MECHANICAL DAMAGE AND FRACTURE OF CONCRETE STRUCTURES

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ABSTRACT

In order to express the progressive deterioration caused by microcracks in concrete under mechanical solicitations, we used the coupling damage-elasticity and introduced a "damage criterion" in terms of strains.

Such concepts have been inserted in the elaboration of a finite element computation program. The results of tests on bending plain concrete and reinforced concrete beams have justified the calculation method used, for it shows a good concordance as far as the local and the global behaviour of the structures are concerned.

KEYWORDS

Behaviour of concrete - Microcracking - Mechanical damage - Fracture

I. INTRODUCTION

The behaviour and fracture of concrete under mechanical solicitations are the consequences of the existence and development of microcracks.
The direct observation of those phenomena by means of methods such as the X rays study of loaded samples or the micro examination of solicited test specimens after cutting (Slate, Olsefski, 1963; Dhir, Shanga, 1974) enables us to conclude that :

- the micro-cracks and micro-voids exist prior to any loading and are linked to the phenomenon of hydration (skrinkage)
- micro-cracks develop preferentially around the larger aggregate particles (bond cracks), then in the matrix
- they develop mostly perpendicularly to the directions of maximum extension.

The latter fact is confirmed by the analysis of wave propagation speed (Benouniche, 1980) and the observation of fracture in uniaxial (Hsu and others, 1963) or biaxial (Kupfer, Hilsdorf, 1969) compression tests, such fracture deriving from the developping of cracks parallel to stress directions.
Thus the study and prevision of the behaviour of concrete consists mostly in modellizing the gradual deterioration of the material and this can be realized if we use such theories as the mechanical damage : it will be the subject of the present

paper in which we will especially be interested in the cases where tensions prevail

II. DAMAGE MECHANICS

The theory has been proposed as early as 1958 by Katchanov and Rabotnov in order to explain the fracture in creeping metals and it has brought about recent developments (Lemaitre, Chaboche, 1978), more particularly with the elaboration of a tridimensional theory which couples damage and elasticity (Cordebois, Sidoroff, 1979).

Deriving from the thermodynamics of irreversible processes, it makes use of the notion of a damage tensor linked to the state of stresses or strains.

The method is interesting because it enables us to express the anisotropic evolution of the deterioration but it is attended with a few difficulties of application in the cases of complex solicitations.
A recent application of the theory to concrete under axisymetrical solicitations (Benouniche, 1980) shows that this way of research is full of promise.
Particularly as far as an isotropic evolution of deterioration is concerned, damage is formulated through a scalar D bearing up on the elasticity modulus E and thus expresses a variation of stiffness of the material $E = E_o\ (1 - D)$
with D = O for the intact material
D = 1 at fracture point

At that stage two problems still remain to be solved
- the damage limit from which an evolution of D will start
- the law of evolution beyond that limit

III. BEHAVIOUR OF CONCRETE

In the most general case the behaviour of concrete is a combination of phenomena such as linear elasticity, viscosity, microslidings (giving birth to permanent strains) and microcrackings (entailing a lower stiffness of the material).
The relative importance of each phenomenon depends on the type of solicitation. The analysis of the results given by uniaxial traction tests with an after peak phase (Terrien, 1980) (fig. 1) or by crack propagation tests on DCB beams (Chhuy Sok and others, 1980) show that :

- elasticity prevails in the first phase
- as far as uniaxial traction is concerned, there is a sudden fall of stresses beyond the peak and a quite perceptible variation of stiffness, due to a fast development of microcracks.
- as far as crack propagation is concerned, an important extension of the damaged zone beyond the visible part of the crack (observation by means of the localisation of acoustic emissions) and a lowering of stiffness which, (in the absence of secondary phenomena below N Level, see fig. 2) allow us to think that it is not accompanied by important permanent strains.

IV. MECHANICAL DAMAGE OF CONCRETE

1) <u>Uniaxial case</u>
From the preceding results we can set a model of the behaviour of the material by using the coupling of elasticity and damage, the damage limit ε_D corresponding to the peak of the curve (fig. 3).

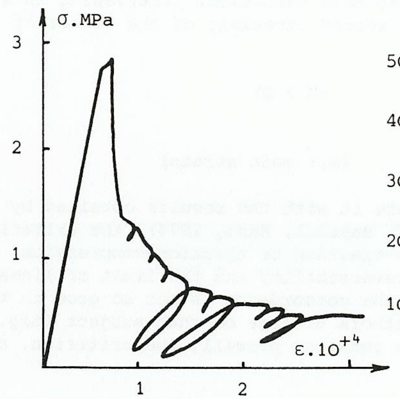


Fig. 1 - *Behaviour of concrete in direct tension*

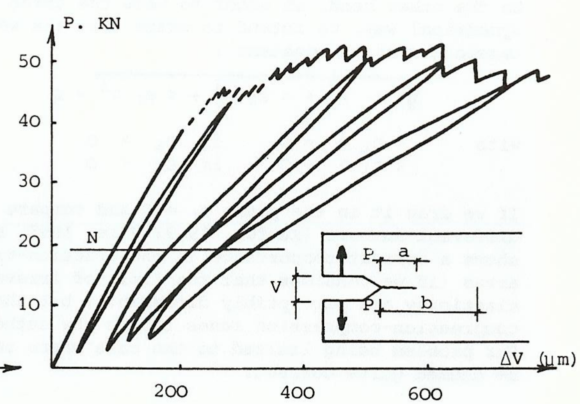


Fig. 2 - *Test on a concrete double cantilever beam*
a - visible crack
b - damaged zone

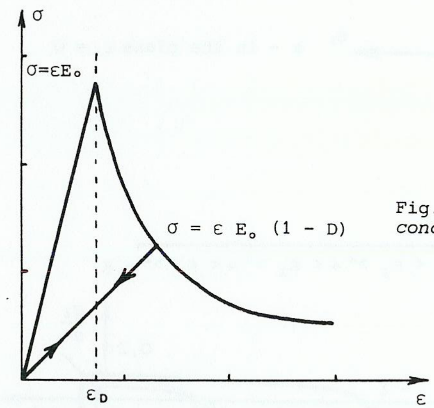


Fig. 3 - *Behaviour in direct tension, with the concept of damage (phenomenon idealized)*

The evolution of damage depends on the maximum strain reached and we propose the following law (resulting from uniaxial traction curves) :

$$D = 0 \qquad \text{if } \varepsilon \leqslant \varepsilon_D$$

$$D = 1 - \frac{\varepsilon_D\ (1-A)}{\varepsilon} - \frac{A}{\exp\left(B(\varepsilon-\varepsilon_D)\right)} \qquad \text{if } \varepsilon > \varepsilon_D$$

A and B being constants characterizing the material

2) <u>Extension to tri-dimensionnal problems</u>
- Damage criterion

As we have seen before, the criterion must express the fact that tensions ($\varepsilon > 0$) produce damage. From the strain tensor formulated in the main directions, we can define the tensor of "tensions" by keeping positive terms only and reducing negative terms to zero.

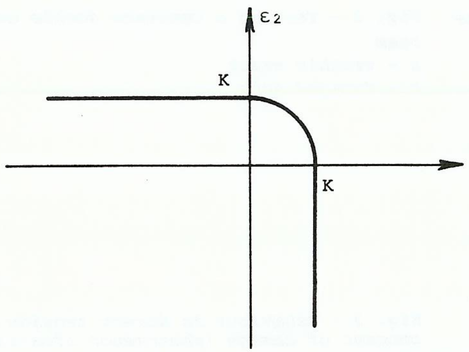
On the other hand, in order to have the three main directions intervening in a symetrical way, we intend to write that the second invariant of the tensor of tensions remains constant :

$$\sqrt{< \varepsilon_1 >^2 + < \varepsilon_2 >^2 + < \varepsilon_3 >^2} = K \qquad (K > 0)$$

with $< \varepsilon_i > \; = \varepsilon_i$ if $\varepsilon_i > 0$

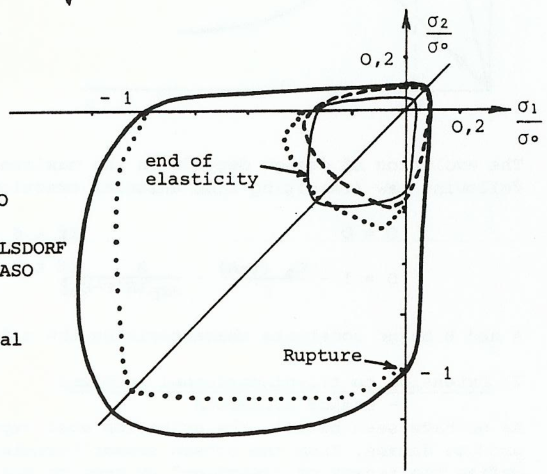
$< \varepsilon_i > \; = 0$ if $\varepsilon_i < 0$ (ε_i: main strain)

If we draw it in the plane $\sigma_3 = 0$ and compare it with the results obtained by different authors (Kupfer, 1969; Vile, 1965; Bascoul, Maso, 1974), the criterion shows a correct concordance in the traction-traction or traction-compression areas (if we consider that the limit of irreversibility and the limit of linear elasticity are perceptibly different); but the concordance is not so good in the compression-compression zones though the authors diverge on that subject (fig. 4). Our problem being limited to the case where tensions prevail, the criterion, can be deemed quite correct.



a - in the plane $\varepsilon_j = 0$

Fig. 4 - *Criterion* $\sqrt{< \varepsilon_1 >^2 + < \varepsilon_2 >^2 + < \varepsilon_3 >^2} = K$



b - in the plane $\sigma_3 = 0$

— tests from KUPFER-HILSDORF
.... tests from BASCOUL-MASO
--- criterion

(σ_o = strength in uniaxial compression)

- Tridimensional problems

The surface of damage limit will be thus defined in the strain space by :

$$f = J - K = 0 \text{ with } J = \sqrt{< \varepsilon_1 >^2 + < \varepsilon_2 >^2 + < \varepsilon_3 >^2}$$

and its evolution :

* if $f = 0$ and $\frac{\partial f}{\partial \varepsilon} \cdot \dot{\varepsilon} \geq 0$

$$\begin{cases} \dot{K} = \dot{J} \\ \dot{\sigma} = (1 - D)\, A_0\, \dot{\varepsilon} \\ \dot{D} = \dot{J} \left[\frac{K(1-A)}{J^2} - \frac{AB}{e^{B(J-K)}} \right] \end{cases}$$

* if $f = 0$ and $\frac{\partial f}{\partial \varepsilon} \cdot \dot{\varepsilon} < 0$

$$\begin{cases} \dot{K} = 0 \\ \dot{\sigma} = (1 - D)\, A_0\, \varepsilon \\ \dot{D} = 0 \end{cases}$$

A_0 being the elastic matrix of the intact material and $\dot{D} = \frac{\partial D}{\partial t}$

V. DAMAGE AND CALCULATION OF STRUCTURES

In a first stage we got interested by plane problems in concrete and reinforced concrete structures.
To answer that purpose we elaborated a computation program making use of the finite element method with variable stiffness.
The elements used are three node triangles : they enclose a constant strain field and consequently a constant damage. The solicitation is applied through imposed displacement.

The principle consists in determining $J = \sqrt{<\varepsilon_1>^2 + <\varepsilon_2>^2}$ for every element and comparing it with K (damage limit) and thus making the damage progress or not. According to that result, we modified the stiffness matrix with $E = E_0\ (1-D)$, a convergence criterion based on the evolution of strains between two iterations deciding whether the process was to be carried on or not.

As far as reinforced concrete is concerned the bonding concrete-reinforcement was, in a first stage considered as perfect.

VI. PERFORMANCES OF THE COMPUTATION AND COMPARISON WITH THE RESULTS FROM BENDING BEAM TESTS

1) The experiment

Rectangular section plain concrete and reinforced concrete test beams were solicited in three point or four point bending (fig. 5).
The beams were of a sufficient size (150 x 220 x 1600 mm) compared with the size of the biggest aggregates (10 mm).
The concrete used answered the following weight composition :

Cement CPJ 45
Sand 0 - 4 } silico-calcareous
Gravel 4 - 10
Water

The tests were effected approximately on the 28 th day after the concrete had been cast and the solicitations were exerted with prescribed displacement (0,02 mm/mn).
Strain gages were used to follow the evolution of the local behaviour, especially in the inferior part (tensed areas) and on the reinforcement (reinforced concrete).
Besides, the measure of the vertical displacement in the middle of the beam enabled us to characterize the general behaviour of the structure.

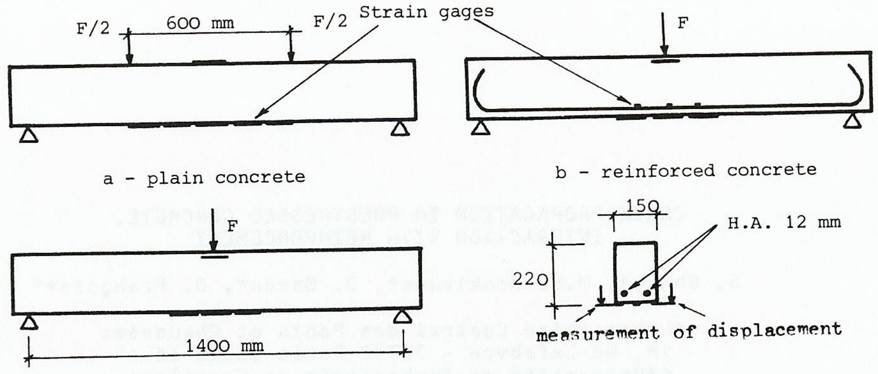


Fig. 5 - *The three types of experimental beams*

2) Results

The calculation of the test beams makes use of symetry and the cutting out of the beam into finite elements consists of 481 modes and 864 triangles.
The results obtained concern 3 types of tests :

- bending tests on plain concrete : 4 points
- bending tests on plain concrete : 3 points
- bending tests on reinforced concrete : 3 points

The identification of a number of tests first enabled us to determine the values of coefficients A and B characteristic of the material according to the damage law,(here, $A = 0.8$; $B = 2.10^4$)and the elastic parameters of the intact material ($E_o = 30\ 000$ MPa, $\nu = 0{,}2$)
Yet according to the kind of test and in order to find the same characteristics of general behaviour in the different structures, we had to adapt the value of the damage limit parameter K :

Plain concrete : four point bending : $K = 0.670\ 10^{-4}$
three point bending : $K = 0.718\ 10^{-4}$
Reinforced concrete : three point bending : $K = 1.150\ 10^{-4}$

Thus verifying conclusions reported by other authors (L'Hermite, 1973) as to the influence of the conditions of solicitation on the strength of concrete.
The results we give on the following page show a good concordance as far as the global behaviours are concerned (effort-displacement) as well as to the local behaviours (effort-strain).
From the analyses of those results, we can make the following acknowledgments :

- Fracture (or the opening of a crack in reinforced concrete) derives from a much localized accumulation of strains (fig.6-11) while the neighbouring areas draw in (fig.7-11).
- The local degradation of the material (in the tensed zones) has only little weight on the behaviour of the structure (plain concrete)
- The cracking phenomena in reinforced concrete are very complex but as the results from the tests and the computation remain comparable, we get comforted as to the use of it.
- The local behaviour is much influenced by the existence of defects which cause fracture at once (concrete in four point bending) or modify the behaviour for some time, yet do not cause fracture directly (three point bending).

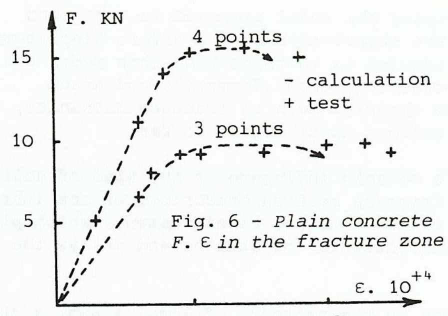


Fig. 6 - *Plain concrete*
F. ε in the fracture zone

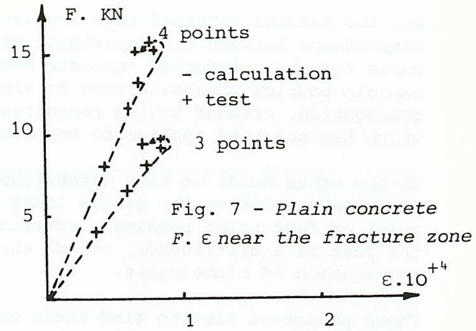


Fig. 7 - *Plain concrete*
F. ε near the fracture zone

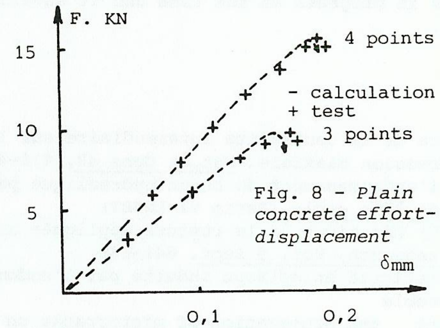


Fig. 8 - *Plain concrete effort-displacement*

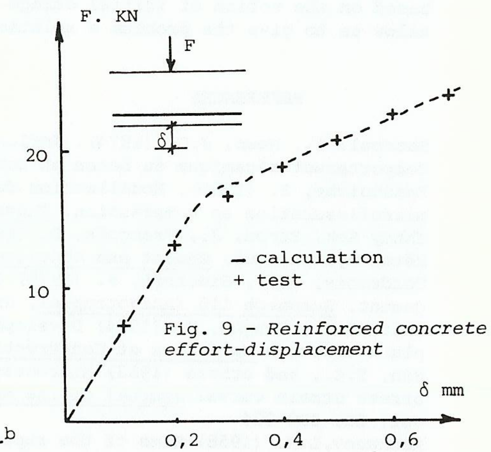


Fig. 9 - *Reinforced concrete effort-displacement*

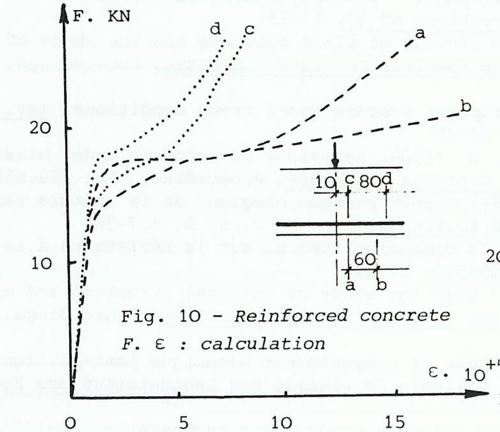


Fig. 10 - *Reinforced concrete*
F. ε : calculation

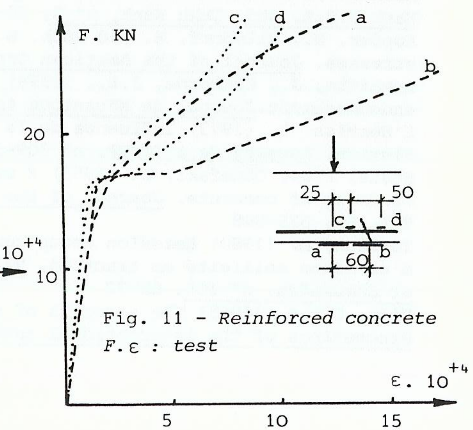


Fig. 11 - *Reinforced concrete*
F.ε : test

THE RESULTS

Comparison tests-calculation

VI. CONCLUSIONS

All the results obtained show how satisfying the model proposed is. The good concordance between the experiment and the computation on the effort-displacement curve for the reinforced concrete beam enables us to think that this model can explain complex phenomena such as microcracking, crack formation and crack propagation, without having rescourse to theories such as fracture mechanics, which has not been applied to concrete without difficulties so far.

On the other hand, we have established a notable influence of the kind of solicitation exerted on the damage limit : fracture section prescribed or not (three point or four point bending), existing or non existing reinforcement, which plays the part of a distributor, avoids the concentration of strains and delays the propagation of microcracks.

Those phenomena seem to find their origin in the presence of local flaws. A study based on the notion of initial damage is in progress at the time and it should allow us to give the problem a solution.

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