STUDY OF CONCRETE'S CRACKING UNDER MULTIAXIAL STRESSES

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ABSTRACT

The authors studied the mechanical behaviour of concrete subject to triaxial loading where the maximal principal stress σ_1 was superior or equal to the lateral pressure P ($\sigma_1 \geqslant P$). A special care was taken for the cracking of the material.

Tests were carried out using a servo-controlled hydraulic testing machine. The samples were made of micro-concrete.

The elastic domain, defined by the linearity of stress-strain curves, is a closed figure in the stresses diagram.

Studying simultaneously curves of deformations and acoustic emission led the authors to define a domain of "non microcracking" of the concrete. We note that in the brittle case the limit of elasticity and the limit of microcracking are quite close, but in the ductile case these two limits differ from each other more especially as the confining pressure increases.

Accordingly three domains are defined in the σ_1, P plane :

- elastic domain,
- non microcracking domain,
- domain of microcracking.

Finally, complementary tests of creep showed that the domain defined by the microcracking limit was "stable". Hence it seems that the domain of microcracking could be included in any calculation for the definition of a coefficient of security.

KEYWORDS

Concrete ; microcracking ; triaxial stress ; acoustic emission ; stable field ; safety factor ; creep.

AIM OF THE STUDY

The powerful means of calculation now at the disposal of engineers enables them to calculate more and more complex structures both in reinforced and prestressed concrete taking into consideration the behaviour of the material under pluriaxial stresses.

Although many authors, J. BERGUES and al. (1971), J. WASTIELS (1979), have gone into the subject over the past fifteen years or so, the rheological behaviour of concrete under triaxial stresses is as yet not well known and the greatest difficulty for the engineer is still to be found in the choice of a safety factor which takes into account all the types of loads which the structures being studied will have to undergo. At the moment, the fixing of a safety factor is empirical ; for example, in the case of the caissons of nuclear power-stations, the rules defining the safety field are generally only based on experimental results considering the ultimate strength of the concrete under an instantaneous load ; in the case of more complex loads, the problem is still ill-defined . Indeed, if the criterion of failure under instantaneous multiaxial sollicitations is beginning to be fairly well known, on the contrary, very little information can be found on its behaviour under multiple stresses in the case of loads over long periods or cyclic loads.

For the engineer, the risk of failure is tied to the existence of a point beyong which the possibilities are that the structure will be completely destroyed, whence the necessity arises of fixing this point starting from systematic laboratory tests taking into consideration all the types of loads which the structures being studied will have to undergo. It is clear that, seen in this light, the study of the rheological behaviour of concrete to fix a stability field may well be very long and tricky to undertake.

Now in rock mechanics, it is found that the appearance of cracking (or its development when it already exists) is often the cause of time dependent phenomena, P. BEREST and al. (1979), and under cyclic loads experience shows that the cracking point corresponds to the fatigue strength of the material, P. MORLIER (1971).

Thus these results and all the foregoing considerations have brought us to develop the study of concrete's cracking under multiaxial stresses, the cracking point being able to delimit the stable field of the material. As this is a vast problem, in the first stage we shall only consider the case of the classical triaxial compression load $\sigma_1 \geqslant \sigma_2 = \sigma_3 = P$ with

σ_1 axial stress,

P lateral stress.

The process of the study is as follows :

- fixing of the cracking point in the plane σ_1 , P ;
- study of stability in time and under repeated loads in the non-cracked field.

This article deals with the first point and touches on the second. An experimental technique is presented making it possible to detect the occurence of cracking. The results are then discussed.

GENERALITIES ON THE CRACKING AND FAILURE OF CONCRETE

Concrete, a composite material, is essentially made up of a binder (water + cement), of aggregates (sand + gravel) and voids (pores and shrinkage cracks). These inclusions and spaces embedded in the matrix constitute, for the material, defects which during the load will be the seat of concentrations of stresses likely to start microcracks which by spreading may progressively lead to the ruin of the material.

A qualitative interpretation of the behaviour of concrete can be given using a simple model containing a spherical pore or a spherical inclusion in an infinite mass. For a unidirectional compression load σ_1 in the case of the pore (figure 1) an elastic calculation shows that there appear traction stresses at the keystone and compression stresses on the equator. If the failure criterion of the matrix is a Coulomb criterion, it is shown that this criterion is passed first of all at the keystone giving rise to a crack. If the load is increased a crushing of the pore may also appear, and this may be comparable to a hardening of the material.

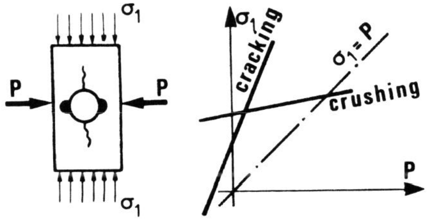

Figure 1
A model with spherical pore.

When the case of the model with inclusions is considered, the mechanisms are similar, but with a different distribution and intensity of local stresses, indeed the extensions capable of giving rise to microcracking are situated on the equator whereas the compressions are localised at the keystone (fig. 2).

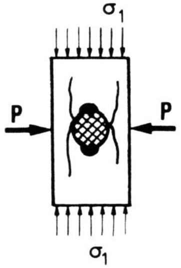

Figure 2
A model with spherical inclusion .

Still using the same models with the same criteria but adding a lateral stress P, elastic calculation shows that the effect of this pressure delays the beginning of cracking and on the contrary promotes the phenomena of crushing and in the case of very strong lateral stresses, cracking may appear very late.

The results obtained from many different types of concretes studied under triaxial sollicitations show that in general for low average stresses the material has a brittle-type behaviour, and that under the effect of a high confinement pressure a transition can be observed towards a ductile behaviour for which limit strength is reached for large strains. It should be noted that even for this last type of behaviour damage to the material is also caused by a considerable network of microcracks appearing well outside the elastic field.

The results show that the conditions of stabilities are different according to whether we are in the brittle field or the ductile field, but that in every case they are essentially dependent on the conditions of cracking.

EXPERIMENTAL TECHNIQUE

The samples, cylinders 36mm in diameter and 72mm long, cored in the mass, were tested at 4 months old after conservation in water. We used a micro-concrete of glass balls (Φ=5 mm) with a dosage of 400kg of cement to the cubic metre. Glass balls are used in the concrete to get closer from theorical model exposed in the uppon paragraph.

The tests were carried out using an installation made up of a 1000 KN press and a 100 MPa triaxial cell servo-controlled by a high performance electronic system capable of controlling simultaneously a parameter in the direction of σ_1 and another in the direction of P. The samples protected from the confinement fluid by an impervious casing, are equipped with strain gages to measure the strain in the two directions of the principal stresses. At the same time as these strain measurements we studied the acoustic emission produced during the loading using an ENDEVCO system (series 3000). The method of counts enabled to minimise the influence of hydraulic noises due to the servo-valves. A previous study completed on this same material, M. TERRIEN (1980), enabled us to fix the best frequency band to use for this type of tests (between 100 and 350 kHz). Figure 3 gives a complete layout of the test.

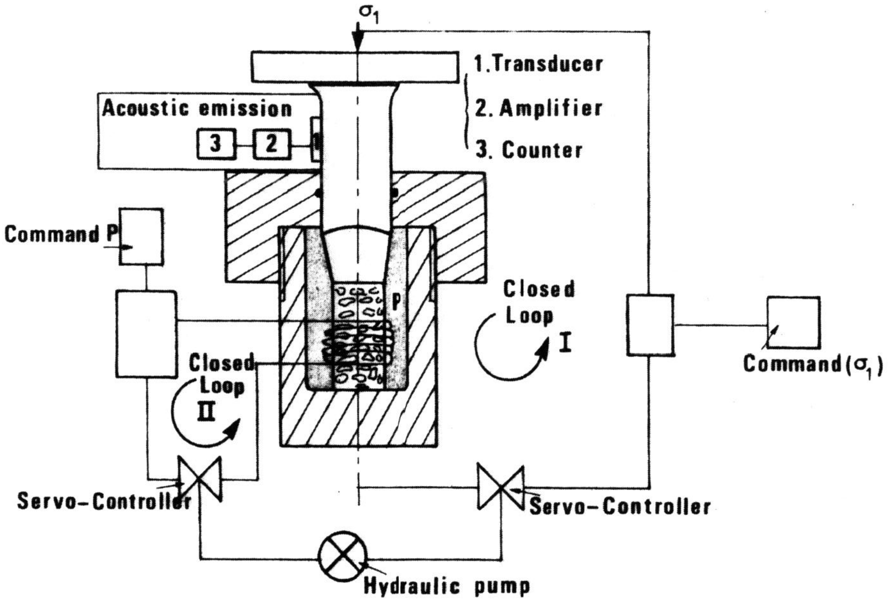

Figure 3 : Schematic diagram of a triaxial test system.

The samples were put under load using the classical course which is broken down into two parts (figure 4) :

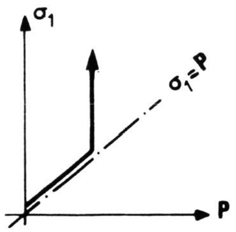

Figure 4

Loading path.

1. Increasing hydrostatic stress up to the desired lateral pressure :

   $$\sigma_1 = \sigma_2 = \sigma_3 = P.$$

2. Increasing axial stress σ_1 keeping a constant lateral pressure :

   $$\sigma_1 > \sigma_2 = \sigma_3 = P,$$

   during this step the load is applied with a constant rate of strain ($\varepsilon_1 = 10^{-5}$/s).
   ε_1 = axial strain following σ_1 .
   $\varepsilon_2 = \varepsilon_3$=lateral strain following P .

DISCUSSION

The specimens were tested with five levels of hydrostatic pressure between 0 and 65 MPa. The results are summerized in figures 5 and 6.

To study the cracking of concrete in more detail we first of all analysed the strains of the material, then the results of the acoustic emission.

- Strains : first of all, it is interesting to note on figure 5 representing the deviatoric stress in function of εv, that under simple compression the increase in volume (change of sign of the variation of volume) caused by an intense development of cracking appears without noticeable hardening phenomenon while at stronger confinement pressures (10 MPa at least), this increase in volume appears relatively late, after a considerable hardening of the material. This tends to show that in the case of sufficient lateral pressure, cracking develops relatively late.

Referring to the curves in figure 6 $\varepsilon_2 = f(\varepsilon_1)$, it can be noted for all the tests that from a certain value onwards these curves present an "anomaly" in that from that value the lateral strains increase abnormally in relation to the axial strains. It can also be noted that this anomaly occurs shortly after the elastic limit for simple compression and for 5 MPa of lateral pressure and, on the contrary, for stronger pressures , this point goes farther and farther away from the elastic limit.

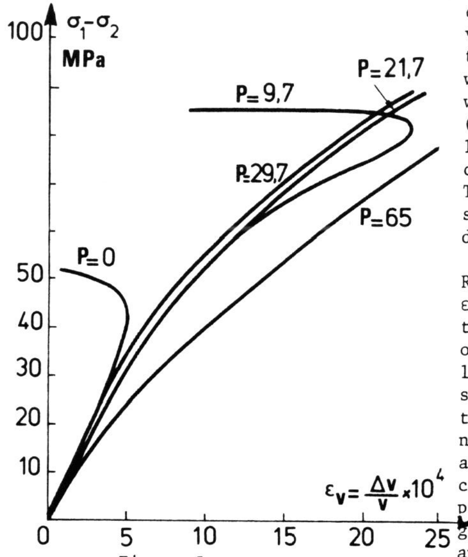

Figure 5

$[\sigma_1 - P] = g(\varepsilon_v)$ *with* $\varepsilon_v = \varepsilon_1 + 2\varepsilon_2$.

It is natural to think that this characteristic point is directly linked with the cracking of the material. Indeed, experience shows that these cracks start in practice in the direction of the principal major stress. This starting, or this creation of cracks with open up, causes an abnormal and considerable increase of the lateral strains without however affecting the axial strains too much, and this can possibly appear on $\varepsilon_2 = f(\varepsilon_1)$ curves as it happens in the case of the concrete studied here. It is interesting to note that this result agrees with the model described in the previous paragraph.

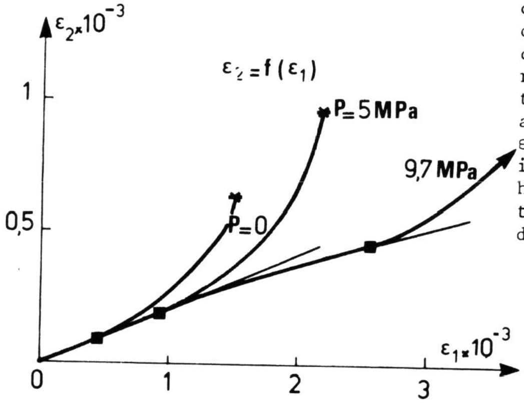

Figure 6 : $\varepsilon_2 = f(\varepsilon_1)$

- Acoustic emission : The figure 7 represents the cumulative acoustic emission (N) curves versus the deviatoric stress (σ_1 - P) for different values of the lateral pressures. The "acoustic emission rate" is defined as :

$$N^* = \left[\frac{\Delta N}{\Delta \sigma}\right]$$

and some constant N^* curves are plotted on the same figure 7.

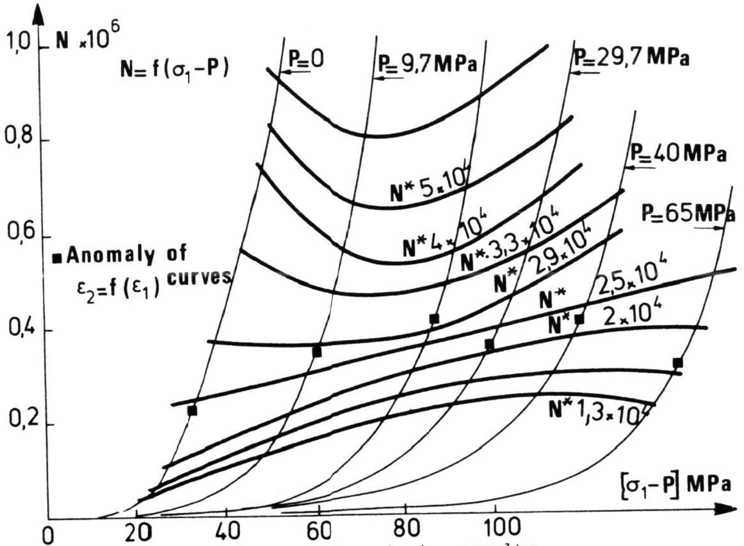

Figure 7 :Acoustic emission results.

This representation reveals that for lower rates than 2 x 10^4 counts per MPa these curves present a maximum, and, on the contrary, for rates higher than 2.9 x 10^4 counts per MPa these curves present a minimum. This change in aspect of this type of curve can also be interpreted by the appearance of new sources of acoustic emission which can only be due to a beginning of cracking. On this chart we have put in the points relative to those corresponding to the "anomalies" recorded on the curves $\varepsilon_2 = f(\varepsilon_1)$; it can be seen that these points oscillate round the straight line corresponding to 2.5 x 10^4 counts per MPa, delimiting the transition between the two forms of curves of the egal rates. It can also be noted that for the point of the curve at P = 65 MPa this is not verified ; the divergence is even quite considerable ; it is possible that this phenomenon may be due to the fact that for this confining pressure the curve $P = k(\varepsilon_v)$ (fig. 8) shows that the elastic boundary has already been crossed in hydrostatic stress, and that thus, at the moment of putting on the axial load, we are no longer confronted with the same material.

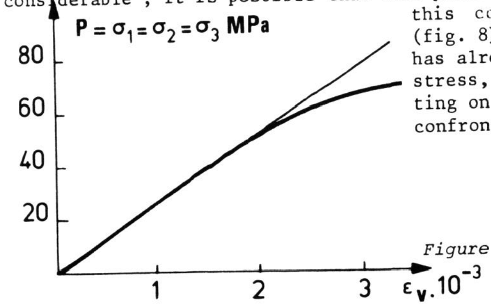

Figure 8 : $\sigma_1 = \sigma_2 = \sigma_3 = P = k(\varepsilon_v)$.
hydrostatic pressure

- "Elastic" and "cracking" limits : On figure 9 the cracking and the elastic field of the concrete being studied (the latter being defined as being the limit of linearity of the stress-strain curves) are represented in the plane (σ_1, P), where three zones are defined (A, B, C). This figure reveals the importance of the non-cracked field in relation to the elastic field especially for high lateral pressures. Some triaxial creep specimens tested in zone B and C (fig. 10) show that in the non-cracked field (B), the creep becomes stable whereas beyond, a delayed failure can be observed in the zone C.

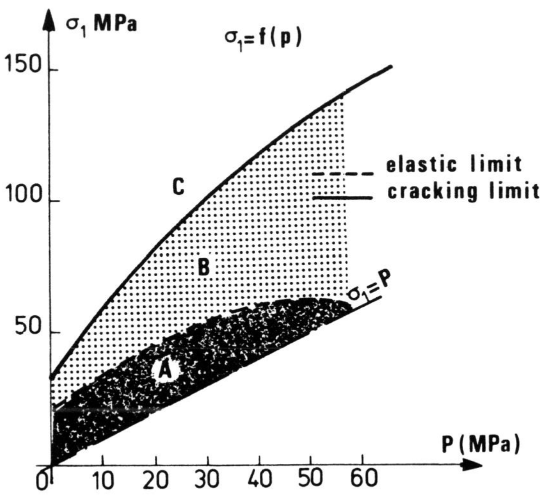

Figure 9

Elastic and cracking limits.

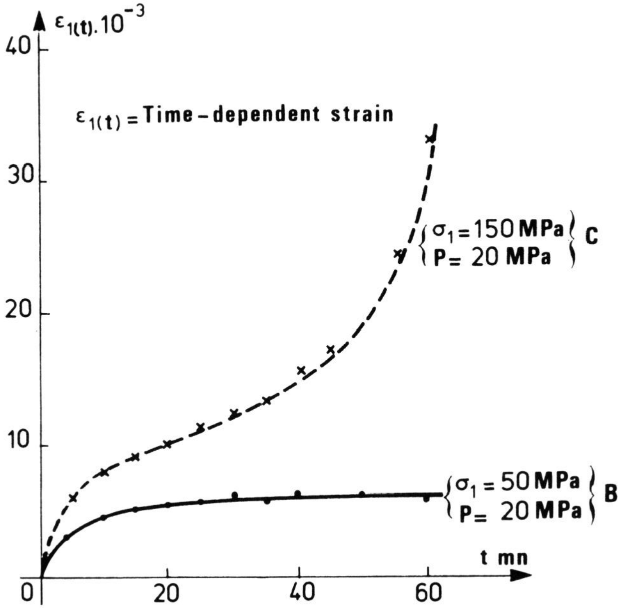

Figure 10

Triaxial creeps $\varepsilon_1 = f(t)$.

CONCLUSIONS

1. Studying simultaneously curves of deformations and acoustic emission led to define a domain of "non microcracking" of the concrete. In the brittle case the limit of elasticity and the limit of microcracking are quite close, but in the ductile case these two limits differ from each other more especially as the confining pressure increases.

2. Accordingly three domains are defined in the σ_1 , P plane :

- elastic domain, [A]
- non microcracking domain, [B + A]
- domain of microcracking. [C - (A + B)]

3. Tests of creep show that the domain defined by the microcracking limit is "stable" ; this domain can be included in any calculation for the definition of a coefficient of security.

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