The mesh for the concrete has been chosen so that the connection with the steel elements may occur at the exact location concerning the longitudinal reinforcements and the hoops of the columns.

Away from the connection and its vicinity, the steel/concrete bond is considered as perfect, and steel is directly connected to concrete. Inside the connection as well as within a 22.5 cm zone on both sides of the interface, the beams flexural steel is connected to the bond elements. The bond characteristics have been calculated on the basis of the formulae proposed in (Fleury 1996-b). The values obtained (Table 1) depend on the reinforcement layer and the section location. A single bond element per layer is used, and the characteristics are calculated for the average diameter of each bed.

The concrete model used is of the type 'smeared fixed crack' with possible double cracking only at 90° (Merabet *et al.* 1995). In the uncracked state, it is based on the plasticity theory, with isotropic hardening and associated flow. The behaviour after cracking is orthotropic, and governed by a uniaxial law in each direction. This cyclic law takes into account the unilateral feature of crack opening and reclosing.

4.3. Results

The comparison of the steel stress distributions along the lower layer at positive and negative peaks (C3P and C3N) of the third cycle close to steel yielding is posted on Fig. 9. A satisfactory concordance is observed for the tension and compression zones even if the experiment shows a steel yielding to the right at point C3N. The differences of experimental strains between the steels of a same layer may explain this difference.

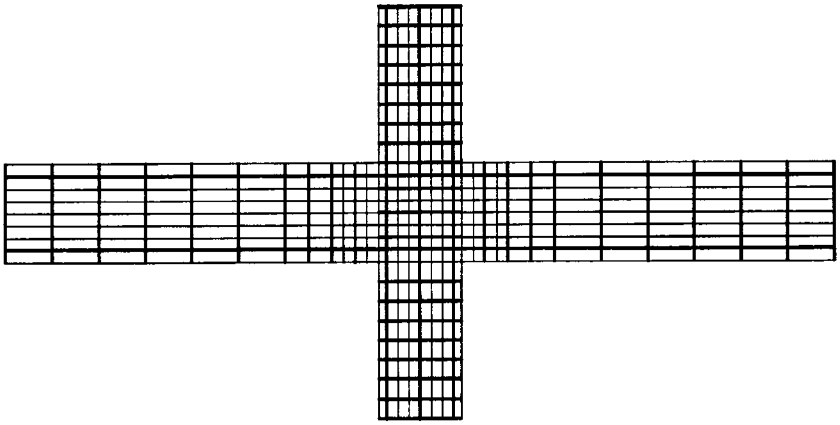


Fig. 8 Mesh adopted for the local modeling of Del Toro's connection

Table 1 Bond characteristics adopted (α=0.4)

Distance from column center (cm)	Bar in compression					Bar in tension				
	τ_1	τ_3	s_1	s_2	s_3	τ_1	τ_3	s_1	s_2	s_3
	(MPa)		(mm)			(MPa)		(mm)		
54.0 - 36.875	12.7	1.9	0.6	0.6	1.0	12.7	1.9	0.6	0.6	1.0
36.875 - 31.25	12.7	1.9	0.6	0.6	1.0	10.9	1.45	0.525	0.525	1.0
31.25 - 25.625	12.7	1.9	0.6	0.6	1.0	7.3	0.55	0.375	0.375	1.0
25.625 - 20.0	12.7	1.9	0.6	0.6	1.0	5.5	0.1	0.3	0.3	0.1
20.0 - 16.0 Bot	21.0	7.95	1.0	3.0	12.0	5.5	0.1	0.3	0.3	0.1
20.0 - 16.0 Top	21.0	7.95	1.0	3.0	12.5	5.5	0.1	0.3	0.3	0.1
16.0 - 0.0 Bot	19.0	5.8	1.0	3.0	12.0	19.0	5.8	1.0	3.0	12.0
16.0 - 10.67 Top	19.0	6.875	1.0	3.0	12.5	11.25	2.95	0.65	1.65	6.75
10.67 - 0.0 Top	17.0	5.8	1.0	3.0	12.5	17.0	5.8	1.0	3.0	12.5

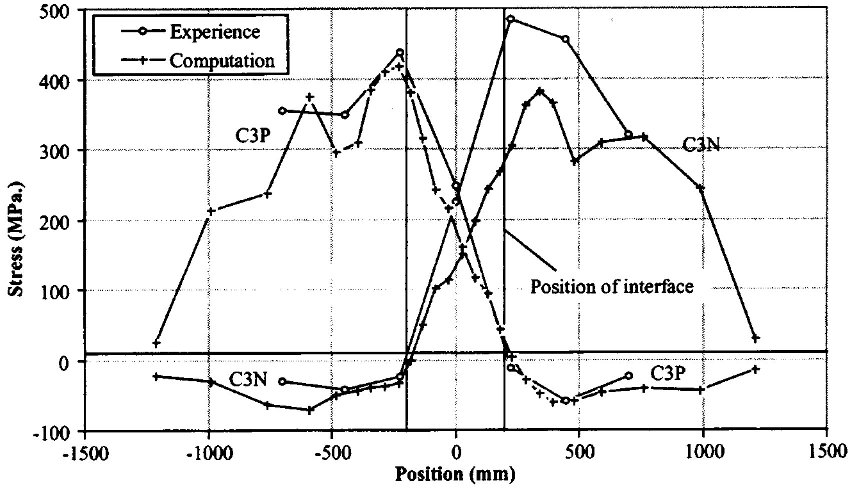


Fig. 9 Stress distribution in the lower layer at points C3P and C3N

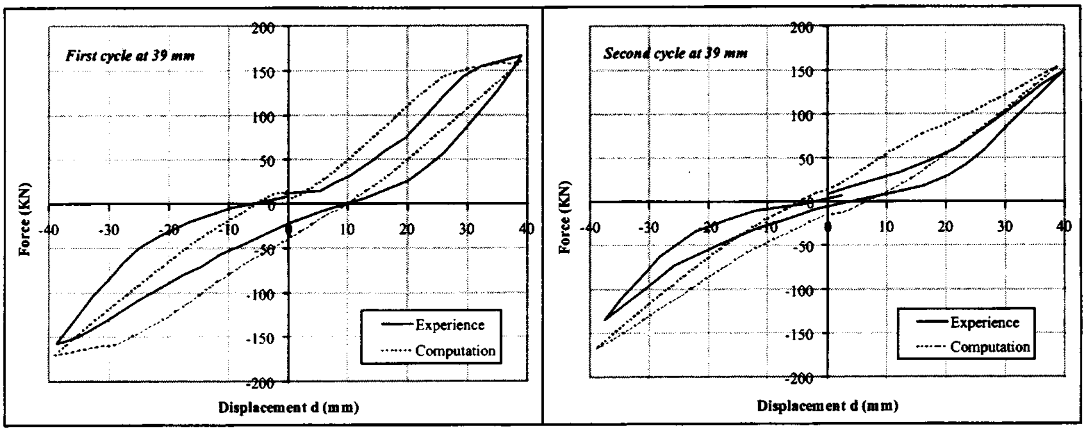


Fig. 10 Global curves for the first two cycles at 39mm

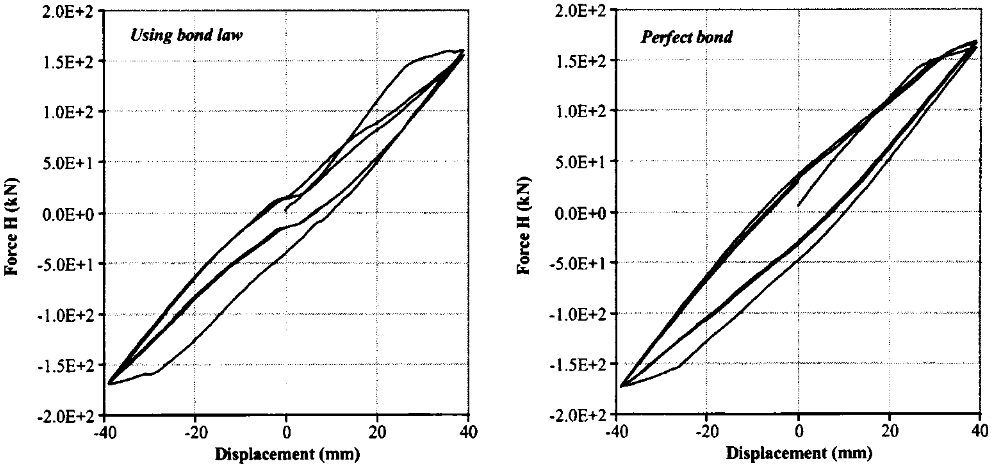


Fig. 11 Global comparisons with and without modeling the bond

The global analytical results agree reasonably well with those of the experiment than for the previous train of cycles, as shown in Fig. 10. Concerning the first cycle, the resistances at the peaks and the residual displacements are well predicted. The dissipated energy is of 3276J for the experiment and 3639J for the computation, that is an 11% difference, which is satisfactory. However, the shape of the loop is not as well described: it is not pinched enough, this could be the expression of a too weak steel/concrete slippage participation to the total displacement.

During the second 39mm cycle, the difference between computation and experience increase slightly, especially concerning the drop of resistance at the negative peak and the dissipated energy. This time, the computation gives a dissipated energy 30% greater than that of the experiment, and the difference between the drop percentages is of 13.5%. The assumptions which are retained to explain these differences are:

- An overestimation of the bond resistance throughout the joint, which results in an underestimation of the resistance drop, the loops pinching, and the drop of the dissipated energy.
- An underestimation of the pinching of the loops characterizing the distortion behavior of the central core. This could be due to the stiffness in crack reopening or reclosing of the uniaxial law, and to the double cracking at 90°.
- The steels isotropic hardening, which by increasing the elastic limit in both directions for each hardening, increases as a result the resistance of the cracked section.

A total absence of pinching is observed when bond is perfect (Fig. 11). For the 39 mm cycles, the perfect bond computation presents an increase in resistance at the positive peaks due to the steel isotropic hardening. With a better modeling of the steel behavior, a higher drop of resistance with cycling would thus have been obtained, in the computation with the modeling of bond.

5. Conclusions

The cyclic uniaxial law of the crack behavior describes the unilateral feature of its closing/reopening. This phenomenon is essential for the correct introduction of the force at the connection interfaces. It is this mechanism which, combined with the steel yielding, is responsible for the 'push-pull' loading of the bond, whose importance has been underlined many times.

The connection distortion is taken into account, this is also a key aspect of the connection behavior: it has been seen that this mechanism contributes in this case to about 30% of the global stiffness.

As for the bond model, it allows to predict the pinching of the hysteresis loops and the drop of the dissipated energy during the cycles. This deterioration of the dissipated energy is one of the most disastrous phenomenon for the good behavior of a structure: the damping which results from an important and stable dissipation is determining in preserving its integrity.

Therefore, it is the combination of these possibilities which confers to this modeling a sizable advantage over the models commonly used to predict the behavior of the frame type structures. On the basis of his experimental results, Del Toro evaluates at about 57% the contribution of the bending of the beams and columns to the global stiffness during the 26 mm cycles, when between 61% and 65% have been obtained here, in the last cycles at that level.

This result is very encouraging, and better results should be obtained with:

- Better evaluated characteristics of the crack cyclic behavior. Some quite large uncertainties remain over the choice of the post-peak cracking slope, of the strain of crack reclosing, of the reopening/reclosing slope, of the shear transfer factor. The best values seem to depend on the type of problem.
- A steel law which takes into account the Bauschinger effect.
- Bond characteristics better evaluated. The data base for this evaluation is actually still too limited to allow their determination with enough precision. In a deterministic approach, the variability of these characteristics will never allow more than the modeling of an 'average' specimen in the statistical meaning of the term.

Therefore, this modeling is judged sufficiently complete and precise to correctly represent the behavior of a connection subjected to cycles of strong imposed displacements. On the other hand, because of the level and the selected type of kinematic approximation, this type of modeling requires large computer resources, concerning computation time and memory space. Its cost and the mesh complexity hinder its use for the modeling of a complete frame type building which may easily contain several tens of interior connections.

The following objective is thus to improve the efficiency of the modeling and to move towards a more global approach (Fleury 1996-a, 1996-b). Moreover, the model will have to be compatible with the semi-global or global elements for beams and columns which are the only ones to possess the necessary efficiency for the seismic computation of a whole building. The shifting from a local model such as it is to a global model must keep a good description of the key mechanisms: unilateral behavior of the interface cracks, connection distortion, and steel/concrete slippage.

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