Experimental study of the behaviour of building structural elements under soft impact

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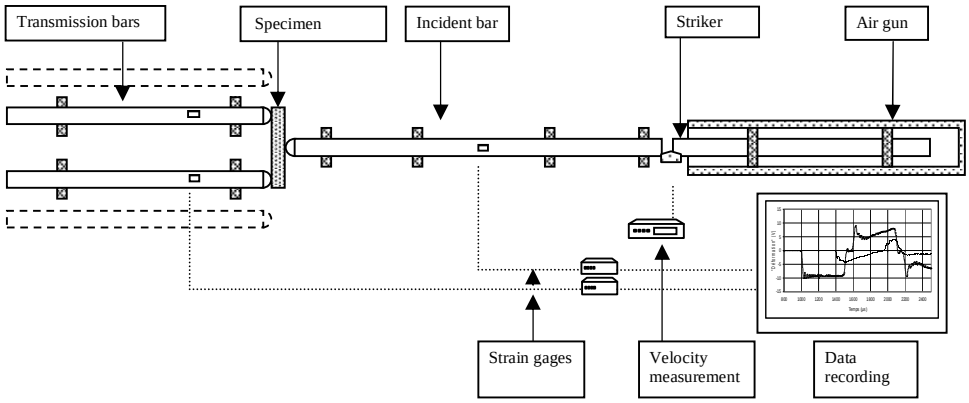
Abstract. Dynamic three-point bend tests are performed on small beams specimen made of a quasi-brittle material. The most classical hypotheses needed to analyse the tests with SHPB, which are based on a state of quasi-static balance of the body, cannot be used. In order to get the behaviour it is necessary to perform a transient analysis of the specimen response. In most cases, the useful duration of the test - the elapsed time between the beginning of the load and the total failure - is lower than the time needed by the waves to reach the supports. The test is thus called a “one point bending test". In the case of bending failure, an analysis of the tests is possible by using only the measures given by the input bar. The technique is based on the knowledge of the analytical solution of the transient elastic response of the specimen. The failure mechanisms involved are specific features of the dynamic response. Different failure modes can be observed according to the loading rate: bending or shear fracture, single or multiple fractures. The determination of the bending moments and the rotations of the beam section, where the failure occurs, lead to an estimation of the strength and the failure energy.

1. INTRODUCTION

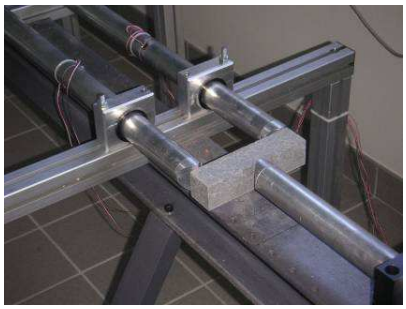
Structural elements such as walls, beams and columns can be damaged by shocks. The study of the behaviour of structural components under soft shocks is possible by using the Hopkinson bars (SHPB). Dynamic three-point bend tests are carried out on specimen made of brittle materials (like concrete, bricks ...). The experimental results show that, in many cases, the failure occurs for small strains and before static equilibrium is achieved in the specimen.

2. EXPERIMENTAL DEVICE

In order to look at the behaviour of quasi-brittle materials under dynamic loads, there are many different types of testing devices [1]. Using a Split Hopkinson Pressure Bars system or Kolsky bars is also a possible choice [2] [3] [4]. The device consists mainly of three bars: an input (or incident) bar impacted by a striker (short bar) coming from an air gun and two parallels output (or transmission) bars. The beam specimen is placed between the incident and the transmission bars. « Three point » bending tests, using SHPB, are usually performed with notched specimen for dynamic fracture toughness tests [5][6][7]. In this article, the specimens are unotched: the figure 1 shows pictures of specimen set onto the device. The strain waves in the bars are measured by resistive gauges glued on the input and the output bars. The velocity of the striker is measured just before the impact.



(a)



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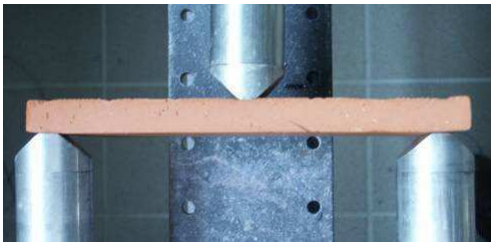

(c)

Figure 1. (a) The SHPB device (aluminium bars ϕ 40), (b) a micro- concrete specimen, (c) a brick specimen

The impact velocity and the impact force applied on the specimen are evaluated using classical formulas (1):

$$V_e(t) = -C_B\left(\varepsilon_i(t) - \varepsilon_r(t)\right) \qquad F_e(t) = -S_B E_B\left(\varepsilon_i(t) + \varepsilon_r(t)\right) \tag{1}$$

The tests, presented in this paper, are carried out on micro concrete and brick specimens. The geometric and material parameters of both specimens are specified in table 1. It has been observed that these tests are highly reproducible. The load reactions at the supports are derived from the measurements on output bars.

Table 1. specimen geometries and material parameters

	Concrete	Brick
Length L	14 cm	20 cm
Thickness a	4 cm	1.7 cm
Width b	4 cm	6.5 cm
Elastic modulus E	12 GPa	7 GPa
Density ρ	2000 kg/m^3	1400 kg/m^3

2. MODELLING OF THE TEST

Analyzing the bending test is possible if and only if stresses and strains characterizing the material behaviour can be derived from the data (force and displacement at the impact point). A relevant model is therefore required to represent the specimen motion and to estimate the internal stresses. This modelling must be time and space adjusted. The characteristic time of the studied phenomenon is greater than the propagation time of a transverse wave within the specimen. The Euler Bernoulli's beam model (2) is used to represent the specimen mechanical response. The authors use the most

accurate analytical model according to the situation [8 - 15]. A key feature of the bending tests performed on quasi-brittle materials is that the time needed for a specimen to fail is lower than the time required for a bending wave to reach the supports: the load reactions at supports remain very small and appear only after the specimen failure. Therefore the problem is equivalent to a problem involving a beam with an infinite length. It is linked to the fact that there is no feedback information from the supports before fracture.
Boundary conditions, at the impact point, are given by (3). Functions $U_e(t)$ and $F_e(t)$ are derived from the measured incident and reflected waves using the relations (1). Functions $\psi(t)$ and M (t) are unknown:

$$\frac{\partial^4 w}{\partial x^4} + 4\alpha^4 \frac{\partial^2 w}{\partial t^2} = 0 \qquad 4\alpha^4 = \frac{\rho S}{EI} \qquad x \in [\,0\,, +\infty\,[\tag{2}$$

$$w(0,t) = -U_e(t) \quad \frac{\partial w}{\partial x}(0,t) = \psi(t) \quad \frac{\partial^2 w}{\partial x^2}(0,t) = \frac{1}{EI} M(0,t) \quad \frac{\partial^3 w}{\partial x^3}(0,t) = \frac{-1}{2EI} F_e(t) \tag{3}$$

The transient dynamic elastic analytical response of the specimen can be derived from the measured boundary conditions (4):

$$w(x,t) = -\int_0^t G_1(t-\tau)\,\Omega_1(x,\tau)\,d\tau + \int_0^t \left(G_1(t-\tau) - G_2(t-\tau)\right)\Omega_2(x,\tau)\,d\tau \tag{4}$$

$$G_1(t) = \int_0^t \frac{V_e(\tau)}{\sqrt{\pi(t-\tau)}}\,d\tau \quad G_2(t) = \int_0^t \frac{F_e(\tau)}{4EI\alpha^3}\,d\tau \quad \Omega_1(x,t) = \frac{1}{\sqrt{\pi t}}\cos\left(\frac{\alpha^2 x^2}{2t}\right) \quad \Omega_2(x,t) = \frac{1}{\sqrt{\pi t}}\sin\left(\frac{\alpha^2 x^2}{2t}\right)$$

Employing the analytical Euler-Bernoulli beam model, one gets many results from experimental data For example, we can deduce the bending moment along the beam. The figure 2.a represents the bending moment at the instant of fracture for a brick. We can also deduce the evolution of the bending moment in the section where the fracture occurs (x=0) and the opening of the crack (the angle ψ). The figure 2.b shows such a result (for the same brick as in figure 2.a). During the whole elastic phase the hypothesis $\psi(t) = 0$ remains valid. The computed results are very closed to this condition even if there exits a delay due to the imperfect initial contact between the input bar and the specimen. On this curve, it is easy to detect the failure and to estimate energy dissipation during fracture.

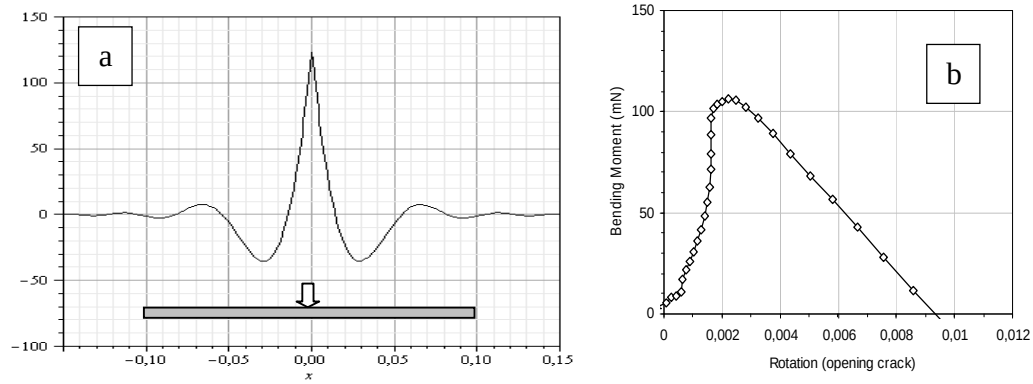

Figure 2. a) Bending moment M (x,T_R) at T_R = 32 µs (first fracture time) for a brick specimen
b) Bending moment M (0, t) versus rotation ψ (t)

3. THE FAILURE MECHANISMS

3.1 Criterion for the quasi brittle fracture

The impact test on a beam made of a quasi brittle material can lead to several mechanisms of failure. The first one is the bending failure like in a quasi static test. The second one is the shear fracture mode which leads sometimes to multi fragmentation. Shear fracture mode and multi fragmentation can usually only occur in dynamic tests.
For a quasi brittle material, the simplest failure criterion is the Rankine criterion. The failure occurs when the highest principal stress reaches a limit σ_T. For a beam, the criterion can be expressed as a function of the generalized stresses like bending moment M and shear force V.
The stress tensor in a beam is given by.

$$\underline{\underline{\sigma}} = \begin{pmatrix} \sigma_{11} & \sigma_{12} \\ \sigma_{12} & 0 \end{pmatrix} \qquad \begin{cases} \sigma_{11} = x_2 \dfrac{M}{I} = x \dfrac{M}{M_T} \sigma_T \\ \sigma_{12} = \dfrac{3}{2} \dfrac{V}{bh^3} \left(h^2 - 4x_2^2 \right) = \dfrac{V}{V_T} \sigma_T \left(1 - x^2 \right) \end{cases} \tag{5}$$

$$x = \frac{2x_2}{h} \qquad k_M = \frac{M}{M_T} \qquad k_V = \frac{V}{V_T} \qquad V_T = \frac{2bh}{3} \sigma_T \qquad M_T = \frac{bh^2}{6} \sigma_T$$

So the criterion is

$$\mathrm{Sup} \left\{ \sigma_I, \sigma_{II}, \sigma_{III} \right\} < \sigma_T \quad \Leftrightarrow \quad \max_{x \in [0,1]} \left\{ x\, k_M + \sqrt{(x\, k_M)^2 + 4\, k_V{}^2 \left(1 - x^2 \right)^2} \right\} < 2 \tag{6}$$

This criterion is drawn in figure 3. We can distinguish two different parts. When $k_M = 1$ and $0 < k_V < 0.92$, there is a bending failure and when $0 < k_M < 1$ and $0.92 < k_V < 1$, there is a shear failure. In quasi static tree point bending test, the shear failure mode is not possible. To obtain this failure mode, the following geometric condition should be verified

$$\frac{3}{4} \frac{F}{bh} > \frac{3}{2} \frac{FL}{bh^2} \quad \Rightarrow \quad L < \frac{h}{2}$$

such a condition does not agree with the hypothesis of a beam structure.

3.1. The loading path

For a dynamic test, the loading path depends on the loading rate. We consider that the loading is an imposed force (this assumption is closed to the real loading).

$$V(0,t) = \beta\, t \tag{7}$$

The beam model leads to

$$M(0,t) = \frac{\beta}{3\alpha\sqrt{\pi}}\, t\sqrt{t} \qquad k_M = \frac{M}{M_T} = \frac{2\beta}{bh^2 \alpha\sqrt{\pi}\, \sigma_T}\, t^{3/2} \qquad k_V = \frac{V}{V_T} = \frac{3\beta\, t}{2\, b\, h\, \sigma_T} \tag{8}$$

The relation (9) gives the loading path.

$$k_M = k_V{}^{3/2} \frac{4}{\alpha} \sqrt{\frac{2b\sigma_T}{\pi\, h\, \beta}} \tag{9}$$

According to the value of β, we can observe bending failure or shear failure. The loading rate corresponding to the transition ($k_M = 1$ and $k_V = 0.92$) is called β_{tr}. The figure 3 shows several loading paths ($d = \beta / \beta_{tr}$), for the specimen made of concrete, $\beta_{tr} = 1.3.10^8$ N/s. The figure 4 shows two

specimens after failure. The first one, where d = 0.38, corresponds to a bending failure. The second one, where d = 7.7, corresponds to a shear failure.

$$\begin{cases} k_V = 0{,}92 \\ k_M = 1 \end{cases} \qquad \beta_{tr} = \frac{25\, b\, \sigma_T}{\alpha^2 \pi\, h} \qquad d = \frac{\beta}{\beta_{tr}} \tag{10}$$

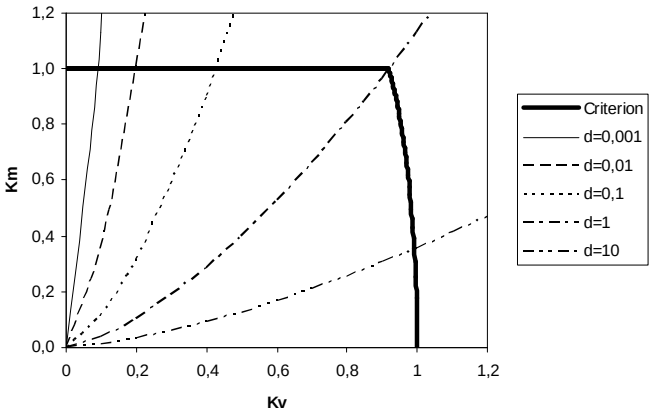

Figure 3. Criterion for a beam made of a quasi brittle material with several loading paths corresponding to several strain rates

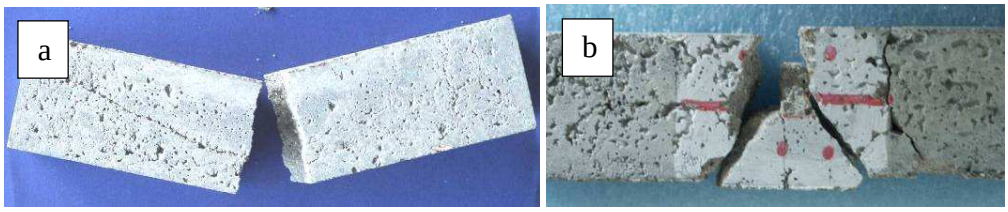

Figure 4. Specimens after the dynamic test. a) Bending failure for a relative loading rate d = 0.38. b) Shear failure for a relative loading rate d = 7.7.

4. MULTI FRAGMENTATION

With a high loading rate, we can observe a first failure followed by a second (or more) one. The figure 5a shows four bricks after impact tested with increasing loading rate. We can observe bending fractures (crack perpendicular to the axis of the beam) and shear fractures (crack inclined at 45° on the axis). The number of fractures increases with the loading rate. This effect may be explained by the elastic transient motion of a part of the specimen, after the first failure [16]. As an example the figure 5b presents, for the second brick, the bending moment at several times after the first fracture which occurs at x = 0.

5. CONCLUSION

This paper shows a procedure to study the three point bending response of quasi-brittle materials using SHPB. The analysis can always be performed even if the measure on the input bar is the only available one. For some brittle tested materials, the effect of strain rate on the behaviour is similar to the one observed in direct tension tests. This dynamic bending test is relatively simple because it does not require building specific specimens: experiments are directly performed on small building components (bricks…).

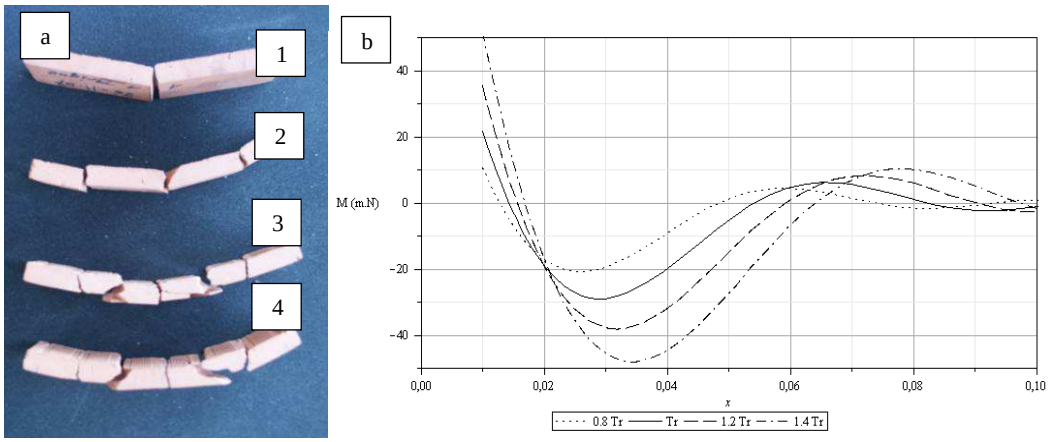

Figure 5. a) Some bricks after the test (strain rate 1: 57 s^{-1}, 2: 279 s^{-1}, 3: 341 s^{-1}, 4: 426 s^{-1})
b) Specimen n°2, bending moment along the beam plotted at several times.

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