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NUMERICAL MODELLING OF LARGE REINFORCED CONCRETE SPECIMENS BASED ON EXPERIMENTAL TESTS FROM BENCHMARK CONCRACK

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Abstract: Most infrastructures are reinforced concrete (RC) structures. In order to deal with the security and durability of RC structures, the stackholders have to survey the noticeable signs of degradation resulting from interactions between the environment and the constitutive materials. As the matter of fact, these interactions can reveal cracks, which are major with respect to durability and sustainability.
In this framework, the French national research programme CEOS.fr (Behaviour and assessment of special construction works concerning cracking and shrinkage) has been launched [1]. It aims to improve knowledge on the behaviour evolution of special concrete structures (large structures) particularly their cracking states (crack openings and spacings) and to develop new tools allowing a structural behaviour prediction. Associated with the project CEOS.fr, an international benchmark named ConCrack (Control of cracking in RC structures) dealt with the modelling of the experimental behaviour of large specimens (www.concrack.org)[2].

In that context, the authors focused on the modelling of one tested mock-up. It is a large RC beam named RL1 subjected to a free shrinkage test followed to a bending test.
The authors focused on the mechanical part. In order to model the experimental test, a 2D plane stress modelling is performed. Two continuum damage concrete models are used, classical Mazars model [3] and Ricrag one [4]. The steel behaviour law is a classical hardening elastoplastic law. These models are supposed to be robust enough to allow complex structural modelling of an entire structure. Analysis of local results, obtained by post-treatment of continuum numerical results, is performed.

In this study, the authors show the effect of the modelling hypothesis and the considered boundary conditions on the numerical results and its importance to get a good accordance with experimental data. The numerical crack openings and spacings obtained are compared with experimental data. At last, the set of numerical results is encouraging,

1 INTRODUCTION

An important aspect of reinforced concrete structure analysis for design purposes is the evaluation of crack spacings and openings (widths). The cracking pattern of concrete structures has to be analysed and controlled, and numerical tools are necessary to give accurate prognosis. It is of primary importance to control the cracking in order to condition the correct functioning of the structures (durability, deformability, safety, waterproofness or airtightness, etc.). The structural design is based on a performance approach given in Eurocode 2 [7]. The estimation of the opening and spacing of cracks is provided by empirical formulas, design from simplified experimental tests. But all these formulations have a limited range of validity, and do not cover special RC structures, such like power plant vessel or dam.

Besides, computational tools used by engineers are not sufficiently powerful to determine a representative state of the cracking of reinforced and/or prestressed concrete structures especially if they are subjected to severe loads. Indeed, for several decades, many studies have been carried out in experimental and numerical way at different scales, in order to develop modelling able to describe and predict suitably and relevantly, according to the observation scale, the concrete behaviour. In this framework, the French National Project CEOS.fr has been initiated and is also associated with the ANR (National research agency) project Mefisto (Sustainable prediction of concrete infrastructures cracking) [1].

Based on the experiments driven by CEOS.fr project, an international benchmark named ConCrack (Control of Cracking in RC Structures) has been launched, dealing with the modelling of the behaviour of the special RC structural elements under monotonic and cyclic loading after free or prevented shrinkage [2].

This paper presents results based on the international benchmark ConCrack in order to predict qualitatively and quantitatively the cracking distribution of RC tested mock-ups, the beam RL1. Numerical analyses are carried out and based on the behaviour of a large RC beam under monotonic loading.

Numerical results were studied in terms of structural global behaviour (Load as a function of displacement, for instance) but also in terms of local behaviour (cracking distribution). In the first part of the paper, the presentation of the experimental test is realised. The second one is devoted to the brief exhibition of the used constitutive material models. The third part shows the kind of considered modelling and the selected boundary conditions. Some comments have been done in the fourth part about the numerical and experimental results. And, to finish a conclusion and some outlooks are presented.

2 EXPERIMENTAL PROGRAM

The experimental results presented here come from the experimental campaign carried out in the framework of the CEOS.fr project on large beams subjected to bending test. This experimental work gave data on the cracking process in RC specimens, with local measure of the cracks propagation [2]. A complementary studied on the cracks pattern have been performed by Digital Image Correlation. The first results obtained with this study are provided in order to compare numerical local results and experimental results. The selected specimen is a large beam called RL1 subjected to a free shrinkage followed to a bending test. Main data for this specimen such as geometry, boundary conditions, loading conditions, reinforced distributions and also the material properties are provided by the benchmark ConCrack.
The length of the beam is equal to 6100 mm, its width and its thickness are equal to 1600 mm and 800 mm respectively (Figure 1). After casting, the beam has been let for 4 weeks to shrink freely.

The longitudinal beam reinforcement is consisted of two layers of 8 32 mm diameter steel rebars disposed horizontally on the top part of the beam, one layer of 8 25 mm diameter steel rebars disposed horizontally on the bottom part, two layers of 3 16 mm diameter steel rebars disposed vertically, one on the front part of the beam and the other on the behind one. RL1 reinforcement is also composed of 19 16 mm diameter stirrups associated with 16 mm diameter U surrounding the longitudinal steels rebars described previously (Figure 2). Steel and concrete mechanical properties of the RL1 beam are gathered in Table 1. The concrete cover is equal to 30 mm.

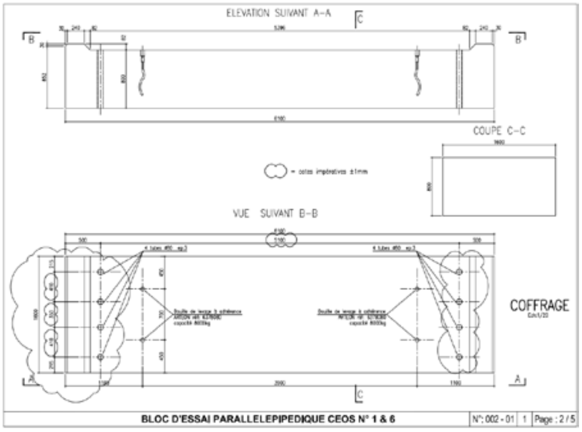

Figure 1: Geometry of the beam.

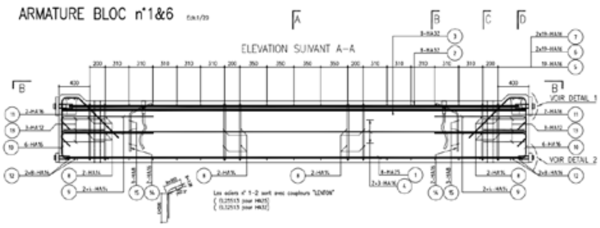

Figure 2: Layout of reinforced of the beam.

The loading system has been consisted in placing the beam on a bending bench. The beam is fixed onto the testing bench thanks to 4 macalloy steel bars crossing the testing bench and also the RL1 beam at each ends (Figure 3). The beam is submitted to a monotonic loading by two rows of 4 100 t jacks spaced of 1600 mm into the central part of the beam (Figures 3 and 4), and then adjusted to lay on the bench, using the macalloy steel bars. This first step implies that the boundary conditions are complex to reproduce exactly in numerical simulation.

Various sensors were placed in the beam and on its facings in order to obtain sufficiently data (displacement field as well as a cracking distribution).

Table 1: Constitutive material properties

	E_b	ν_b	F_{cb}	F_{tb}
Concrete	40.20 GPa	0.19	63.7 MPa	4.65 MPa
	E_a	ν_a	E_l	
Steel	200 GPa	0.3	500 MPa	

Where E_b is the concrete Young modulus, ν_b, the concrete Poisson coefficient, F_{cb}, the concrete compressive strength, F_{tb}, the concrete tensile strength, E_a, the steel Young modulus, ν_a, the steel Poisson coefficient and E_l the steel elastic stress limit.

3 PRESENTATION OF THE USED MODELS

Since the study took place in the framework of a benchmark, the purpose is to perform a numerical modelling of the RC specimen test with available numerical tools. Several constitutive laws are used. For the sake of simplicity, the used models will not be described in details. The finite element software used is Cast3M which is the software developed by the CEA (Atomic energy and alternative energies commission).

Testing bench with mock up

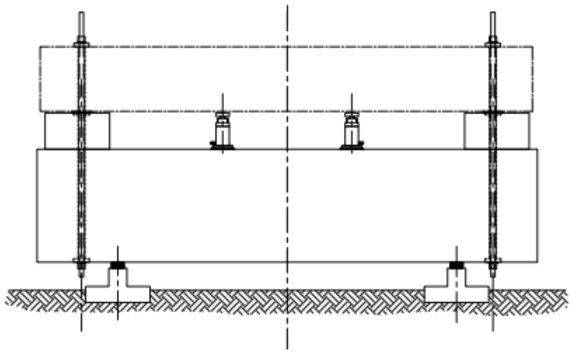

Figure 3: Loading setup of the beam.

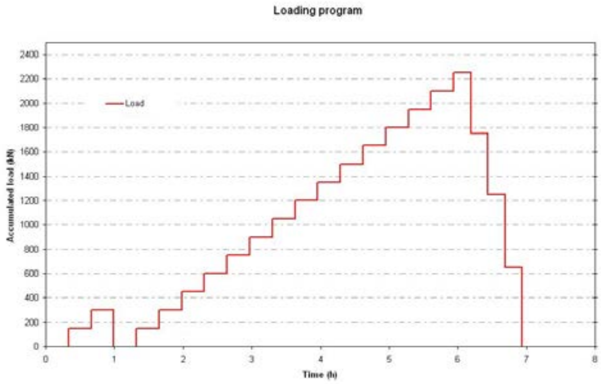

Figure 4: Monotonic loading applied on the beam.

3.1 Concrete behavior

Two concrete models have been used. These models are based on the damage continuum mechanics. The first adopted model is the classical Mazars model [3]. It is an isotropic damage model, which considers the dissymmetry between tension and compression responses. It is considered as a reference model in the concrete modelling by reason of its simplicity and robustness, and it allows simulation of complete large structures without convergence problems.
The second used concrete model is the Ricrag model developed at the French institute, Ifsttar. The latter considers elasticity, isotropic damage, internal sliding [4]. It allows taking into account some main well-known phenomena: dissymmetry between tension and compression responses, permanent strains, partial unilateral effect and also, hysteretic effects in cyclic loadings due to the occurring of friction between the crack lips. This model proved its efficiency to predict damage zone and the associated crack propagation [10].
Besides, in order to get a mesh independency, regularization technique has been adopted in both cases [6].

3.2 Steel behaviour

The steel rebars are modelled thanks to a classical elasto-plastic law with hardening proposed in the finite element software, Cast3M.

4 MODELLING OF THE BEAM

In the field of numerical modelling of cracking of concrete structures, almost all the approaches fall into main families of models: models addressing explicitly the propagation of one or several cracks within the structure (discrete approaches), models that consider cracks through a distributed damage and do not take into account cracks explicitly. The numerical modelling developed here tries to find simple post-treatment of damage modelling to obtained discrete information from damage field. The accuracy of the numerical simulation depends on the used constitutive material models, but also the boundary conditions too.

Because of the specificity of the beam, previous rough calculation showed that no slip between rebars and concrete occurred. Then a perfect steel/concrete bonding can be used. Moreover, a 2D model aims at predicting correctly the behaviour. This experimental test could be considered as a 4 point bending test but because of a flexure limitation (due to the macalloy bars which bring prestressing and the contribution of the testing bench), several boundary conditions are performed. The objective of this study is to find the most representative boundary conditions to reproduce the real bending test (Figures 3, 4). In this case, two models and four boundary conditions have been tested BC11, BC12, BC13 and BC14 (Figure 5). The second modelling is more complex and tried to be as close as possible to the experimental setup, with including the external bench (Figure 6). And for this modelling, two boundary conditions are tested, BC21 and BC22.

For both modelling, a two-dimensional finite element modelling in plane stress has been realized for the whole beam. The main steel rebars which are HA32 and HA25 rebars are considered as well as some steel rebars (located at the beam’s ends and in front of and behind the beam), the U and stirrups and the macalloy steel rebars. Both platens located below the central part of the beam are loading platens.

A schematic representation of the first modelling of the RC beam RL1 is given in Figure 5. And the one of the second modelling is given in Figure 6.

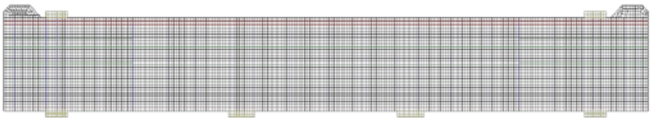

Figure 5: Finite element mesh used in the first modelling. Concrete is represented in black, steel reinforcement bars in red and green, U and stirrups in blue, macalloy steel rebars in violet and the loading and support platens in light yellow.

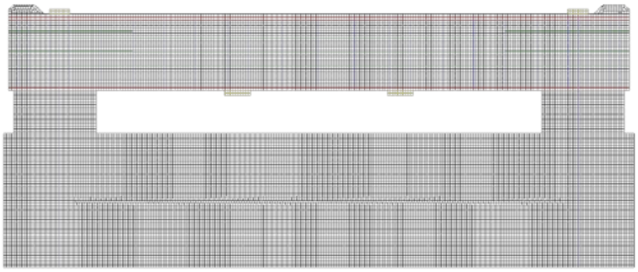

Figure 6: Finite element mesh used in the second modelling. Concrete is represented in black, steel reinforcement bars in red and green, U and stirrups in blue, macalloy steel rebars in violet and the loading and support platens in yellow.

The load is displacement controlled in order to provide a numerical robustness.

The four boundary conditions from the first modelling are gathered in Figure 7. And both of the second modelling are shown in Figure 8. Regarding the second modelling, the vertical and horizontal displacements of the base of the testing bench are not allowed to move.

The identification of material parameters associated to the concrete and steel constitutive laws are not detailed in this paper. For the steel, it is a classic elastoplastic model including hardening. And for concrete, two models are used Mazars and Ricrag models. To identify the material parameters of both concrete laws, the recommendations of the latter have been used in the present work [3, 4]. And the material parameters for the steel are chosen according to the steel experimental characteristics.

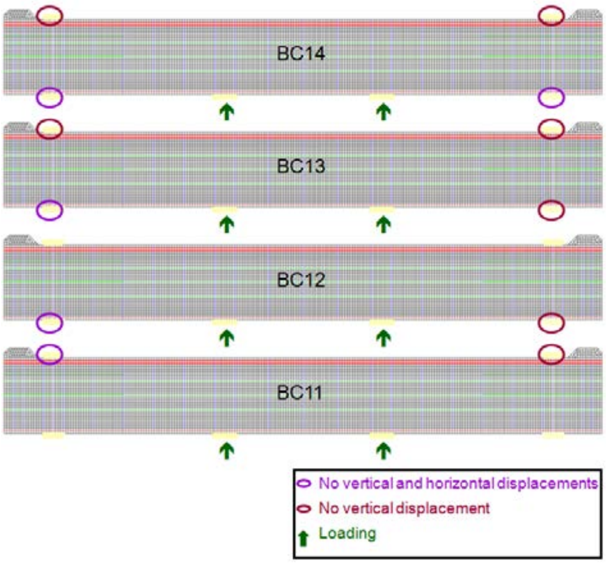

Figure 7: Boundary conditions for the first modelling.

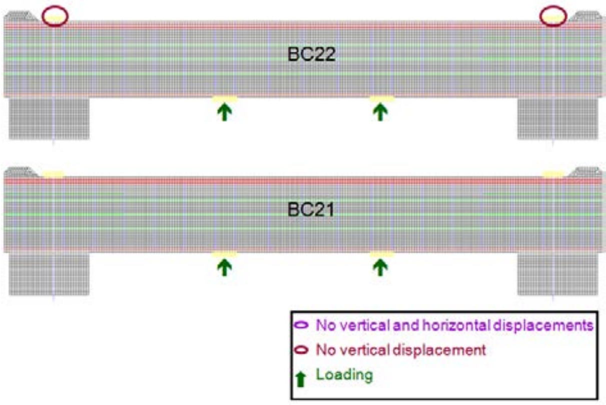

Figure 8: Boundary conditions for the second modelling.

5 RESULTS ANS COMMENTS

As seen previously, two modelling have been realized. In accordance with the modelling, several calculations were performed in order to evaluate the boundary condition effects on the mechanical behaviour of the beam. The numerical results of the tested RC beam are obtained and compared to the experimental one.

5.1 Displacement response

To compare numerical results to experimental data, a number of displacement sensors are used (Figure 9 and Table 2).

Having a 2D modelling, the x direction cannot be taken into account. Consequently, the analysis did not perform on the point P12. For symmetrical reason and our interest for the central part of the beam, the analysis has been performed only on the points P7 and P9 [7]. The CEOS.fr team is performing a complete treatment and analysis of the experimental data. Validated data will be published in January 2013.

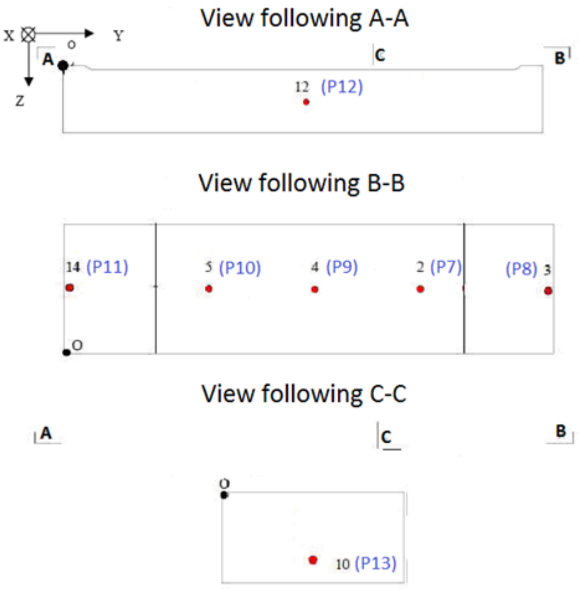

Figure 9: Position of the displacement sensors.

Table 2: Coordonates of displacement sensors

Number	Position (mm)			Direction
	x	y	z	
P7	800	4035	82	z
P8	800	5430	82	z
P9	800	2880	82	z
P10	800	1745	82	z
P11	800	400	82	z
P12	0	3050	400	x
P13	760	6100	485	y

Figures 10 and 11 represent the obtained numerical load/displacement curves and the experimental ones.

From Figures 10 and 11, the first remark is that all the boundary conditions whatever the modelling except BC22 boundary conditions shows nearly the same trend. Second, by focusing on the initial stiffness, the experimental one is higher than the obtained numerical initial stiffness. Nevertheless, with the boundary conditions BC22, the initial stiffness is still the closest to the experimental one. Third, during the cracking stage, numerical and experimental curves are not so close, while with the boundary conditions BC22, the numerical results are satisfactory. Four, by comparing Mazars and Ricrag models, a sudden decrease of the carrying capacity appears for Ricrag model which is not the case with Mazars model. Obviously, Ricrag law due to its complexity can model sudden crack propagation. At last, the simulation performed with the boundary conditions BC22 is the closest to the experiment. We can conclude that if 2D model is sufficient to reproduce the global behaviour, it is important to take into account representative boundary conditions, including the bench in this particular case, to predict a realistic global behaviour.

Moreover, the boundary condition choice is not sufficient to ensure a correct simulation. And, it is obvious that the structural response is influenced by the kind of selected modelling as well as how boundary conditions are taken into account.

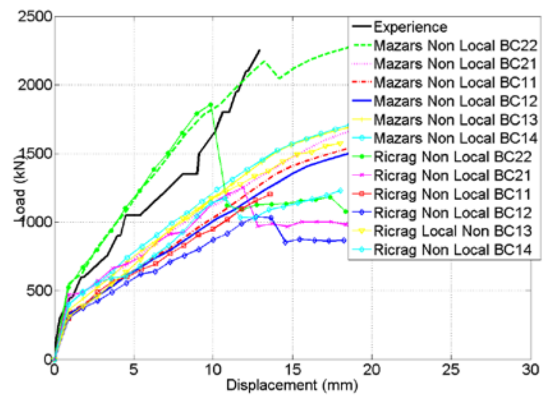

Figure 10: Load /displacement curves obtained at P7.

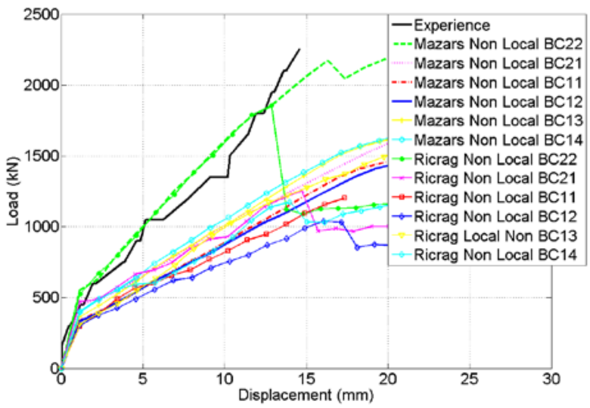

Figure 11: Load/displacement curves obtained at P9

5.3 Damage patterns and crack profiles

Damage patterns obtained for mid span displacement equal to 14 mm (central part of the beam) are shown in Figures 12 and 13. A representation of the degradation kinematics is given for Mazars and Ricrag models. Although this representation is related to a continuum damage model, cracks localization can be roughly observed. We can note that these boundary conditions are not perfect and/or damage models not so representative, as the damage profile show a higher influence on shear stresses than in the experiment.

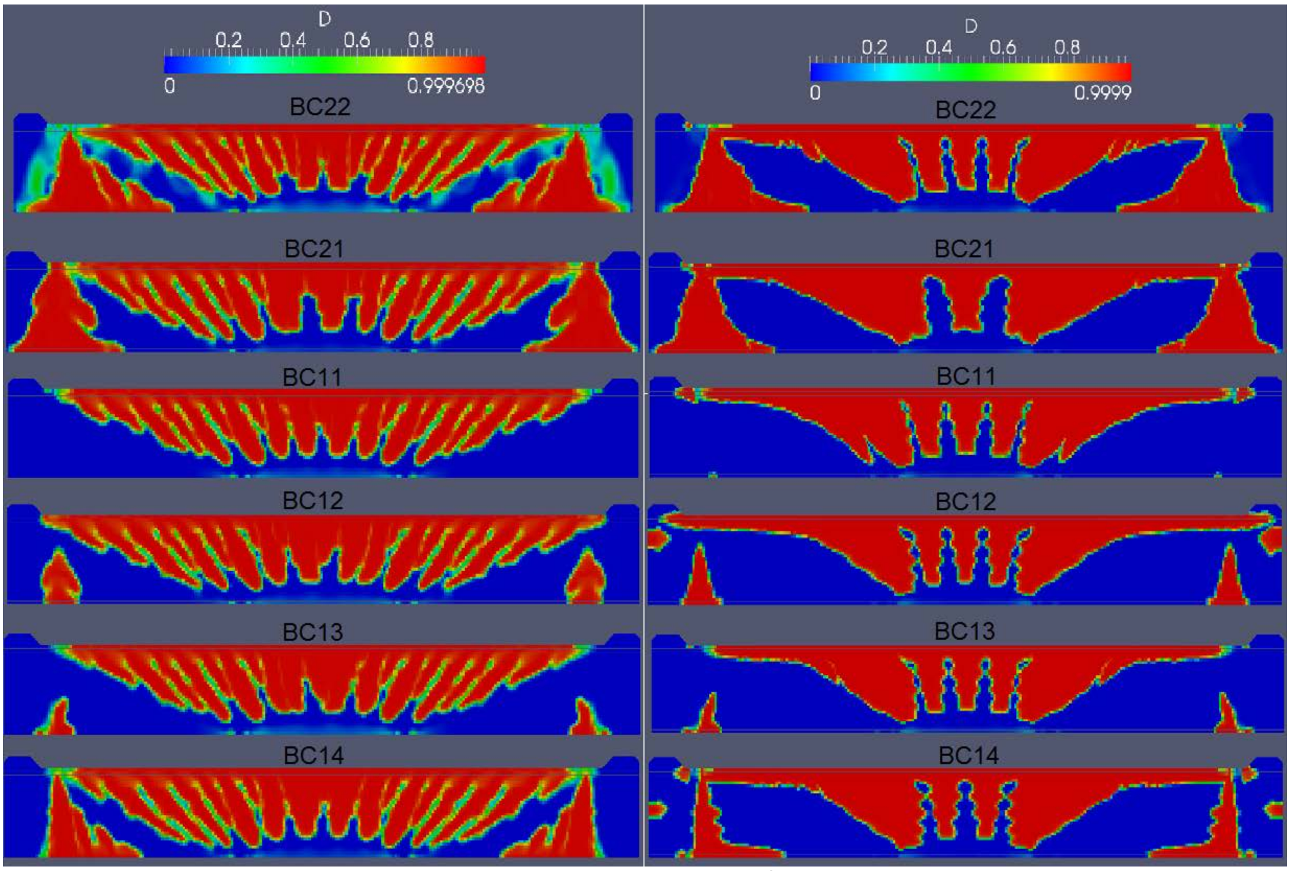

Figure 12: Damage pattern with Mazars and Ricrag models obtained for 14 mm mid span displacements with BC22, BC21, BC11, BC12, BC13 and BC14 boundary conditions.

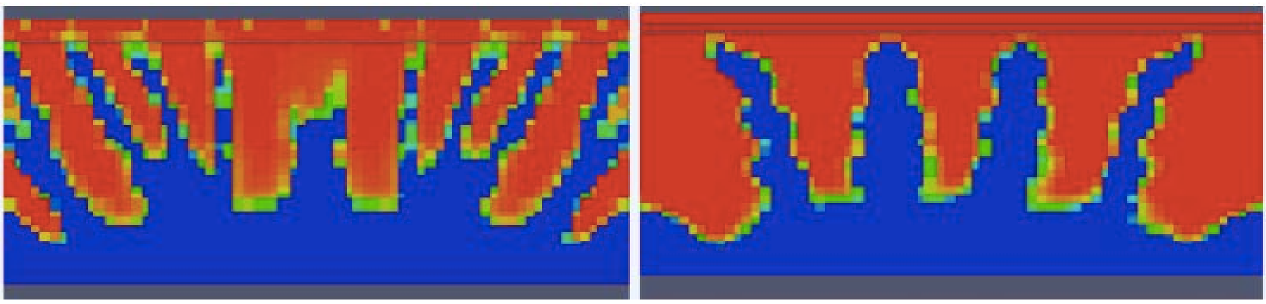

Figure 13: Damage patterns in the central part of the beam (1600 mm) obtained with Mazars (left) and Ricrag (right) models obtained for 14 mm mid span displacements with BC22 boundary conditions.

The damage pattern is different according to the concrete model used. As far as Mazars model is concerned it is not easy to distinguish clearly cracks in the central part of the beam, which is not the case with Ricrag model, whatever the boundary condition used. This is due to crack openings that are more important in Mazars case than in Ricrag one, which spread the damage along the central part of the beam. Using Ricrag model, in the central part of the beam, the number of main cracks seems to be the same for the first modelling with the boundary conditions BC11, BC12 and BC13 and for the second modelling with the boundary condition BC22. That implies that it is possible to get a damage patterns almost similar while the quantitative results are different with this concrete model (Figures 10 and 11). We can conclude that regarding the global force – displacement curves and damage patterns, BC22 are the best boundary conditions to take into account the effect of the macalloy bars which bring prestressing at the bottom of the experimental bench. For the further analysis, only this boundary condition (BC22) is considered.

For these load case that leads a multiple crack pattern, an estimation of the crack opening and spacing can be performed. The pattern obtained by both concrete models is analysing in the central upper part in tension of the beam for BC22 conditions (Figure 13). Two approaches are performed, one using local post-treatment and one using the global cumulative displacement of the central part of the beam.

An estimation of crack opening and spacing can be performed considering the Cast3M procedure developed by C. LaBorderie and M. Matallah. Detailed explanation can be found in [5]. Due to the continuous nature of the constitutive laws used, taking into account a kinematic discontinuity seems difficult. The multiple cracks observed are numerically represented by a continuum damage that is not entirely concentrated at the crack location. Because of the multiple cracks, a direct post-treatment is not accurate enough to represent quantitatively the crack opening in this particular case (Figures 14 and 15). New advanced model, using accurate regularization procedure [6] has been studied in the frame of the Mefisto project but they are still in development to deal with multiple cracks problems [9].

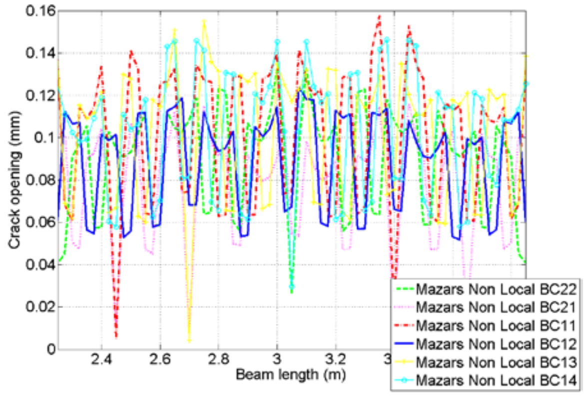

Figure 14: Cracks opening according to Mazars model along the horizontal top line (at y = 800 mm).

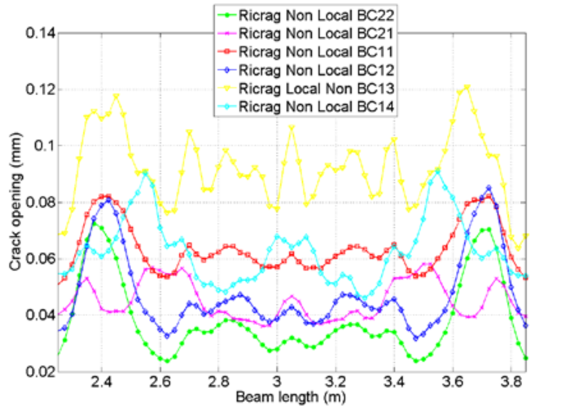

Figure 15: Cracks opening according to Ricrag model along the horizontal top line (at y = 800 mm).

Moreover, an estimation of the crack spacing and the crack opening can be done, using the crack pattern obtained numerically in the central part of the beam. At the maximum load (Figure 13) the cumulative displacement of the upper part of the beam (tensile zone, y = 800 mm) in the central zone can be simply estimated. For the Mazars model, the corresponding cumulative displacement is measured at 9.52 mm, so for the 9 main cracks obtained in the simulation (Figure 13), the mean opening is 1.057 mm and the mean spacing is 1800 mm. For the Ricrag model (Figure 13), the corresponding cumulative displacement is 5.59 mm, so for 5 main cracks obtained in the simulation the mean opening is 1.12 mm and the mean spacing is 320 mm.

From experimental data (Figures 16, 17), 10 cracks are obtained in the central part (1600 mm).

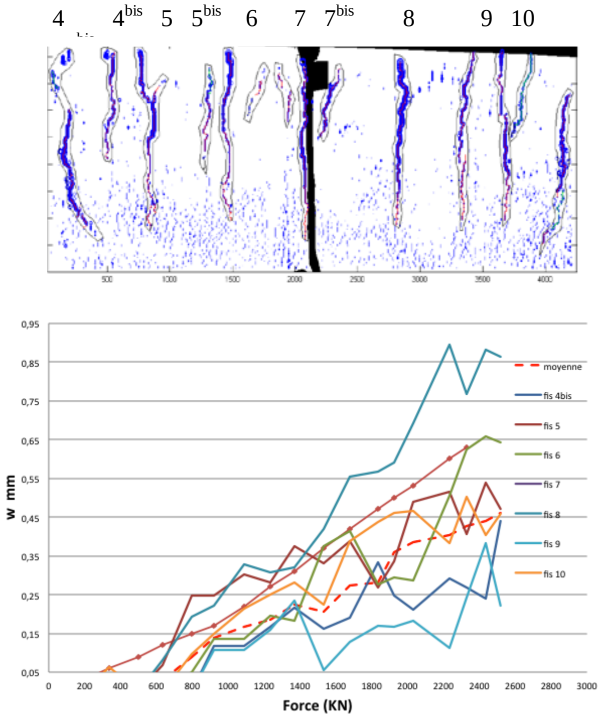

Figure 16: Experimental crack distribution (crack openings) for a 2500 kN loading along 1800 mm in the central part of the beam, digital image correlation (DIC).

At the maximum load, the measured crack opening is 0.45 mm for the mean value with a crack (situated in the middle of the beam) that reaches a width of 0.85 mm. The measured crack spacing is 200 mm (mean value) with a maximum of 450 mm.

One should noticed that the number of cracks obtained with Mazars model varies between 10 and 13 cracks approximately according to the way to count them, and to include or not shear cracks and with Ricrag model between 5 and 9 approximately. These results could be strongly improved using a better regularization technique. That implies that with Mazars model the average crack spacing is about 180 mm while with Ricrag model the average crack spacing is about 200 mm (result with 8 cracks). These results have to be compared with the experimental data of 0.20 m. As far as crack spacing is concerned, both concrete models are able to predict with a good accordance cracks spacing observed on this RL1 beam.

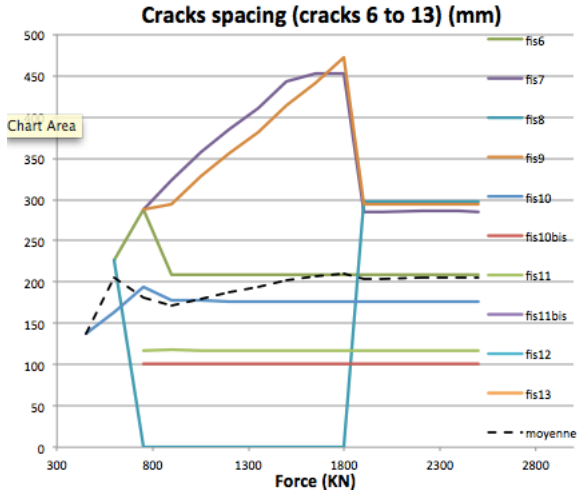

Figure 17: Cracks spacing estimated by experimental data from digital image correlation, for a 2500 kN loading along 1800 mm in the central part of the beam.

While experimentally, an average crack opening of 0.45 mm is obtained with a maximum of 0.85 mm, the simulations reach more than 1 mm for both models: 1.057 mm for Mazars and 1.12 mm for Ricrag model. Whatever the concrete model used, the numerical crack opening is well predicted, with a good accordance and reasonable safety coefficient if we consider that it is the maximum crack opening that govern the durability of the structure. But, on the other hand, if we are interested by the mean value for both models, the result is twice more that the experimental measure.

Given the Figures 10-13, it may be seen that the numerical crack opening and spacing depend on the modelling, the boundary conditions and also the kind of concrete model used to represent the concrete behaviour. But, using these robust concrete models, with a simple post-treatment analysis, using the cumulative displacement, it is possible to predict with good accordance the local data, like crack spacing and maximum crack opening.

6 CONCLUSIONS

In this paper, a finite element analysis has been performed on a large beam that was loaded thanks to a bending test. The experimental testing aimed at quantifying the durability and serviceability of large beam when subjected to a mechanical load. In this study, one focused on the improvement of the modelling of the bending test, especially the effects of the boundary conditions and the ability of two concrete models to simulate the structural behaviour correctly. Two 2D modelling were performed. The first one is simple and affected by 4 boundary conditions. The second one is more complex and 2 boundary conditions are considered.

This analysis aimed at describing the influence of the modelling on the quantitative and qualitative response of the beam: global behaviour and local behaviour (crack pattern).

The damage pattern (cracks opening and spacing) evolution pointed out the differences between the concrete models used for the same modelling and boundary conditions. This difference is not so considerable in terms of load/displacement curve. Although, the number of crack is different for each concrete model, the spacing between the central cracks is correctly simulated and the cumulative displacement along the beam insures a correct force – displacement curve. Widths of the cracks are different for each concrete model but the predicted values are over the experimental measures. For engineering purpose, the simulation gives interesting values, considering the durability as the major phenomenon, with a good prevision of the largest crack.

This present paper highlights the need to use a concrete model able to consider non-linearity of concrete behaviour, an appropriate modelling and suitable boundary conditions in order to simulate the behaviour of a structure and depict its damage pattern and crack pattern correctly. But as continuum damage models spread the damage along the beam, only good range of order of local crack width can be obtained with this modelling. For accurate data, discrete models should be used in addition to the continuum approach, or accurate regularization technique must be developing in order to concentrate damage along the cracks.

ACKNOWLEDGMENTS

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REFERENCES

[1] CEOS.fr, French National Research program, Comportement et Evaluation des Ouvrages Spéciaux, Fissuration – Retrait (*Behaviour and Assessment of special reinforced concrete works – cracking & shrinkage)*, www.ceosfr.org, 2009-2013.

[2] Buffo-Lacarriere L., Rospars C., Delaplace A., Duong A.V., Jason, L., 2011, International benchmark ConCrack, Synthesis of the results - *Proceeding of the 2nd workshop on Control of Cracking in reinforced concrete structures (ConCrack 2),* June 20-22, 2011**,** Paris.

[3] J. Mazars, 1984. *Application de la mécanique de l'endommagement au comportement non linéaire du béton de structure*, PhD Thesis, Ecole Normale Supérieure de Cachan.

[4] B. Richard, F. Ragueneau, C. Cremona, L. Adelaide and J.L. Tailhan, 2010. Isotropic continuum damage mechanics for concrete under cyclic loading : stiffness recovery, inelastic strains and frictional sliding. *Engineering Fracture Mechanics* **77**:1203-1223.

[5] Matallah, M., La Borderie, C. and Maurel, O., 2010. A practical method to estimate crack openings in concrete. *International Journal for Numerical and Analytical Methods in Geomechanics*, 34:15:1615-1633.

[6] Pijaudier-Cabot, G. and Bazant, Z.P., 1987. Non-local damage theory. *Journal of Engineering Mechanics ASCE*, 113:1512-1533.

[7] Jason, L., 2011, Synthesis of the results related to RL1 large beam, *2nd workshop on Control of Cracking in reinforced concrete structures (Concrack 2),* June 20-22, 2011**,** Paris, France.

[8] Eurocode 2, *calcul de structure en béton*, norme Européenne, norme Française, AFNOR, 2005.

[9] C. Giry, F. Dufour and J. Mazars, 2011. Modified nonlocal damage model based on stress state influence. *International Journal of Solids and Structures* **48**:3431-3443.

[10] B. Richard, S. Epaillard, C. Cremona, L. Elfgren and L. Adelaide, 2010.
Non linear finite element analysis of a 50 years old reinforced concrete trough bridge, *Engineering Structures*, **32**:**12**:3899-3910.