Using damage mechanics to model a four story RC framed structure submitted to earthquake loading

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ABSTRACT: A simplified model is proposed to simulate the nonlinear behavior of a four-story full-scale reinforced concrete framed structure subjected to severe dynamic loading. The structure has been tested pseudodynamically in the European Laboratory for Structural Assessment (ELSA) at the Joint Research Center of the European Commission. The proposed model uses 2D multi layered Bernoulli beam elements and uniaxial constitutive laws based on damage mechanics and plasticity. Comparison with the experimental results shows the efficiency of the approach.

Keywords: reinforced concrete, framed structure, pseudodynamic tests, damage mechanics, nonlinear dynamic analysis

1 INTRODUCTION

Treating the non-linear dynamic behavior of a reinforced concrete structure can be a complex and costly process. Thus using simplified finite element methods may be an advantageous solution, as long as major phenomena are finely and realistically described (Mazars 1998).

The purpose of this paper is to study a simplified model able to simulate the behavior of reinforced concrete framed structures submitted to seismic loadings. The model is based on multi-layered 2D Bernoulli beam elements and 1D constitutive laws based on damage mechanics and plasticity. Comparison with the experimental results of a four-story full-scale reinforced concrete structure tested in the European Laboratory for Structural Assessment (ELSA) at the Joint Research Center of the European Commission shows the efficiency of the approach. The layouts of the tested structure were designed using the drafts of EC2 (Eurocode 2, 1984) and EC8 (Eurocode 8, 1988), in the framework of a research program (Cooperative Research, 1991) on the seismic response of reinforced concrete structures.

2 EXPERIMENTAL CAMPAIGN

2.1 *Framed structure*

The tested building was a four-story, high-ductility, framed structure (Figure 1). Dimensions in plan were 10mx10m, measured from the column axis. Interstory heights were 3.0m, except for the ground story with 3.5m.

Figure 1. Overall view of the building.

The structure was symmetric in one direction (direction of loading), with two equal spans of 5.0m, whilst in the other direction it was slightly irregular due to the different span lengths (6.0m and 4.0m). This irregularity was introduced to obtain a more realistic building. All columns had square cross section with 400mm side, except for the interior column, which had 450mmx450mm. All beams had rectangular cross section, with total height of 450mm and width of 300mm. A solid slab, with thickness of 150mm was adopted for all stories (Figure 2).

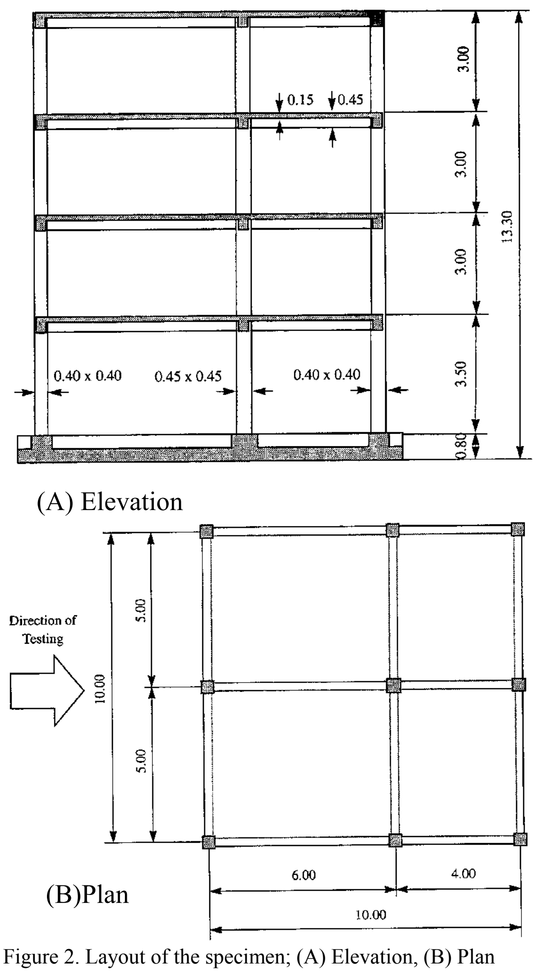

Figure 2. Layout of the specimen; (A) Elevation, (B) Plan

2.2 *Materials*

The materials used for the specimen were normal-weight concrete C25/30 as specified by Eurocode 2, and B500 Tempcore rebars and welded meshes (Negro et al, 1994). The use of this kind of steel has reduced the number of bars to be placed in the members and led to a much easier construction. Compressive strength tests on 150mm side cube yielded an average strength of 46.8 MPa. Tensile tests performed on steel bars allowed to obtain the following average results: yielding stress (570 MPa), ultimate stress (655 MPa) and ultimate strain (23 %).

2.3 *Additional masses*

Additional masses were attached to the floor slabs (Figure 1) to represent the additional dead loads and factorized live loads which, according to the combination factors suggested in EC8, act together with the seismic forces. The total added mass was of 24.3 ton for each of the first three stories and 26.1 ton for the top story.

2.4 *Snap-Back tests*

Prior to the principal pseudodynamic tests, two Snap-Back tests were performed by pulling the structure against the reaction wall by means of a steel bar. The steel bar was attached to one story, in correspondence of the central column of the side toward the reaction wall, so that uniform deformations have been imposed in the direction of testing. Two different tests were performed, one with the steel bar attached to the third story, and the other with the bar attached at the top story. This was done to capture the contribution of the four main vibration modes. The steel bars were dimensioned so that they would have broken for a load of 150 KN in the first test and 85 KN in the second, values, which were supposed not to lead to significant cracking inside the structure.

The behavior of the structure was reasonably linear during the loading phase, and the average value of the damping ratio was found to be about 1.8%.

2.5 *Pseudodynamic tests*

Initially developed in Japan (Takanashi, 1975), the pseudodynamic test method is a hybrid testing technique that combines the numerical integration of the dynamic equilibrium equation with experimental information about the structure, acquired quasi-statically, to provide realistic dynamic response histories, even for the nonlinear behavior of severely damaged structures. Displacements are imposed on the structure by means of hydraulic actuators.

The pseudodynamic tests were performed using a set of artificial accelerograms generated by using the waveforms derived from the 1976 Friuli Earthquake (Negro et al, 1994) (Figure 3).

A Low-Level test with the reference signal scaled by 0.4 was first performed. The resulting nominal ground acceleration, was thought not to cause significant yielding inside the structure and it could be assumed as the one corresponding to the serviceability limit-state. Thus the stiffness of specimen was not considerably changed after this test.

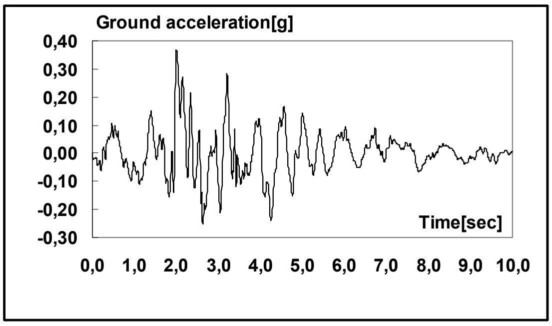

Figure 3. Friuli-like generated accelerogram.

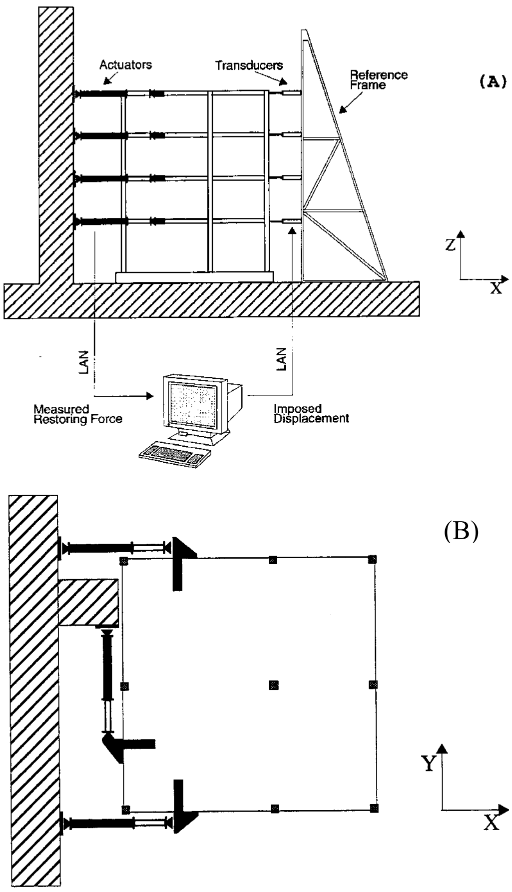

Figure 4. Experimental setup A) Side view B) plane view

A High-Level amplitude pseudodynamic test was performed afterwards using the reference signal multiplied by an intensity factor of 1.5. The nominal peak ground acceleration was representative of the maximum seismic actions for which the framed had been designed. This intensity level is particularly meaningful for defining the damage resulting from the design-level seismic actions

The pseudodynamic setup configuration for the tests is shown in Figure 4. The degrees of freedom (dof) coincide with each floor in the direction X-X. The structure was then modeled as a 4 dof system; neither rotational (about Z-Z) nor translational modes (orthogonal to the plane X-X) were modeled analytically for the test.

3 SIMPLIFIED MODEL

3.1 *Numerical tools*

The finite element code EFICOS was used in order to perform non-linear dynamic calculations. The program EFICOS was conceived on the bases of 2D Bernoulli or Timoshenko multi-layered beam elements allowing to simulate the behavior of a wide variety of beam/column structures. (Dubé, 1997; Ghavamian et al. 1998; Kotronis, 2000).

Constitutive behavior laws are described at each layer for concrete and steel. The constitutive model for concrete under cyclic loading must take into account some observed phenomena, such as decrease in material stiffness due to cracking, stiffness recovery which occurs at crack closure and anelastic strains concomitant to damage. The proposed damage mechanics model (La Borderie, 1991) is based on two scalar damage variables, one for damage in tension and the other for damage in compression. Unilateral effects, stiffness recovery and inelastic strains are taken into account. Figure 5 gives the uniaxial cyclic response, from tension to compression, of this model.

The total strain is given by:

$$\varepsilon = \varepsilon^{e} + \varepsilon^{an} \tag{1}$$

$$\varepsilon^{e} = \frac{\langle \sigma \rangle_{+}}{E\,(1-D_{1})} + \frac{\langle \sigma \rangle_{-}}{E\,(1-D_{2})} + \frac{\nu}{E}\left(\sigma - Tr(\sigma)\boldsymbol{1}\right) \tag{2}$$

$$\varepsilon^{an} = \frac{\beta_{1} D_{1}}{E(1-D_{1})}\frac{\partial f(\sigma)}{\partial \sigma} + \frac{\beta_{2} D_{2}}{E(1-D_{2})}\boldsymbol{1} \tag{3}$$

Where ε^{e} is the elastic strain tensor and ε^{an}, the inelastic strain tensor. ***1*** denotes the unit tensor and $Tr(\sigma)=\sigma_{ii}$. $<.>_{+}$ denotes the positive part (in the principal directions). The crack closure function $f(\sigma)$ is:

$$\begin{cases} Tr(\sigma) \in [0,+\infty] \rightarrow \dfrac{\partial f(\sigma)}{\partial \sigma} = 1 \\ Tr(\sigma) \in [-\sigma_f, 0] \rightarrow \dfrac{\partial f(\sigma)}{\partial \sigma} = \left(1 + \dfrac{Tr(\sigma)}{\sigma_f}\right) \boldsymbol{1} \\ Tr(\sigma) \in [-\infty, -\sigma_f] \rightarrow \dfrac{\partial f(\sigma)}{\partial \sigma} = 0.\boldsymbol{1} \end{cases} \tag{4}$$

The damage evolution is given by:

$$D_i = 1 - \frac{1}{1 + [A_i(Y_i - Y_{0i})]^{B_i}} \tag{5}$$

Where Y_i are the thermodynamical forces associated to damage:

$$Y_1 = \frac{<\sigma>_+ : <\sigma>_+ + 2\beta_1 f(\sigma)}{2E(1 - D_1)^2} \tag{6}$$

$$Y_2 = \frac{<\sigma>_- : <\sigma>_- + 2\beta_2 f(\sigma)}{2E(1 - D_2)^2} \tag{7}$$

σ_f represents the crack closure stress; Y_{0i} is the initial elastic threshold; A_i , B_i are damage evolution parameters, β_i are inelasticity parameters. The material parameters can be determined by fitting the uniaxial stress-strain response (E, ν, Y_{02}, A_2, B_2 and β_2 from a compression test; Y_{01}, A_1, B_1 and β_1 from a tension test), σ_f is usually of the same order of the tensile strength (3-4 MPa).

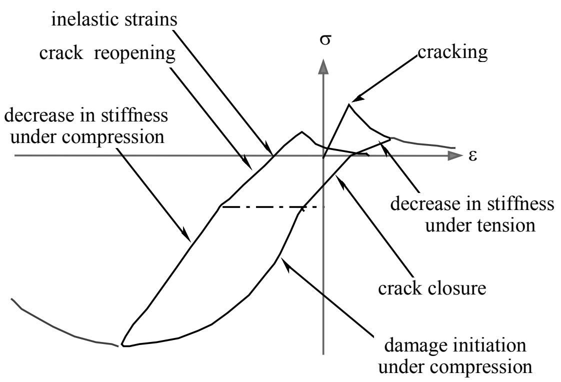

Figure 5. Uniaxial damage model for concrete.

A classical plasticity model with kinematic hardening is used for steel (Figure 6). Hardening can be either linear or non-linear depending on the information provided from the steel tensile strength tests. Reinforcement bars are introduced with special layers whose behavior is a combination of both the behaviors of concrete and steel.

The seismic loading is applied by means of an acceleration at the basis of the structure.

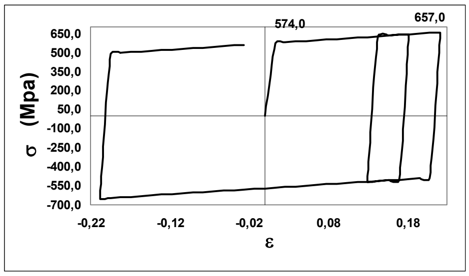

Figure 6. Uniaxial plasticity law for reinforcement bars.

3.2 *Modeling of the structure*

The structure is symmetric in the direction of testing (X) and there is no loading in the perpendicular direction (Y). Therefore one can represent the real structure using an equivalent 2D reinforced concrete frame (Figure 7) where the sections of beams and columns are equal to the sum of real sections (Figure 8). The weight of the equivalent RC frame equals the total weight of the specimen.

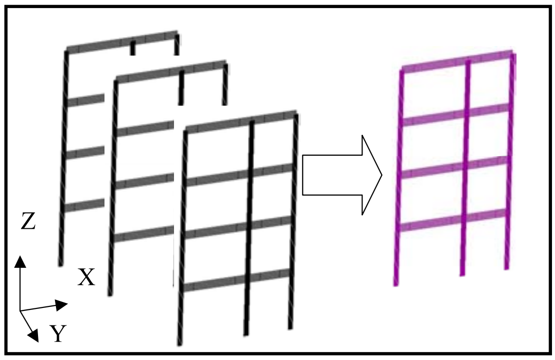

Figure 7. Equivalent RC frame.

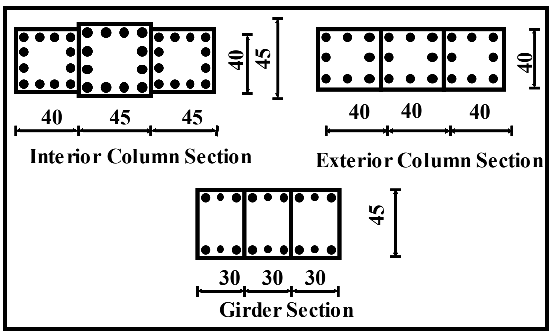

Figure 8. Section of columns and beams of the equivalent RC frame.

175 multi-layered Bernoulli elements are used to model the entire structure. Each element has 12 or so sections (Figure 9).

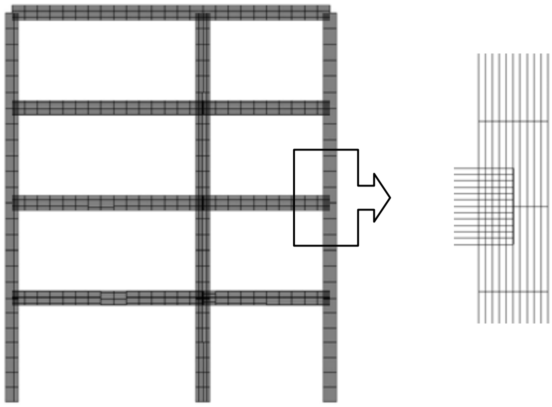

Figure 9. Mesh of the structure with multi-layer beams

3.3 *Modeling of the materials*

The uniaxial damage based model used for the modeling of concrete behavior requires calibration of 10 different parameters (La Borderie 1991):
- Young's modulus: E_0
- Damage parameters for tension : Y_{01}, A_1, B_1
- Damage parameters for compression : Y_{02}, A_2, B_2
- Inelasticity parameters: β_1, β_2
- Closing crack parameter: σ_f

The following assumptions are made in order to derive the necessary values:
1. a ratio equal to 25/30 between ultimate stresses in compression of a cylindrical and a cubic specimen (Eurocode 2, 1984).
2. a ratio equal to 1/10 between ultimate stresses in tension and in compression.

According to the first assumption and to experimental results obtained with 150 mm side cubic specimens, the ultimate stress in compression is chosen equal to 38 MPa. Following the second hypothesis the ultimate stress in tension is chosen equal to 3.8 MPa. Then, the final parameters identified for this study are given in Table 1.

Table 2 and Figure 6 present the stress-strain relation and the principal characteristics of the reinforcement bars (defined according to a series of tensile tests).

Table 2. Characteristics of reinforcement bars.

Young's Modulus [GPa]	Yielding Stress [MPa]	Ultimate Stress [MPa]	Ultimate strain %
200.0	570.2	655.4	23.0

4 VALIDATION OF THE MODEL

4.1 *Snap-Back tests*

The Snap-Back tests are simulated statically using the values of the displacements obtained during the experiment. Two numerical simulations are presented hereafter. In the first, displacements are prescribed at the third story level in the second simulation, they are prescribed at the top of the structure. The maximum values of the imposed displacements are 4.2 mm and 3.33 mm respectively. The whole structure remains elastic. Figures 10 and 11 show the comparison between simulation and experimental results that allows the validation of the elastic stiffness of the structure.

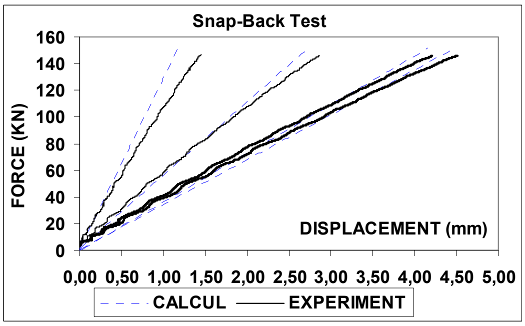

Figure 10. Simulation of the first Snap-Back test (loading applied at the third story).

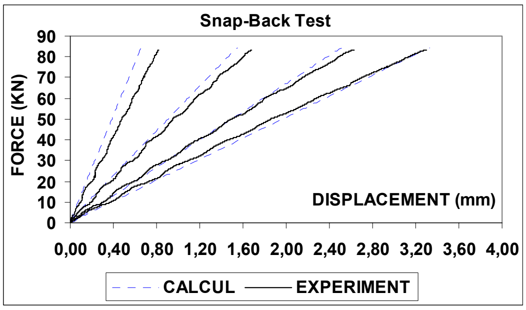

Figure 11. Simulation of the second Snap-Back test (loading applied at the top of the structure).

Table 1. Parameters used for concrete (Abbasi, 2003).

E_0 [GPa]	Y_{01}[MPa]	A_1 [Mpa^{-1}]	B_1	β_1 [MPa]	σ_f [MPa]	Y_{02}[MPa]	A_2[MPa^{-1}]	B_2	β_2 [MPa]]
32.0	3.6E-4	5.0E+3	1.1	1.0	3.8	1.0E-2	5.0	1.5	-30.0

4.2 *Pseudodynamic tests*

According to the Snap-Back tests the damping ratio is taken constant and equal to 1.8% throughout the calculations (Negro et al., 1994). The displacement time history at the top of the structure and the time history of the base shear are presented in Figures 12-13 respectively. The model reproduces correctly the global behavior of the specimen in terms of maximum values and frequency content.

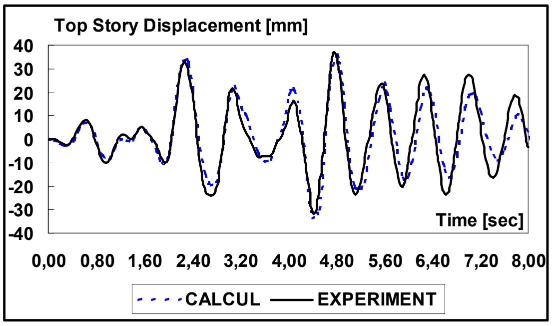

Figure 12. Low-Level pseudodynamic test: top displacement versus time.

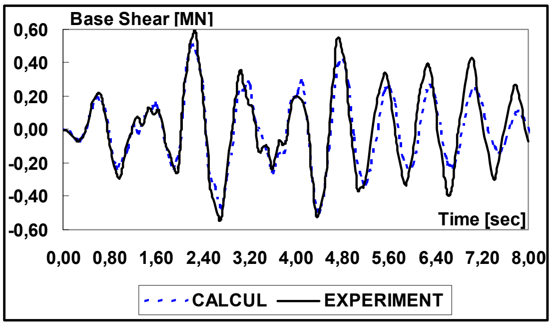

Figure 13. Low-Level pseudodynamic test: base shear versus time.

Figure 14 gives the comparison between the calculated and measured diagrams of top displacement versus base shear. According to Figure 14, the structure does not reveal any important damage, the dissipation mechanisms are not yet activated during this pseudodynamic test.

The distribution of damage due to tension at the end of the calculation is shown in Figure 15 (damage variable D_1 between 0.90 and 1.00). Damage is concentrated near the beam column joints. However no yielding of reinforcement (plastic hinge) or damage due to compression has occurred. These results are in accordance with the experimental observations showing that the structure after the Low-Level pseudodynamic test is in serviceability state.

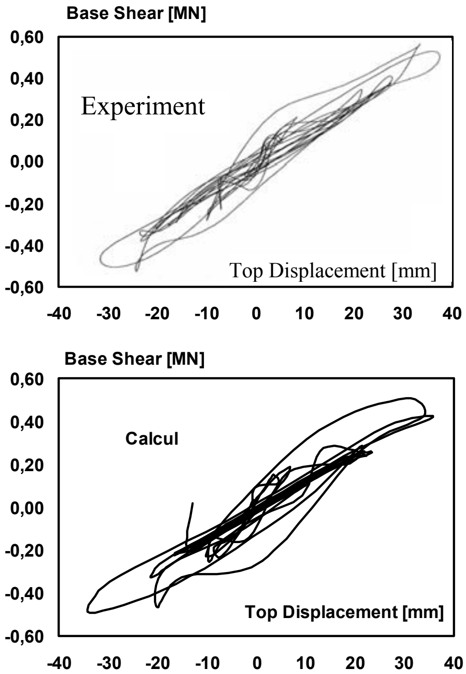

Figure 14. Low-Level Level pseudodynamic test: base-shear versus top-story displacement diagrams.

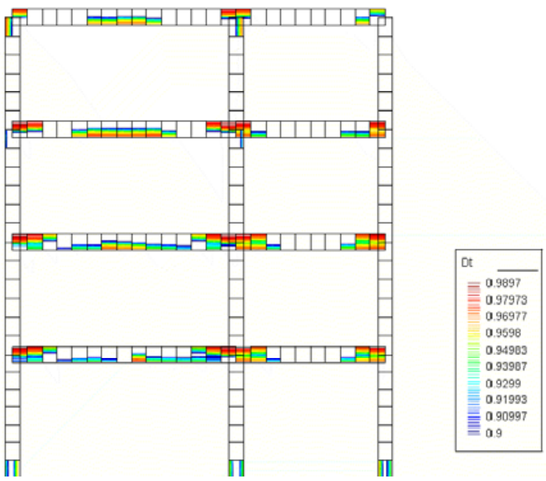

Figure 15. Damage pattern at the end of the Low-Level pseudodynamic test.

The High-Level pseudodynamic test is performed using the reference signal multiplied by an intensity factor of 1.5. This intensity level is particularly meaningful for defining the damage resulting from the design-level seismic actions.

The displacement time history at the top story and the time history of the base shear are given in Figures 16-17. At the beginning the model reproduces correctly the global behavior of the structure. However, as damage increase some differences appear in terms of maximum values.

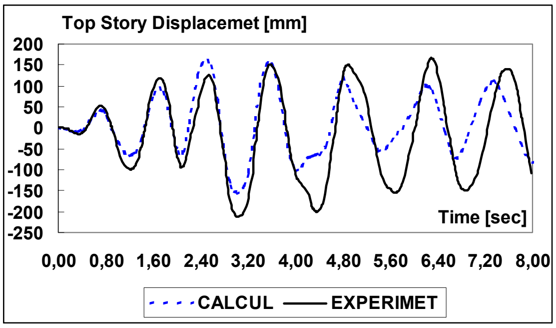

Figure 16. High -Level pseudodynamic test: top displacement versus time.

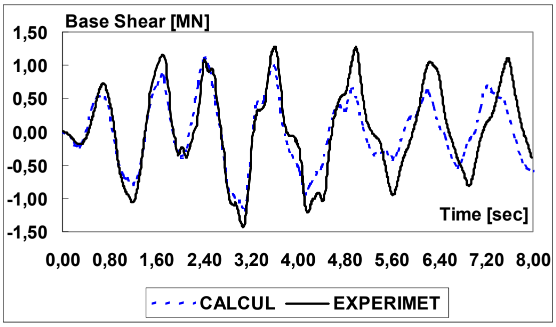

Figure 17. High -Level pseudodynamic test: Time history of the base shear.

In Figure 18 the resulting top-displacement versus base-shear diagram is given for both the experiment and the model. Large dissipative cycles appear that indicates concentration of damage and yielding of steel bars. During the test, cracks opened in the critical regions of beams at the first three stories and of most of the columns. At the end of the test however only the cracks at the beam column joints remained permanently open. Figure 19 represents the position of yielded reinforcement bars at the end of the calculation. Yielding is concentrated at the beam column joints.

5 CONCLUSION

The use of multi-layer beams is an excellent compromise between numerical cost, quality of results and facility of modeling for the majority of reinforced concrete structures. Combined with models based on damage mechanics they allow a detailed study of the distribution of damage in the structure.

An example of a four story RC framed structure was presented throughout this work. Comparison with the experimental results show the efficiency of the adopted modeling strategy in terms of global but also local quantities.

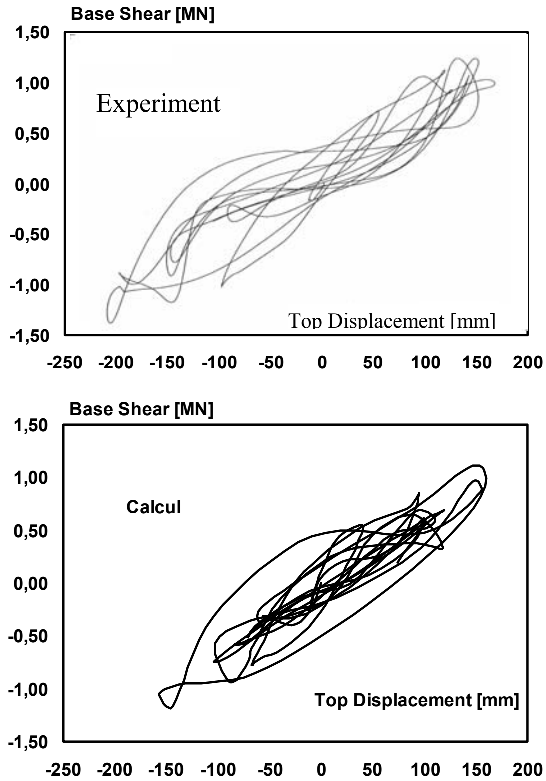

Figure 18. High-Level pseudodynamic test: base-shear versus top-story displacement diagram.

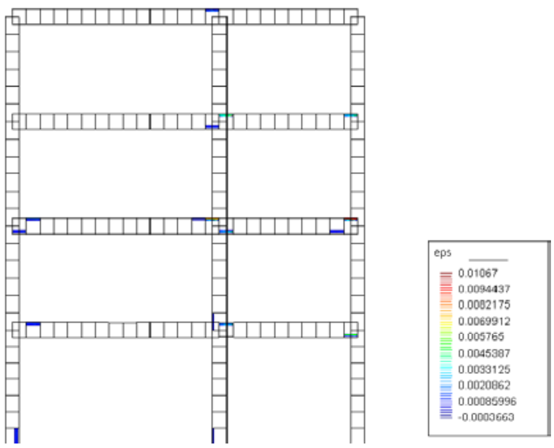

Figure 19. Position of yielded reinforcement bars at the end of the High-Level pseudodynamic test.

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