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A FINITE ELEMENT MODEL FOR NONLINEAR STRUCTURAL EARTHQUAKE ANALYSIS

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ABSTRACT

Towards the analysis of damage in reinforced concrete structures subjected to earthquakes, we propose a numerical model capable of describing the non-linear behaviour of reinforced concrete beams and columns under alternate cyclic loading, which can be efficiently used also in dynamic analysis: with the assumption of a local uniaxial state of stress, we are able to obtain the rapidity needed for the time integration of the dynamic equations of equilibrium for real structures, within a time interval corresponding to a seismic action.

The model is presented: path-dependant constitutive material law and finite element formulation. An short example of validation serves to evaluate some characteristics of the model. A methodology is then developed to extend the applicability of the model for limit cases, regarding slenderness and semi-rigid limit conditions.

1 INTRODUCTION

At present, in order to predict the behaviour of a structure loaded into its nonlinear domain by an earthquake, it appears necessary to first be able to predict its behaviour in a deterministic way, and thus to be able to model the history of its motion, state of degradation. As a guide to design or as a help in writing a construction code, the tool has moreover to be fast enough to allow parametric studies. Therefore, it is necessary to specialise these tools, as an effort to increase their efficiency, understood as the ratio precision/time of calculation. Towards this objective, we will here focus on the modeling of lineic elements and of their connections.

A local approach is chosen in order to deal with simple, physical material characteristics, and to model general loading histories, using the uniaxial stress-strain curves of the materials. To gain in efficiency, two assumptions are made to specialize the model.

The first assumption is kinematic: plane sections remain plane and orthogonal to the neutral axis. This allows to deal with only six degrees of freedom per two-node element, and brings more speed in solving the system of equations.

The second assumption consists in degenerating the stress tensor to one direction, and letting only a uniaxial stress field develop within the element. This restriction, which yields to neglecting the effect of shear, permits a finer description of the cyclic behaviour of concrete.

2 MATERIAL BEHAVIOUR MODELS

The model developed for concrete is elastoplastic in compression, coupled with the correct treatment of the unilateral behaviour of tension-cracked concrete [1],[2]. The uniaxial version [3],[4] based on experimental observations, has two extra features, which refine its cyclic behaviour:

- the elastic modulus of concrete and its resistance in tension is progressively damaged when the inelastic deformations increase;
- as soon as a crack starts to close, the concrete develops some compression, according to the imperfect imbrication of the crack lips.

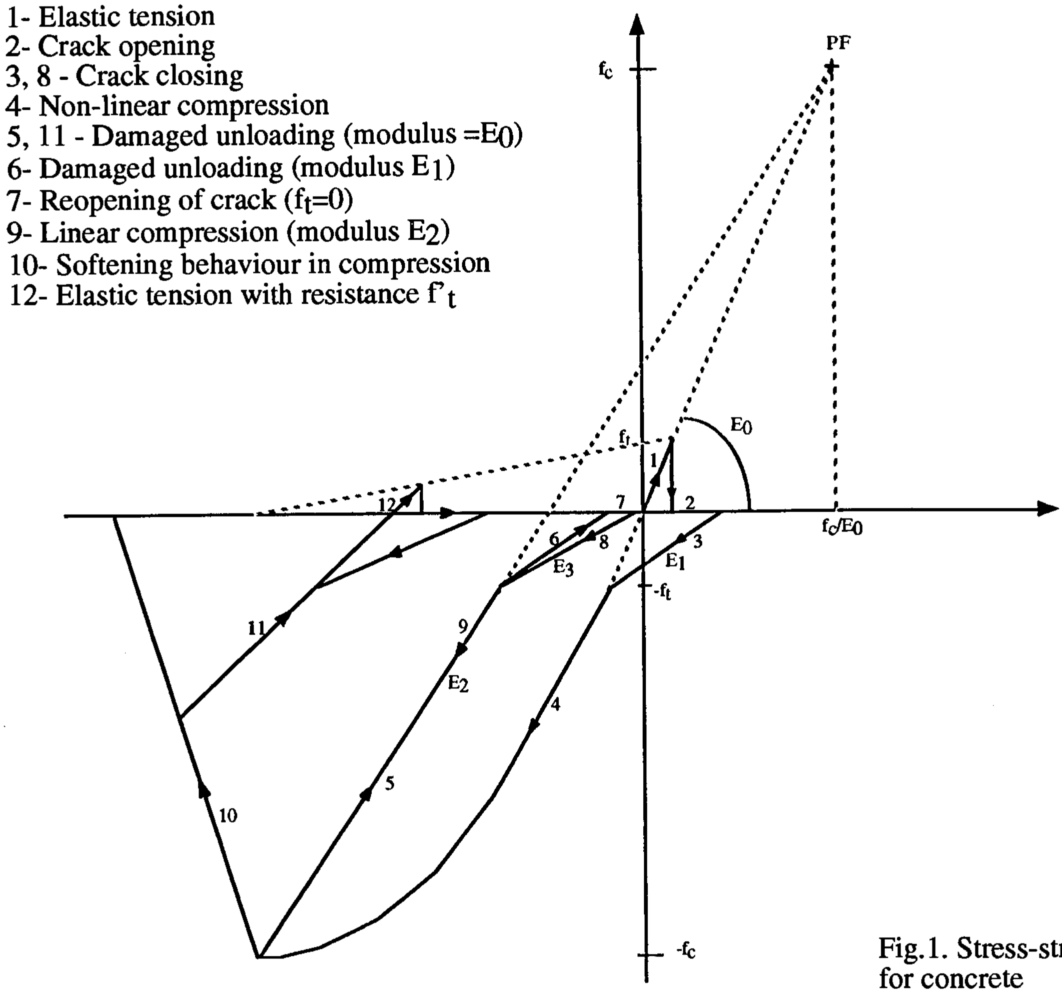

Fig.1. Stress-strain law for concrete

Paths 1 through 9 of Fig. 1 represent the behaviour of concrete initially in tension: when the tensile stress reaches f_t, (path 1&2), a crack is created perpendicular to the direction of traction, and the stress falls to zero until the material is subjected to compression (path 3). As the crack closes progressively, the stress then becomes negative up to $-f_t$, the level at which the rigidity is completely restored (path 4). Along path 5, the compressed concrete follows the unloading line ($E_2 \neq E_0$) passing through a focal point (f_c, ε_0). Indeed, experiments show that the elastic modulus at unloading is different from the initial modulus E_0 due to the damage of concrete through compression. When the stress becomes inferior (in absolute value) to $|-f_t|$ (path 6), modulus E_1 corresponding to the closing, is restored. Paths 7, 8 and 9 obey the same criteria as paths 2, 3 and 4. For a point initially in compression, the tensile resistance is degraded when the material has been previously damaged in compression (path 12).

This law enables us to calculate from the deformation field, the corresponding stress state, described at a number of points, distributed in the volume of the element. By integration of the stress at these integration points, the corresponding internal restoring forces are obtained.

The modeling of steel can be elastic-perfectly plastic, multiple line strain hardening, or with the real stress-strain curve described.

3 VALIDATION

This model has been validated with simple structures, composed of slender elements, subjected to a reduced number of quasi-static alternate cycles [3],[5]. The calculation presented here serves to evaluate the applicability of the model to structures subjected to seismic loads. Indeed, when addressing dynamic problems, one has to face:

- larger time and memory consumption ;
- usually more than five alternate cycles ;
- more complex loads, the production and the effect of which have to be modeled correctly ;
- damping, which is taken into account implicitly, only through material hysteretic dissipation.

The behaviour model itself (calculation of the internal restoring force) is not modified, and thus, the effects of viscosity, fatigue, shear, sliding between steel and concrete, are neglected.

The structure is a reinforced concrete column tested by the C.E.A.(Comission for Atomic Energy) on a shaking table [6]. It was chosen for its good agreement with the basic assumptions: slenderness of 8.5, and complete fixed end due to the good anchorage of the flexural steel. It is subjected to a series of earthquakes, of increasing amplitude, and only the third is modeled here, lasting about 4s, with a maximum base acceleration of 1.9 $m.s.^{-2}$.

The experimental results available concern the horizontal displacement versus time for a point in the middle of the column which is compared to the analytical, on Fig. 2. The measured and calculated maximum amplitudes and frequencies are summed in Table 1.

Table 1. Global results

	experimental	calculated
amplitude max	1.64 mm.	1.55 mm.
frequency	3.33 hz.	3.21 hz.

Even though the corresponding extrema do not occur at the same times, the results are considered to be satisfactory. Indeed, in reality the column is initially damaged when undergoing the third accelerogram, and the initial conditions of displacement and material state are different from the ones imposed in the numerical study. Furthermore we observe indeed that the numerical global results are closest to the experimental ones when the earthquake starts to be strong enough to induce more damage in the specimen.

When the excitation starts to decrease (around t=3.5 s.), the model suffers from a complete absence of damping. Indeed, the materials have entered a new nonlinear but elastic domain, reducing the hysteretic dissipation to zero. It can thus be concluded that the local approach of the degradation models correctly its influence at the global level, for cycles of increasing amplitude, but it should be extended to include eventually fatigue hysteresis as well.

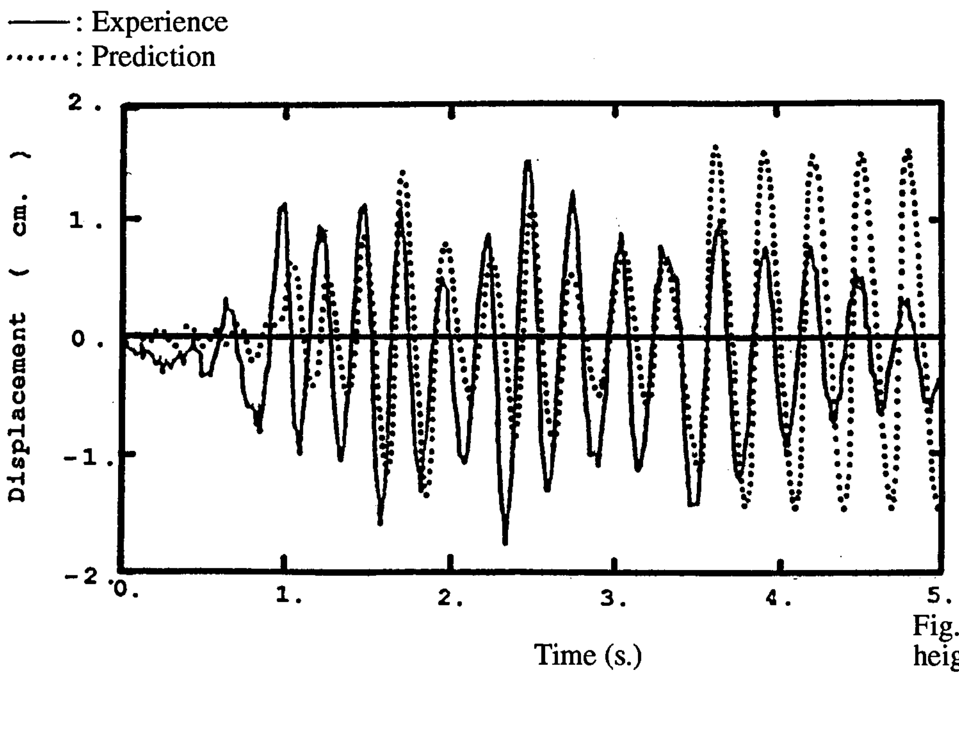

Fig.2: d = f(t) at mid-height of the column

4 APPLICATION TO LIMIT CASES

It is owing to the Bernouilli assumption and the uniaxial modeling of concrete that the speed of calculations allow a dynamic nonlinear analysis. It is thus particularly interesting to use it instead of a 2-D approach whenever possible, and thus to explore the borders of its field of application.

In order to evaluate the effect of neglecting shear in the nonlinear states, and to model the effect of particular limit conditions, we will compare the results obtained using both the 1-D and 2-D models on a structure verifying only very grossly the assumptions: slenderness of 2.7, and unilateral limit conditions.

This structure has been tested within the CASSBA program, to which the CEBTP, CEA, and GRECO Geomaterials participated. The program (Seismic Design and Analysis of Reinforced Concrete Structures,) funded by the Ministery of Research and Technology, the National Building Federation, and the CEA, is aimed at:

- delivering experimental data on the seismic behaviour of R/C shear walls;
- demonstrating the interest of this kind of structures;
- testing and developing numerical models.

A one third scale model of an eight storey building has thus been tested on the Azalée shaking table of the CEA/DMT. It is briefly described fig. 3, and more detail can be found in [7]. The structure being symmetrical and loaded in the planes of the walls, only these will be modeled. The biaxial model used is the one developped by the CEA and implemented in the CASTEM 2000 computer program[8],[9]. The loads are composed of:

- the self weigh, and other dead loads;
- vertical loads concentrated on the anchorage points of the transversal beams on the foundation sole;
- the incremented load: either static equivalent for the 1-D/2-D comparisions, or a base acceleration for the dynamic analysis.

4.1 Base completely fixed

The results of the static monotonic calculations show the influence of the assumptions used for the 1-D model, as a function of the degree of degradation.

In the elastic domain, the beam model is stiffer of about 20%, due to its inability to deform shear distorsion. Thus, the natural frequency will be 10% higher, and this is acceptable since the spectrum of the earthquake is relatively flat around this value.

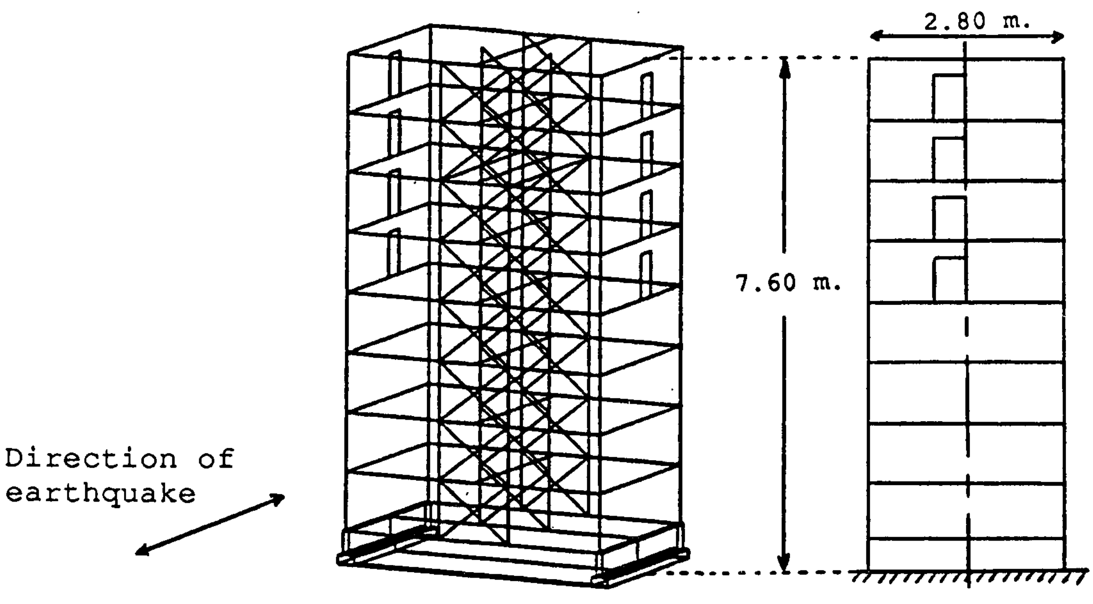

Fig 3: Schematic description of the specimen.

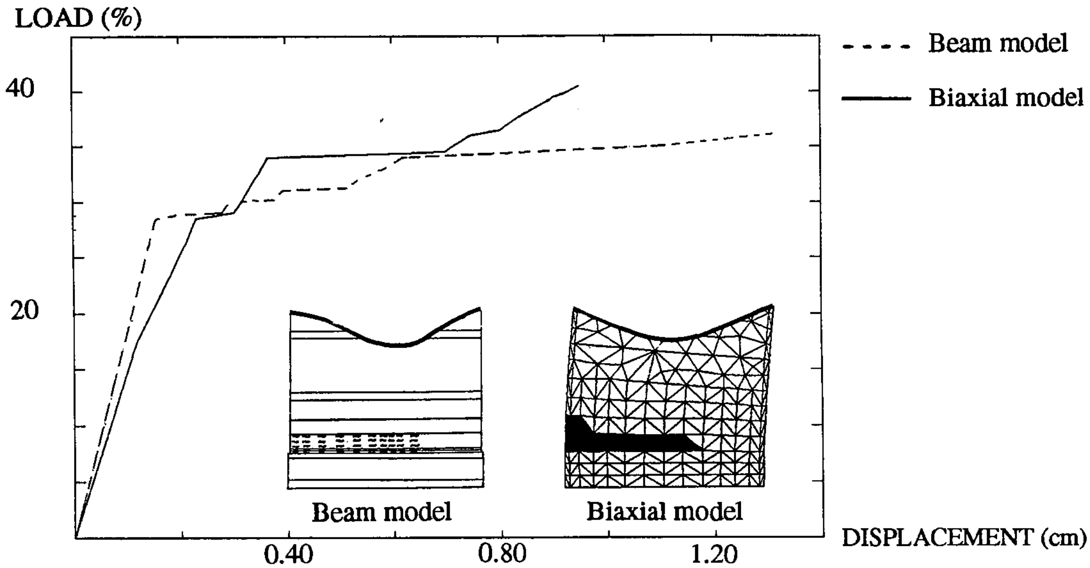

Fig 4: Comparision of 1-D and 2-D models

The local deformation due to the concentrated loads at the end of the base foundation beam triggers the first very local cracking according to the biaxial model, causing the first breakpoint. This cannot be detected by the beam model since then the loads can only be applied on the neutral axis. This is not due to the low shear span ratio.

Both models enter the first crack plateau for the same load, and from then, the biaxial model is stiffer because it can develope a compressive strut, shear transfer is allowed along the lips of a crack, and distorsion becomes relatively less important. Nevertheless, the crack pattern is almost identical in the two approaches.

The second crack plateau is also reached for the same loads, but then, as the biaxial model regains stiffness, the beam model predicts yielding of the reinforcement. One has to consider that the constitutive model for the biaxial behaviour of concrete does not allow the shear transfer factor to decrease with crack opening, as is the case in reality. This overestimation is felt to be responsible for the stiffening observed. At this stage, the beam model overestimates the cracking.

With the assumption that the similarities of behaviour calculated above still hold for a cyclic loading on one hand, and for different but relatively close horizontal loads on the other, it can then be concluded that the presented model will yield correct amplitudes and frequencies, and a good idea of the crack pattern, when used in dynamic analysis for this structure.

4.2 Influence of unilateral limit conditions

The object is now to evaluate the possibility of using the beam model even in the case when limit conditions are imperfect, here for unilateral conditions on the base of the specimen, free to translate upwards. This brings another approximation to the assumption of plane sections in the neighborhood of the base.

In order to take this into account with the beam model, a fixed-end rotation type model able to reproduce the differences in the behaviours of a perfectly fixed specimen and a simply supported one will be constructed on the basis of the biaxial analysis.

This analysis showed that the effect of unilateral conditions on the global results depend on the concentrated loads distribution, the elastic modulus, the tensile resistance, the tensile softening behaviour. A priori, these characteristics vary during a seimic excitation, and one cannot know beforehand the correction to apply to a dynamic analysis performed with a perfectly fixed end. One should then model the limit condition itself, with a model including only the semi-rigidity of the foundation, (which stays practicaly undamaged,) corresponding to its particular caracteristics.

The proposed model is composed of a moment-rotation law, and a linear relation between the base rotation and the vertical displacement of its center. The moment-rotation law should be symmetrical with respect to the origin, and elastic nonliear, in order that the energy produced be only kinematic. The simplest approach is a bilinear law, with three parameters:

- the initial stiffness, which brings the flexibility due to very local loss of contact at the end of the foundation,
- the stress level at which the centre-point of the foundation beam starts rising, bringing much more rigid body rotation to the structure,
- the residual stiffness, taking into account the real deformation of the foundation.

The linear relation between rotation and vertical displacement allows us to choose the point about which the base "toples".

5 CONCLUSION

A reinforced concrete Bernouilli beam model is proposed and validated through a simple example. From this it is concluded that it is necessary to improve the constitutive law of concrete and/or that of bond between steel and concrete in order to model the hysteretic dissipation throughout constant or decreasing load cycles.

A methodology is developed to use this model for dynamic analysis instead of a biaxial model in the cases when its basic assumptions are only approximately verified: a static bidimensional analysis is carried out to measure the degree of precision that can be expected from the dynamic analysis, and to calibrate a fixed-end rotation model to take into account the unilateral limit conditions.

The validations of this approach will be based on the CASSBA results.

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