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Some aspects of local and global behaviour of RC structures submitted to earthquake – Experiments and modelling

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ABSTRACT: Many European buildings are situated in seismic region of low or moderate seismicity. Among these, a large part is not designed under parasismic regulations. Within this context the evaluation of the vulnerability of existing structures is an important issue. In the framework of the European program Ecoleader, a seismic research project has been performed around shaking table tests on mock-ups representing parts of reinforced concrete buildings, the structure of which is based on structural walls. The structure studied in this work is composed of two parallel walls linked with a perpendicular one that has openings. The walls are reinforced according to the current design practice in France that induces a small amount of reinforcement. Two kinds of finite elements models have been performed: a simplified one, based on multifiber beams and a refined one using a complete 3D description of the mock-up. It is shown that both models are able to describe the global behaviour of the structure and qualitatively the distribution of damage at the base of the specimen. Based on the results obtained, the simplified model is used to investigate numerically the behaviour of a wider variety of configurations that is practically impossible to study experimentally.

1 INTRODUCTION

In the framework of the EC Ecoleader program (Bisch & Coin, 2005, Mazars & all, 2005, Kotronis & all, 2005a), a seismic research project has been performed around shaking table tests on mock-ups representing parts of reinforced concrete buildings, the structure of which is based on structural walls. The program concerns two mock-ups: a Slovenian one and a French one, the tests being performed on the large shaking table located at LNEC in Lisbon.

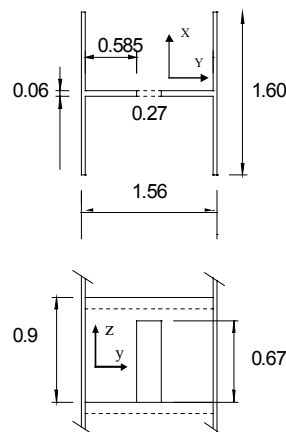


Figure 1. Geometrical data of the French mock up (m)

This work is related to the analysis of the response of the French mock-up. The structure is characteristic of a typical building met in France designed ac-

ording to the European regulation EC8-1 with the French appendix. It is a 5 storey building cell composed of two parallel RC walls braced with a third wall including openings and connected to six square concrete slabs (see Figure 1). The main walls are 5.10 m height, 1.60 m long and 6 cm thick. The total mass is 31.6 tons. All the walls are designed for the seismic level prescribed for a typical seismic region in France. The mock up was designed according to Eurocode 8. The reinforcement is shown in table 1.

Table 1: Reinforcement of mock up

	Vertical ties along the openings walls	Vertical ties for the wall	Vertical ties at the crossing of two wall
5 th storey	2 Φ 6	1 Φ 4.5	2 Φ 4.5
4 th storey	2 Φ 4.5 + 2 Φ 6	1 Φ 4.5	2 Φ 4.5
3 rd storey	3 Φ 4.5 + 2 Φ 6	2 Φ 4.5	2 Φ 4.5
2 nd storey	4 Φ 4.5 + 2 Φ 6	2 Φ 5 + 1 Φ 4.5	2 Φ 4.5
1 st storey	4 Φ 4.5 + 2 Φ 6	2 Φ 5 + 3 Φ 4.5	2 Φ 4.5

Two orthogonal directions of loading are considered X (parallel to the main walls) and Y (parallel to the wall connecting the main ones). Natural accelerograms at different levels have been used (PGA = from 0.3g to 0.85g for direction X and from 0.14g to 0.50g for direction Y) and various data have been collected from the different tests (strain on reinforcements, displacements, accelerations...). In or-

der to follow the evolution of the stiffness, the apparent mode has been measured after each test.

Two kinds of modelling have been performed: a simplified one using multifiber beams and a refined one, based on a complete 3D finite element description of the mock-up.

Constitutive laws are based on damage mechanics and plasticity to describe cracking of concrete and the plastic behaviour of steel. In order to reproduce correctly the behaviour of the structure the stiffness of the shaking table has to be considered in the model.

It will be shown that both models are able to describe the global behaviour of the structure and qualitatively the distribution of damage. They also confirm the conclusions of previous works done on structural walls in France.

2 OUTLINE OF TEST

Two kinds of artificially generated earthquake motions, independent to each other, were applied in the X and Y directions. The sequences of tests are given in Table 2 and Figure 2.

Table 2. Sequences of the tests

Tests	In plane (direct. X)	Out of plane (direct. Y)
T0	0.3 g	-
T1	-	0.14 g
T2	0.24 g	0.13 g
T3	0.45 g	0.27 g
T4	0.55 g	0.30 g
T5	0.74 g	0.36 g
T6	0.85 g	0.50 g

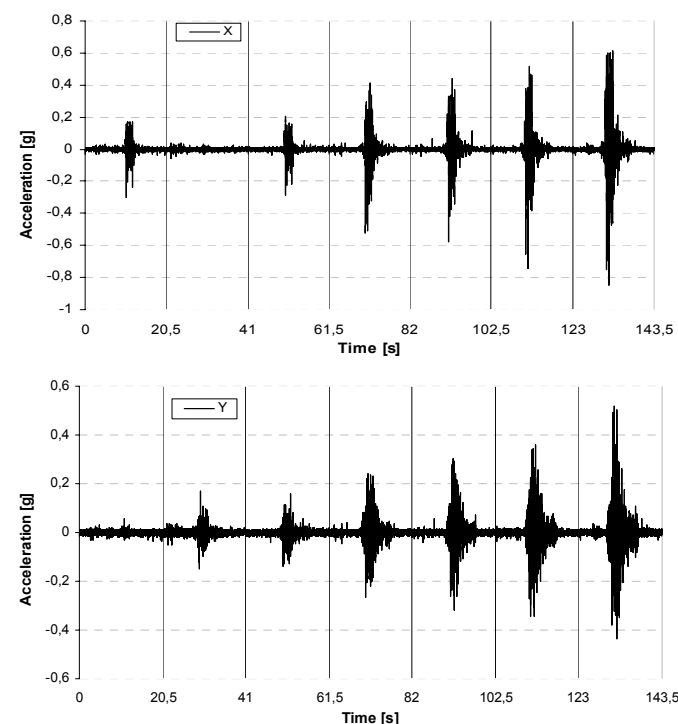


Figure 2. Accelerograms used for the X and Y directions.

The main damages appeared at the base of the walls and one rebar has buckled (but not broken) for a signal closed to the design level (X PGA = 0.45g – Y PGA = 0.27g). For higher levels damage increased and at 0.85g some rebars broke at the base of the walls and a large fracture appeared.

3 NUMERICAL SIMULATIONS

3.1 Refined 3D modelling approach

To predict the inelastic seismic response with sufficient accuracy, due care has been given to create detailed model of the specimen, taking into account the necessary geometric characteristics, construction details and boundary conditions. An example of the 3D finite element mesh used in the analyses is reported in Figure 3.

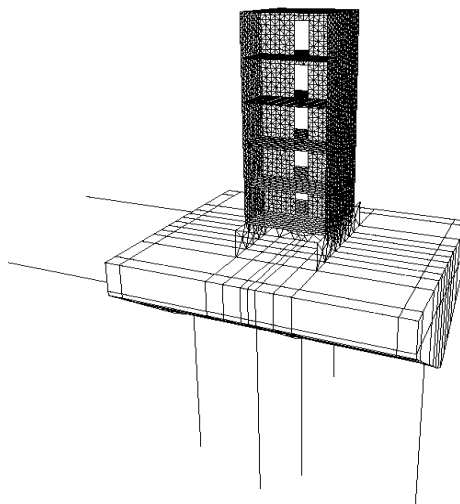


Figure 3. 3D finite element mesh of the specimen.

Due to the direction of the applied loading, in-plane as well as out-of-plane behaviour of the walls need to be analysed. A discrete modelling is adopted to represent the reinforcement through the use of two-node truss-bar elements. The structure is assumed fully restrained at all nodes along the base of the shear wall. During previous tests on CAMUS specimens, it was observed that the specimen oscillation have induced vertical and rocking displacements on the shaking table, leading to significant reductions of the corresponding natural frequencies. In these conditions, the shaking table itself in terms of mass and their external supports in terms of stiffness had to be included into the numerical model. Perfect bond was assumed to exist between concrete and reinforcement. The possibility of non-linear material behaviour was specified for all wall concrete and reinforcing bar-elements, while the behaviour of the foundation and bracing system was considered as elastic. Assuming a 1% critical damping factor for the first and second vibration mode, the damping parameters α and β were calculated and used subsequently to form the Rayleigh damping matrix $[C] =$

$\alpha[M] + \beta[K]$, M and K being the mass and stiffness matrix.

The concrete model used in analysis (Merabet & Reynouard, 1999), adopts the concept of a smeared crack approach with a possible double cracking only at 90° . It is based upon the plasticity theory for uncracked concrete with isotropic hardening and associated flow rule. Two distinct criteria describe the failure surface: Nadai in compression and bi-compression and Rankine in tension. Hardening is isotropic and an associated flow rule is used. When the ultimate surface is reached in tension, a crack is created perpendicularly to the principal direction of maximum tensile stress, and its orientation is considered as fixed subsequently. Each direction is then processed independently by a cyclic uniaxial (Figure 4) law, and the stress tensor in the local co-ordinate system defined by the direction of the cracks is completed by the shear stress, elastically calculated with a reduced shear modulus to account for the effect of interface shear transfer: The model has been described in detail and verified elsewhere (Ile & Reynouard 2000, Ile & all. 2002).

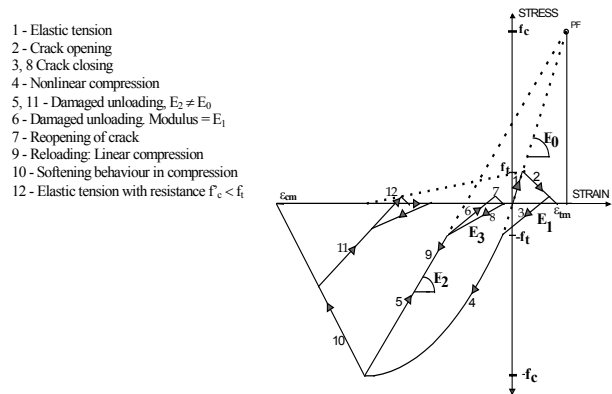


Figure 4. Uniaxial cyclic law : point initially in tension

For steel, the cyclic behaviour is described by the formulation proposed by Giuffrè and Pinto and implemented by (Menegoto & Pinto, 1973).

3.2 The simplified model

Non-linear dynamic analysis of civil engineering structures requires large scale calculations. The necessity to perform parametrical studies led us to adopt a simplified approach in order to reduce the computational cost.

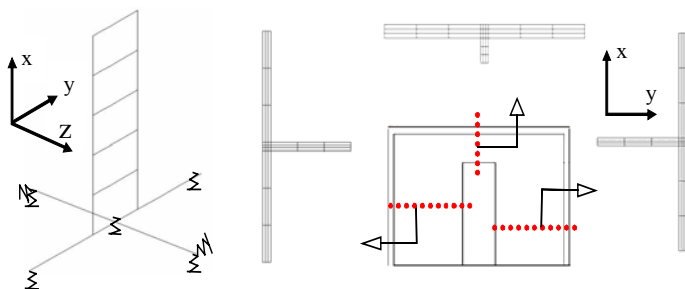


Figure 5. Simplified model : finite element mesh

The structure is modelled with beam elements to reduce the number of degrees of freedom of the problem. Using an Euler Bernoulli formulation (Spacone et al. 1996), the shear deformations are not modelled so we can use 1D version of the non linear constitutive laws in the fibers (torsion is also kept linear). The finite element code used is Aster (Ghavamian & all, 2002)

The finite element mesh is presented in Figure 5. The additional masses and the weight load of each floor are concentrated at each storey. The Rayleigh damping coefficients have been adjusted to ensure a value of 1% on the two first modes.

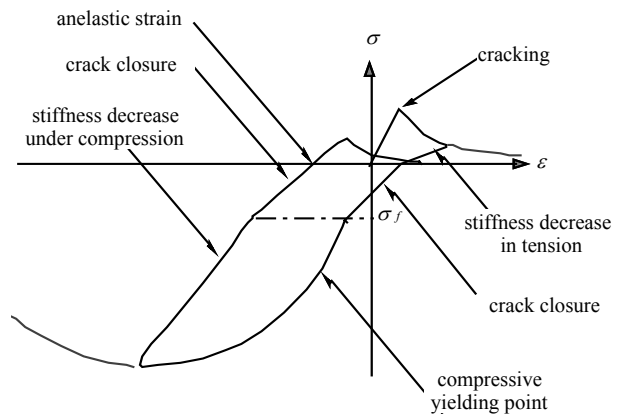


Figure 6. 1D cyclic response of the La Borderie model

The reinforcement steel is modelled with an isotropic cinematic hardening law. Constitutive model for concrete under cyclic loading ought to take into account some observed phenomena, such as decrease in material stiffness due to cracking, stiffness recovery which occurs at crack closure and inelastic strains concomitant to damage. To simulate this behaviour we use a damage model with two scalar damage variables one for damage in tension and one for damage in compression (La Borderie, 1991). Unilateral effect and stiffness recovery (damage deactivation) are also included. Inelastic strains are taken into account thanks to an isotropic tensor.

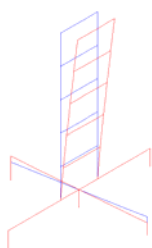
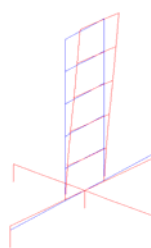
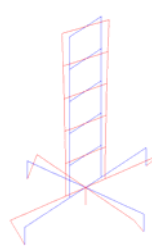
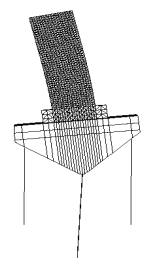
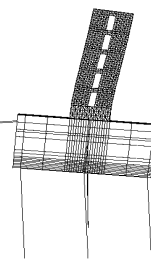
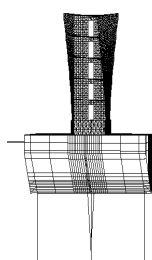
4 MAIN RESULTS

4.1 Eigenfrequencies

The modal analysis has been performed with both models in order to insure that the boundary conditions and the distribution of the masses are well represented. The stiffnesses of the springs below the shaking table are identified to fit the first eigenmodes measured on the virgin structure before the test. Table 3 gives a comparison of the two approaches with the experimental results for the first three natural modes. A part for the mode of torsion, for which the refined model is most certainly better,

the comparison of the results between calculation and experiment is of good quality.

Table 3. Modal analysis.

Model	In plane (direction X)	Out of plane (direction Y)	Torsion
Fiber model	 4.54 Hz	 7.0 Hz	 11.0 Hz
3D model	 4.5 Hz	 7.06 Hz	 9.9 Hz
Experiment	4.5 Hz	7.13 Hz	not known

4.2 Dynamic response analysis and comparison

For the refined model numerical analyses have been performed using CASTEM2000 code. All the seismic signals applied to the specimen were considered in chronological order.

First comparisons presented in Figure 7 and 8 concerns relative horizontal displacements in X and Y directions corresponding to the T5 (0.74g) input motion, which caused significant damage to the specimen.

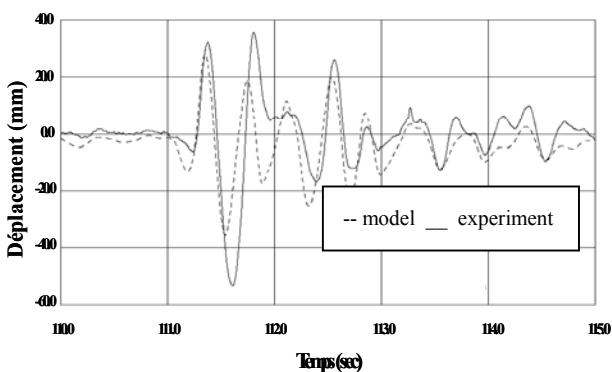


Figure 7. Refined model, comparison between calculated and measured horizontal relative top - X displacement for T5 motion (0.74g)

As can be seen, the calculated response is generally underestimated in both directions, but the correlation between analysis and experiment seems to be

better for the X direction as compared to the Y direction. This may be due to the fact that the axial stiffness of the vertical rods supporting the shaking table may evolve during seismic response. It is difficult to take into account this aspect, when the variation of the axial stiffness of the rods is not known in advance. However, even if the numerical results do not match exactly the experimental ones, they give the opportunity to highlight some important characteristics of the structural behaviour.

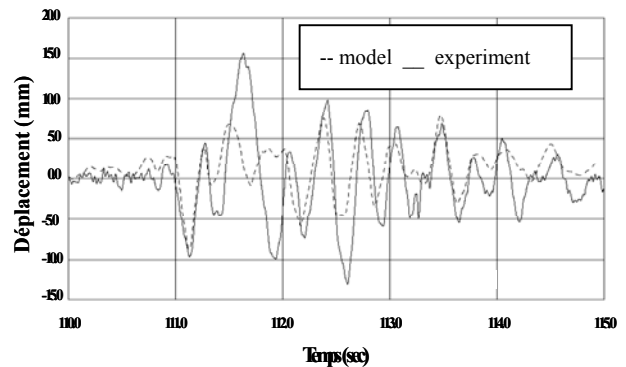


Figure 8. Refined model, comparison between calculated and measured horizontal relative top - Y displacement for T5 motion (0.74g)

The numerical results of the simplified model (code Aster) are presented in Figure 9 and 10. The time history of the calculated roof displacement is compared with the corresponding measured displacement for the T0, T5 and T6 tests. Simulation predicts satisfactory the maximum displacement for both sequences and there is no significant shifting between the curves.

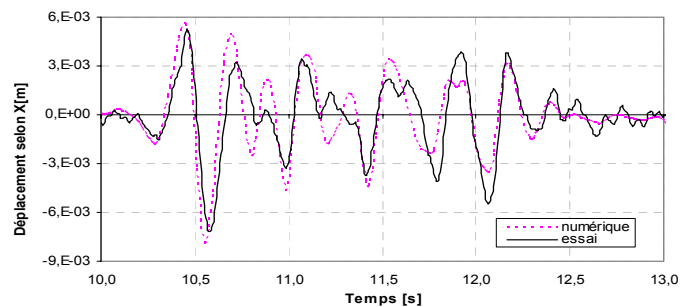


Figure 9a. Simplified model, comparison between calculated and measured horizontal relative top-X displacement for T0 motion (0.3g on X)

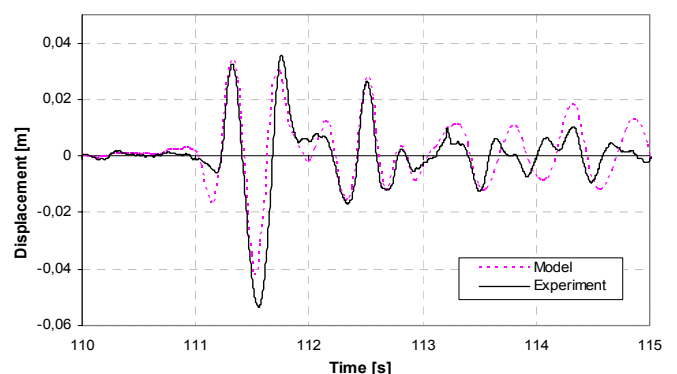


Figure 9b. Simplified model, comparison between calculated and measured horizontal relative top-X displacement for T5 motion (0.74g on X and 0.36g on Y).

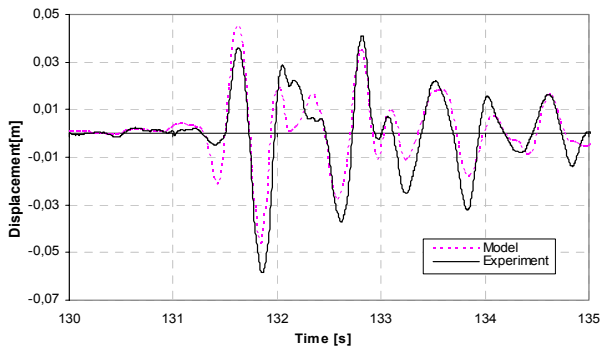


Figure 10a. Simplified model, comparison between calculated and measured horizontal relative top-X displacement for T6 motion (0.85g on X and 0.50g on Y)

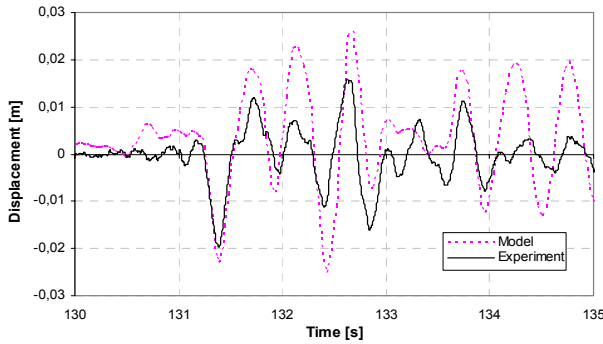


Figure 10b. Simplified model, comparison between calculated and measured horizontal relative top – Y displacement for T6 motion (0.85g on X and 0.50g on Y)

Table 4 gives the maximum values of some typical results obtained from the time history analysis (test T6):

Table 4. Maximum values for the time history analysis

Test T6	Refined model	Simplified model	Experiment
Top relative displacement (dir. X) [mm]	53.6	46.24	58.21
Top relative displacement (dir. Y) [mm]	13.4	26.01	19.8
Bending moment at the base (dir. X)- left wall [kN.m]	249	245	299
Bending moment at the base (dir. X)- right wall [kN.m]	214	244	265
Shear force at the base (dir. X)- left wall [kN]	85	74	111
Shear force at the base (dir. X)-right wall [kN]	91	63	128
Variation of axial force at the base, left wall [kN]	-224 +127	-257 +126	-
Variation of axial force at the base, right wall [kN]	-195 +132	-314 +97	-

An interesting point is the proximity of the results given by the two calculations for the X analysis (dashed line of the figures 7 and 9b); for the Y direction the discrepancy for both calculations with the experimental results is due to the difficulty to simu-

late accurately the real behaviour of the table. Complementary analysis are necessary to improve that part of the work.

Local results obtained from the dynamic analysis are seen in Figure 11 (refined model). This figure depicts the damage distribution corresponding to the maximum top displacement attained in the X direction for the T5 applied motion. The analysis results indicate that more damage is to be expected in the X walls as compared to the Y wall. They also show large compressive strains at one end of the X wall, indicating that concrete may fail in this location due to excessive strains. Actually, this seems to be in reasonable agreement with what was experimentally observed (Figure 12): the wall extremities were heavily damaged in compression and steel bars buckled and have broken after that at this location.

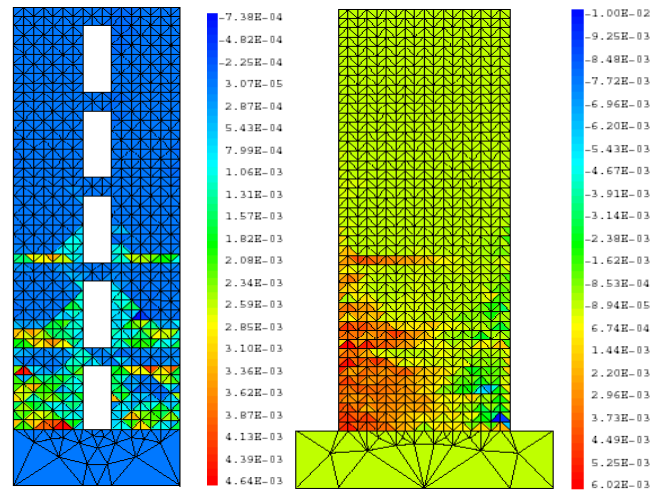


Figure 11. Refined model, vertical concrete strain contours for T5 motion

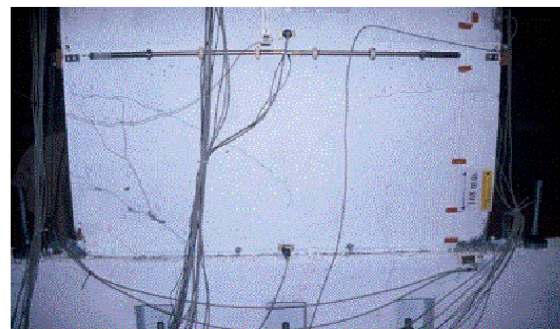


Figure 12. Experimental damage pattern for T5 motion.

Figure 13 presents numerical results in terms of bending moment – axial force interaction diagrams at the base of the 1st storey together with the variation of axial load and moment. This confirms the observed behaviour and failure mode, since limit states tend to be obtained with high axial force values.

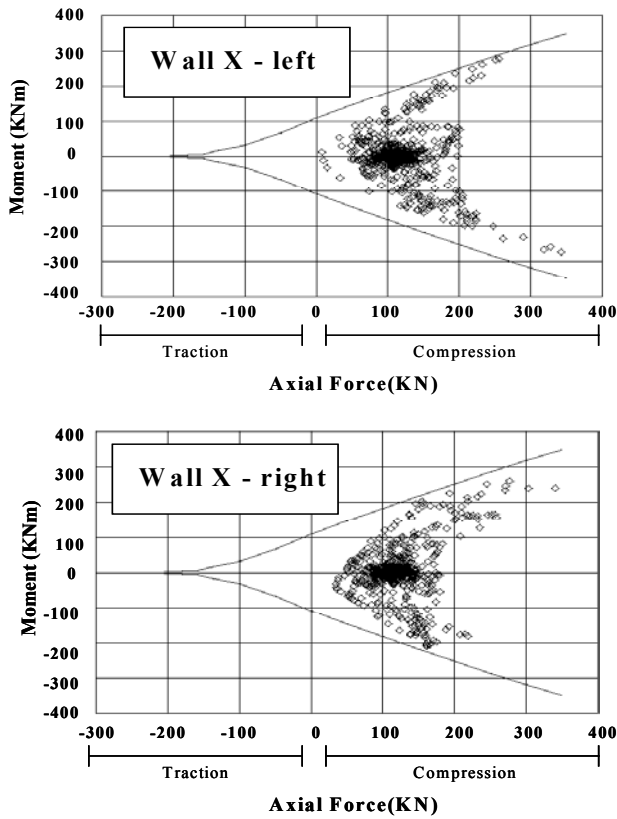


Figure 13. Refined model, bending moment- Axial force interaction diagrams and variation of the bending moment and axial force at the base of the 1st storey (T5 motion)

Figure 14, 15 shows the strains of reinforcement for T4 and T5 levels of loading. The model is improved to described progressive broken effects of reinforcements : buckling and failure, as shown for T5 on figure 15.

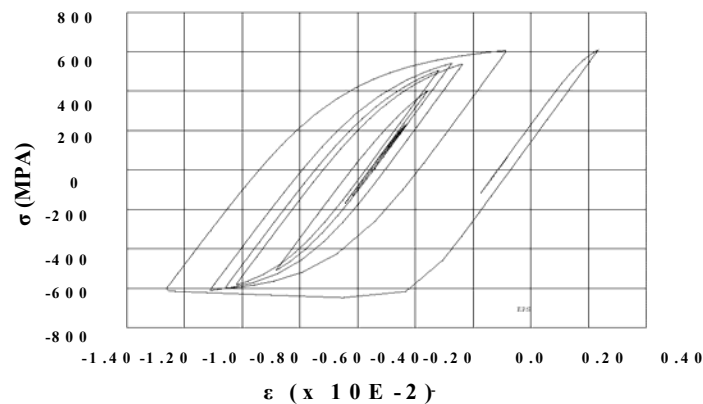


Figure 14. Refined model, steel strain for T4 motion (left wall)

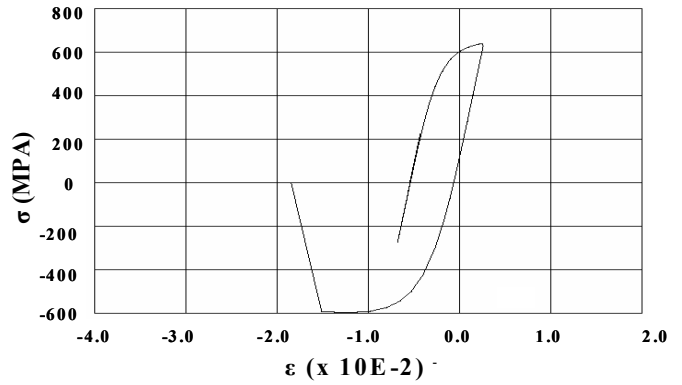


Figure 15. Refined model, steel strain for T5 motion (left wall)

For the simplified model, the damage variable vary normally between 0 (non damaged section) and 1.0 (completely damaged section). By filtering their values between 0.95 and 1.0 we omit the micro-cracks and we have a picture of the visible cracks of the model. Figure 16 presents the damage pattern due to tension at the end of the calculation for the T5. This picture gives the zone where visible cracks are located.

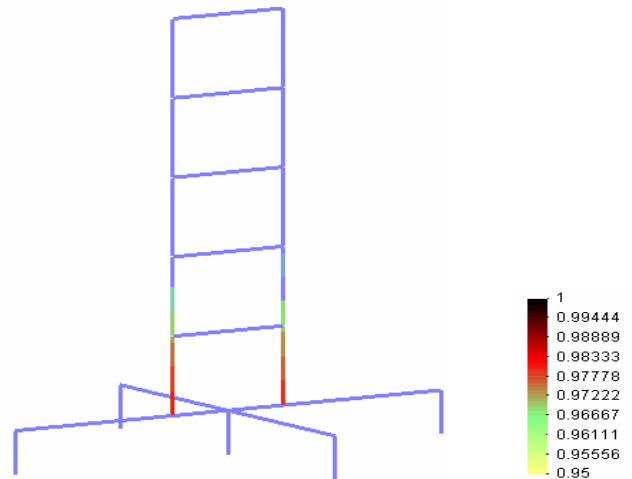


Figure 16. Simplified model, state of damage for T5 motion

Comparison of the distribution of damage (Figure 17) and strains in the R-bars (Figure 18) with the actual position of cracks (Figure 19 and 20) shows again that the model is able to reproduce qualitatively the local behaviour observed experimentally.

Main damages are located at the base of the specimen. As shown on Figure 18 the simplified model gives for reinforcement at the base of the structure, strains greater than 2%. Previous tests performed on a sample of R-bar shown that failure occurs before this value. Then this result is in good accordance with the test where it was observed that R-bars were broken (Figure 20). However the model must be improve to include the progressive broken effect of reinforcement (debonding and sudden loss of strength).

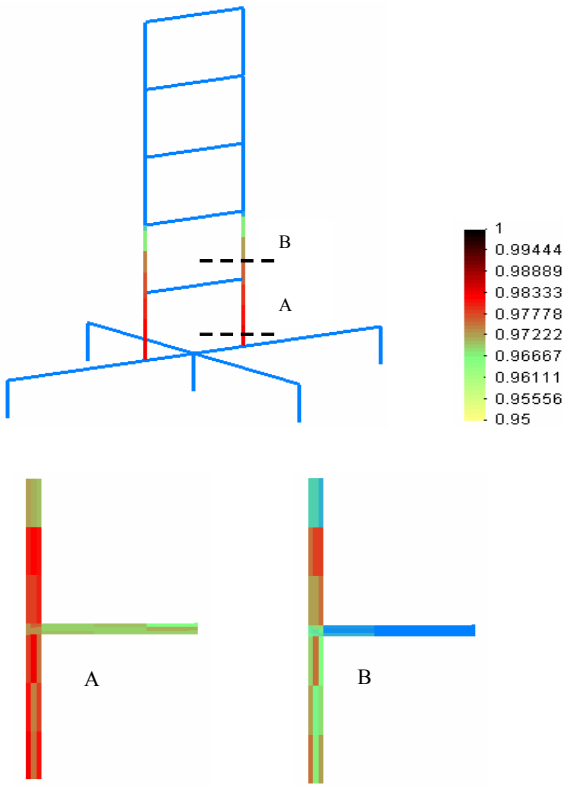


Figure 17. Simplified model, state of damage for T6 motion

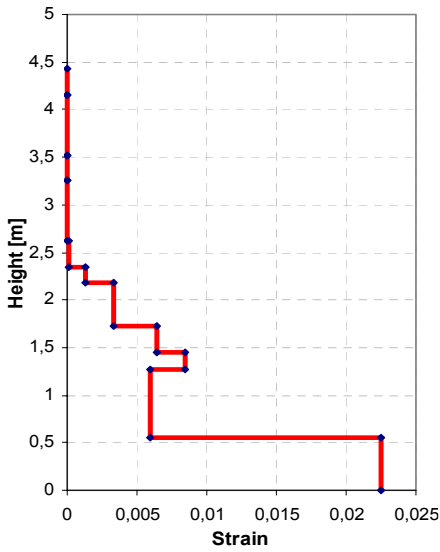


Figure 18. Simplified model, strain of reinforcement at the base of the wall left (T6)



Figure 19. Experimental damage pattern for T6 motion



Figure 20: Broken steel at the base of the walls X right (T6)

5 EFFECTS OF THE APPLICATION OF THE LOAD IN MULTIPLE DIRECTIONS

In order to understand better the effects of a 3D earthquake in the global and local behaviour of the structure, different parametrical studies are performed according to the tests sequences given in table 4.

Table 4. Sequences of the numerical tests

Tests	Direction X	Direction Y	Direction Z
A1	0.5 g	-	-
A2	0.5 g	0.24 g	-
A3	0.5 g	0.24 g	0.35 g
B1	0.85 g	-	-
B2	0.85 g	0.50 g	-
B3	0.85 g	0.50 g	0.595 g

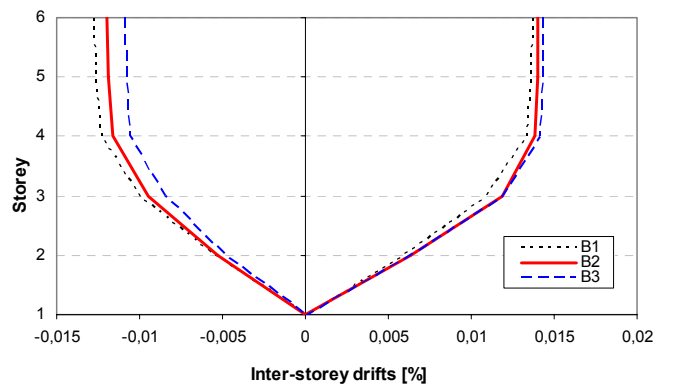
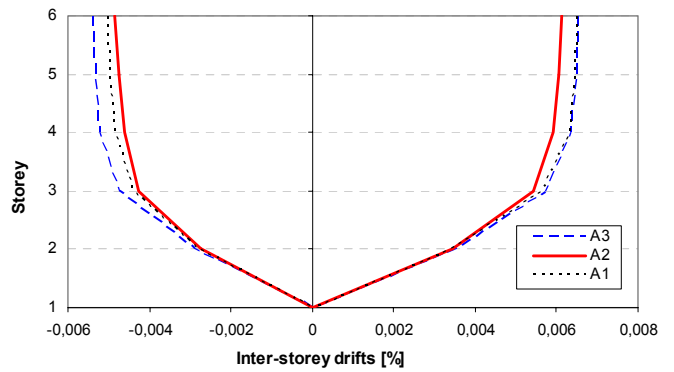


Figure 21. Simplified model, effects of the loading conditions, inter-storey drifts (left wall)

The inter-storey drifts predicted by the numerical model for all the sequences are shown in figure 21. The introduction of the second horizontal excitation clearly introduces an asymmetric behaviour of the structure.

The quantity more influenced by the loading conditions is the variation of the axial force (Figure 22). The results show that a seismic load in the direction perpendicular to that of the walls increases the variation of the axial load. In the present case the total amplitude of variation can be of the order of three times the weight of the specimen (15.8 tons for a wall). More specifically, due to this effect the walls are subjected to a complete unload (even to a positive load, i.e. tension), or to a total vertical load equal to the double of the dead load.

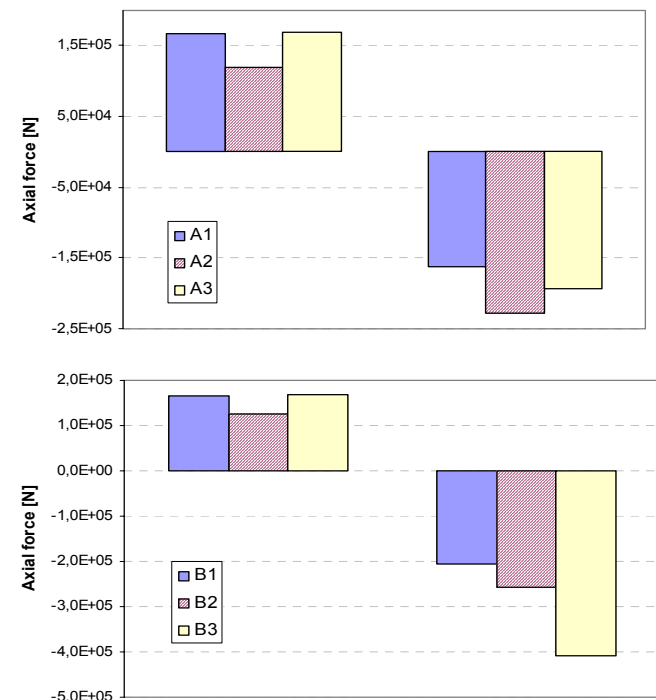


Figure 22. Simplified model, effects of the loading conditions (1, 2 or 3 directions of loading) on the value of axial load on the left wall – dynamic effect.

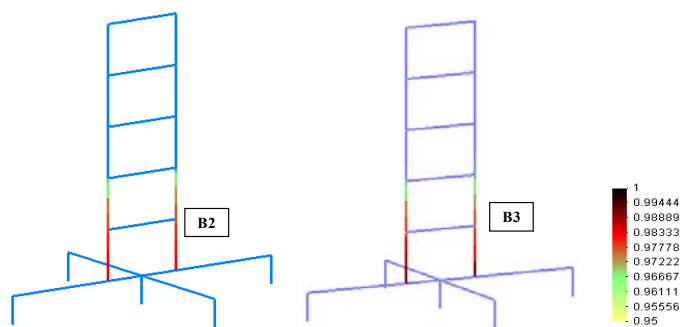


Figure 23. Simplified model : effects of the loading conditions showing an increase of damage when a vertical loading is added (B3) to a bi-directional seismic loading state (B2=T6).

Local results as obtained from the dynamic analysis are seen in figure 23 (simplified model). A picture is given for the location of bigger cracks in both situa-

tions : B2 corresponding to the T6 level previously studied and B3 where a vertical loading is added to the T6 situation, showing the amplification of damage at the base of the specimen mainly due to the increase of the amplitude of variation of the axial load observed figure 22.

6 CONCLUSIONS

As demonstrated by the results presented in the paper it clearly appears that modelling is relevant to reproduce with good approximation the global response of the structure and qualitatively the distribution of damage.

More precisely one could notice that the two models (simplified and refined) make possible to correctly predict the main aspects of the experimental behaviour observed during experiment. It appears that only non-linear dynamic tools are able to evaluate accurately the margin of security for a given structure.

Of course, the refined model, based on shell description allows obtaining more precise detailed informations at the local level, whereas the simplified approach, based on Timoshenko multi-fiber beams, helps reducing time to prepare, to interpret calculation and for computation. In that framework the simplified modelling is useful for parametric studies in order to enlarge the results issued from the experiment, such as the effect of different loading situations as shown in this paper.

Both models highlight the importance of the axial force variation and its influence on the failure mode to be expected in the case of lightly reinforced walls commonly used in France and other countries in Europe. Strongly linked with boundary conditions this aspect of the work must be improved introducing realistic soil-structure interactions. On this topic research is in progress at 3S Grenoble in order to introduce a 3D interface element taking in charge the main phenomena at the level of foundations of the structure (elasto-plasticity of soil, uplift,...- cf. Grange 2005).

At the local level it appears that the risk of buckling is clearly described by the refined model. Such a tool could be used to help engineers to improve local layout introducing for example stronger confinement layout into the zones submitted to high compression stress.

Another point not debated in this paper is the description of damping. The results presented here has been obtained through the use of a Rayleigh type damping law with a rate of damping equal to 1%. However it has been shown that damping is mainly

due to hysteretic loops related to friction effects at the crack level during local cyclic loading (Ragueneau et al. 1999) and the improvement of the modelling in that way is another issue of the work in progress.

As a conclusion one can say that this work follows up the previous researches performed on such type of structures in France (CASSBA, CAMUS - cf. Mazars 98) and at the European level (ICONS, SAFERR, LESSLOSS cf. Kotronis et al. 2005b). Modelling for 3D situation is now under control and a complete simplified modelling able to help earthquake engineering is on the way.

7 ACKNOWLEDGEMENTS

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