SEISMIC BEHAVIOUR OF R/C SHEAR WALL STRUCTURES DESIGNED ACCORDING TO THE FRENCH PS92 AND EC8 CODES: A COMPARISON BETWEEN SHAKING-TABLE RESPONSE DATA AND 2D MODELLING

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SUMMARY

The provisions of the EC8 do not cover an important class of multi-storey shear walls structures, that of reinforced concrete bearing walls with limited reinforcement ratio, which are commonly used in France for buildings. In view of such analysis an experimental programme took place in the CEA facilities in Saclay which consisted of shaking-table tests on slightly and normally reinforced concrete shear wall structures (CAMUS I and CAMUS III). The CAMUS I specimen had the vertical, horizontal and confinement reinforcement according to the French PS92 seismic design code. The geometry of CAMUS III was the same than the CAMUS I specimen, but the reinforcement was designed according to EC8. The objective of this study is to evaluate through comparison with the experimental results, the performance of various FEM analytical techniques. Towards the analysis of the damage and behaviour of this shear wall, a plasticity based concrete model is used. Bond-slip interaction between steel bars and concrete is explicitly modelled through the use of special continuous contact elements. Conclusions regarding the quality of the numerical results, specific aspects of the seismic behaviour of this structure and further research needs are also included.

INTRODUCTION

The design of structures to withstand earthquake excitations is a difficult problem in earthquake engineering due to the uncertainties in the evaluation of seismic hazard, the dynamic nature of the structural response and the need to insure, through energy dissipating capacity of the structure, their survival under strong earthquakes. The development of Earthquake Engineering in the last years have shown a multiplicity of very sophisticated methods of dynamic analysis and their use has been gradually accepted in the earthquake regulations, namely Eurocode No.8 (EC8). However, the provisions of the EC8 do not cover an important class of multi-storey shear walls structures, that of reinforced concrete bearing walls with limited reinforcement ratio, which are commonly used in France for buildings. As limited researches have been done until now, further research concerning the behaviour of this type of structure is needed. In view of such analysis an experimental programme took place in the CEA facilities in Saclay, this project being supported by the Commisariat à l'Energie Atomique (CEA), Fédération Nationale du Bâtiment (FNB), Plan Génie Civil and Electricité de France (EDF). Shaking-table test results on a slightely reinforced concrete shear wall structure (CAMUS I) are firstly compared with numerical results which were obtained using various FEM analytical techniques. For comparative purposes, results from predictive calculations on a similar structure, but designed according to EC8 (CAMUS III) are then presented.

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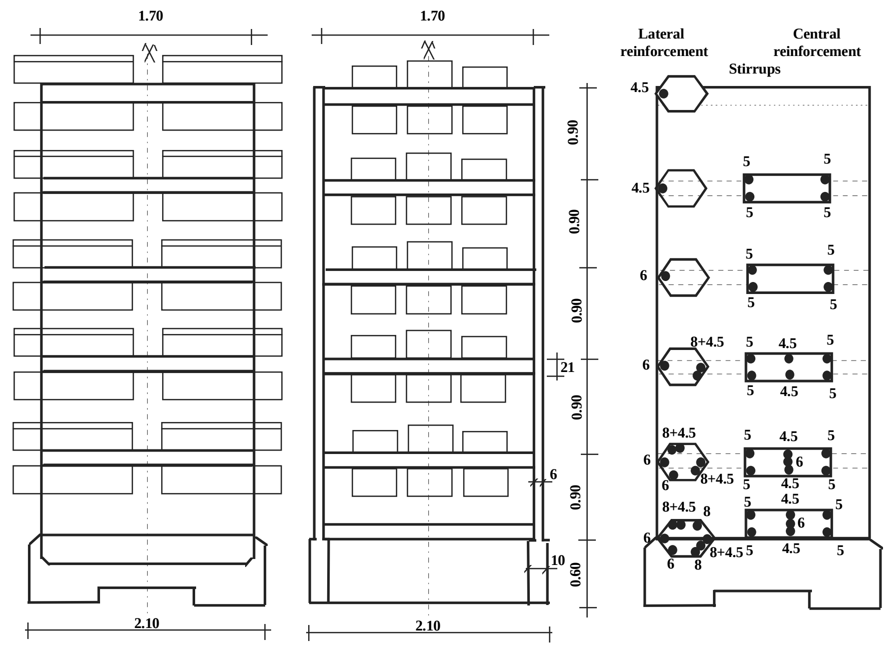
EXPERIMENTAL TEST PROGRAM

The CAMUS I specimen consist of a 1/3rd scaled model, composed of two parallel 5-floor RC walls without openings, linked together by 6 square floors, the reinforced concrete footing being anchored to the shaking table (Figure 1) [Locci et al., 1998]. The walls are each 5.10 m height, 1.70 m long and 6 cm thick. They were cast in order to reproduce the construction joint at the level of each floor. At the level of construction joint, horizontally localised cracking was already visible before the test. All the floors are 1.70 m long 1.70 m wide and 21 cm thick. The wall footing is 2.10 m, 0.60 m height and 10 cm thick. A mortar layer of 1 cm thick was provided between the shaking-table and the bottom part of the wall footing. A C20 microconcrete was used in order to reach a compressive strength of 25 MPa and a Young’s modulus equal to 28 000 MPa. The walls of the mock-up have the vertical, horizontal and confinement reinforcement according to the French PS92 seismic design code. The flexural reinforcement ratio changes 10 cm just under the lower side of each floor. Additional mass was added to the upper and lower part of each floor (excepting the first floor) in order to simulate the gravity load compatible with the vertical stress values commonly found at the base of this type of structure (about 1.6 MPa in this case). The loading program involved subjecting the mock-up to 6 sequentially horizontal accelerations along the direction parallel to the shear walls up to collapse of the mock-up. The synthetic seismic signal NICE S1 representative of the French design acceleration spectra and the SAN FRANCISCO signal were used as input signals (Table 1).

Table 1. CAMUS I loading program

TEST	TYPE OF SIGNAL	MAXIMUM HORIZONTAL ACCELERARION (g)
CAMUS02	NICE	0.24
CAMUS12	SAN FRANCISCO	0.13
CAMUS14	SAN FRANCISCO	1.11
CAMUS16	NICE	0.25
CAMUS17	NICE	0.40
CAMUS19	NICE	0.71



Side view Front view Reinforcement

Figure 1: Geometry and reinforcement of CAMUS I specimen

STRUCTURAL MODELLING

The 2-D finite element model used in the analysis represents the different parts of the mock-up and the shaking-table as an equivalent plane mesh. Four-nodded membrane elements were used to represent the shear wall, the slabs, the additional masses and the shaking table. At the level of the construction joints double nodes were provided from the beginning along the concrete interelement boundaries and unilateral imposed cinematic conditions were considered in these locations. A discrete modelling was adopted to represent the reinforcement through the use of two-nodded truss elements. Bond-slip interaction between steel bars and concrete was explicitly modelled through the use of four-nodded continuous contact elements. In accordance with the construction details a 1 cm thick four-nodded finite element layer was introduced at the base of the wall footing to model the elastic contact between the mock-up and the shaking-table. The shaking-table was considered as a rigid block fixed to 4 vertical restraining rods: 2 rods are situated at the level of the centre of the wall and the other two ones at the extremities (the axial stiffness of each rod being estimated equal to 400 MN/m). Figure 2 shows the different parts of the 2-D finite element mesh adopted in this study.

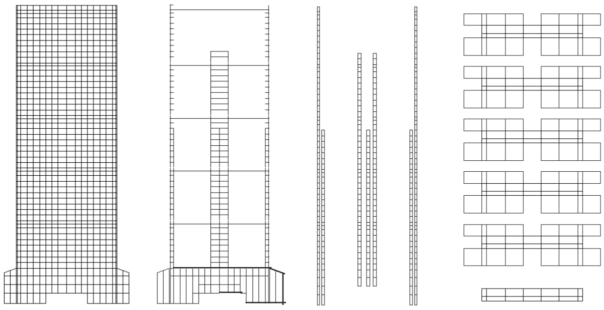


Figure 2: 2-D finite element mesh of the CAMUS I mock-up

MATERIAL CONSTITUTIVE MODELS

The concrete model used here was developed at INSA de Lyon [Merabet et al., 1995] and is based on the plasticity theory. The concrete is assumed as a softening material both in tension and in compression with a correct description of the unilateral opening and closing of cracks throughout the loading cycles. The model considers the permanent strains, the stiffness degradation and the stiffness restitution when the crack closes. The elasto-plastic strain-hardening model developed for uncracked concrete is based on the four-parameters Ottosen's failure criterion with isotropic hardening and associated flow rule. The yield criterion is assumed on the basis of the known failure criterion by selecting it as a proportionally reduced shape of the failure surface and an associated flow rule is employed. For the concrete in tension a smeared fixed crack approach is considered. With this formulation the cracked concrete is treated as an orthotropic material with principal axes normal and parallel to the crack direction and uniaxial behaviour is considered in these two directions according to Figure 3. Rough crack behaviour is incorporated in the model transmitting shear forces across the crack. According to the constitutive cyclic law for concrete, as soon as a crack starts to close, the concrete develops some compression, due to the imperfect overlapping of the crack surfaces. Furthermore the model considers damage of the elastic modulus and of the tensile resistance as the inelastic compressive strains increase.

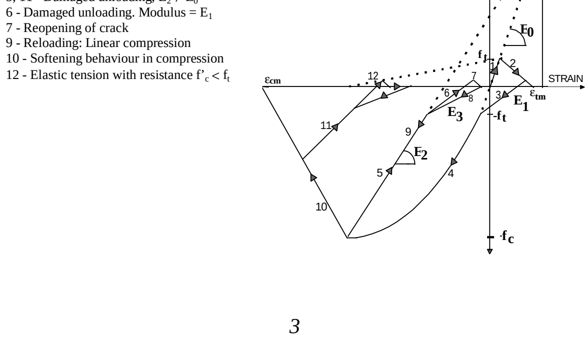


Figure 3: Concrete cyclic law

For all the wall concrete elements an initial elastic modulus of 30 000 MPa, Poisson coefficient of 0.2, tensile strength of 2.4 MPa and compressive strength of 30 MPa were considered. The stress-strain relationship of reinforcement was considered as for an isotropic hardening material. The material characteristics were based on the tensile test results for the different reinforcing steel bars used in the construction of the mock-up (yield stress f_y = 467 - 515 MPa, elastic modulus Ea = 200,000 MPa).

The bond behaviour under alternating loading is described through the law proposed by Eligehausen et al. [Eligehausen et al., 1983] and implemented by Fleury in the CASTEM 2000 code [Fleury, 1996]. This law is based on a theory of the mechanisms that govern the resistance and degradation of bond, and is thus relatively general to all classic reinforced concrete problems, even if the identification of its six parameters has to be based on tests that represent the particular boundary problem considered. The characteristics of the law are illustrated in Figure 4 in which a typical cycle (corresponding to a well-confined region) is shown. The behaviour under a monotonic load is described by the curve OABCD or $OA_1B_1C_1D_1$ and is entirely defined by the knowledge of s_1, s_2, s_3, τ_1, τ_3, and α. This curve is composed of four segments:

- a non-linear ascending branch, its equation being given by $\tau = \tau_1.(s/s_1)^{\alpha}$.
- the ascending branch is followed by a plateau for $s \in [s_1, s_2]$, defined for $\tau = \tau_1$.
- a descending branch which is linear up to point C of co-ordinates (s_3, τ_3).
- a last segment which defines the friction plateau, $\tau = \tau_3$.

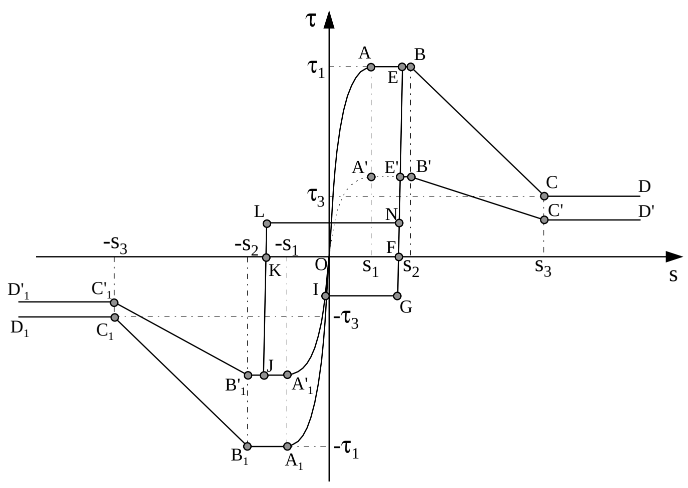


Figure 4: Cyclic bond law

As bond test results on CAMUS wall specimen were not available, two different cases were considered through a parametric study: CASE 1 representing code prescribed values for bond-slip and CASE 2 which assumes "weak bond-slip " values for the elements connecting the lateral extreme reinforcement to the surrounding concrete. CASE 2 is supposed to account for the effects of a cracked cone formation and for steel yielding in tension.

ANALYSIS OF RESULTS

Non-linear behaviour was assumed for the entire wall structure (concrete, steel and interface elements), while the behaviour of the finite elements representing the slabs, the additional masses, the shaking table and contact layer was taken as elastic. A viscous mass and stiffness dependent damping factor of 2% and 4% was assumed with reference to the first and second natural frequencies. At the level of the construction joints unilateral imposed cinematic conditions were considered. The following five artificial motions were considered: CAMUS02, CAMUS14 and CAMUS19. For each input motion three parametric studies were undertaken, corresponding to perfect bond between steel and concrete, code prescribed values for bond (CASE 1) and weak bond values (CASE 2). The results corresponding to CAMUS02 and CAMUS14 will not be detailed for sake of brevity. For these input motions local and global results corresponding to perfect bond are practical identical to CASE1 and CASE2 results. It is also to be mentioned that structural damage was not very important for these input motions. The CAMUS19 (0.71g) seismic input motion caused failure of the specimen, by extensive yielding and fracture of the lateral reinforcement at the level of the 3rd and 4th floor. Strain gages situated just above the level of the construction joint indicated high strain values at these levels on the one hand, and much lower values at the level corresponding to the 1st and 2nd floor, on the other hand. The calculated top-displacement time-history is almost identical for all the three cases under analysis and only the result corresponding to perfect bond is presented in comparison with the measured displacement (Figure 5). Good agreement is obtained between the calculated top displacement and the measured one (the difference between the max. calculated and max. measured values is equal to about 5%). Examination of the moment-curvature relationships in Figure 6 and Figure 7 shows larger hysteresis loops for the 3rd floor where failure occurred as compared to the 1st and 2nd floor levels where the loops are very pinched. The calculated moment-curvature relationships being similar to those experimentally deduced it can be concluded that the 2-D model gives good failure location.

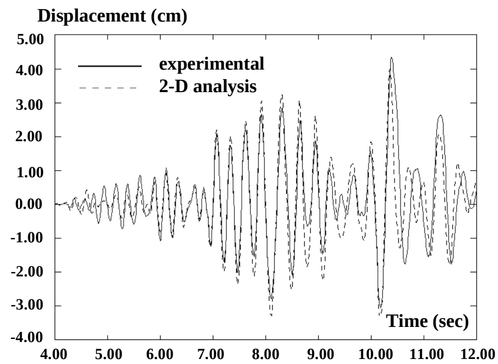
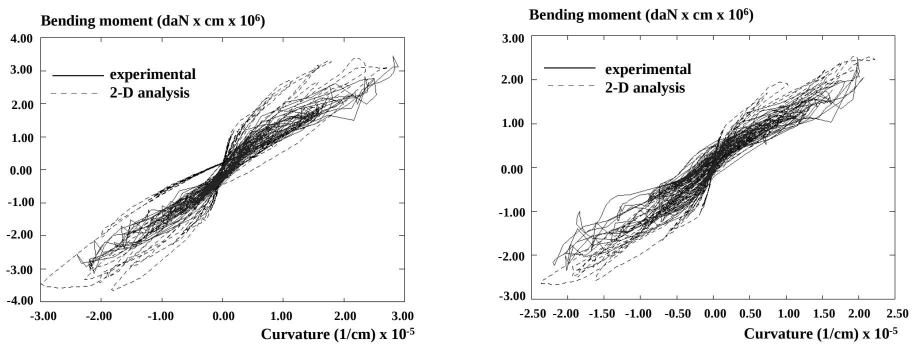


Figure 5: Time-history of horizontal top-displacement for CAMUS19 input motion



1st floor level 2nd floor level

Figure 6: Moment-curvature relationships at 1st and 2nd floor levels (perfect bond assumption)-CAMUS19 input motion

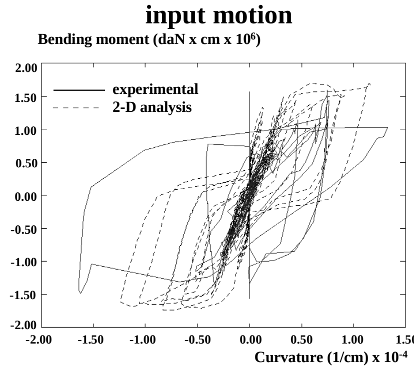
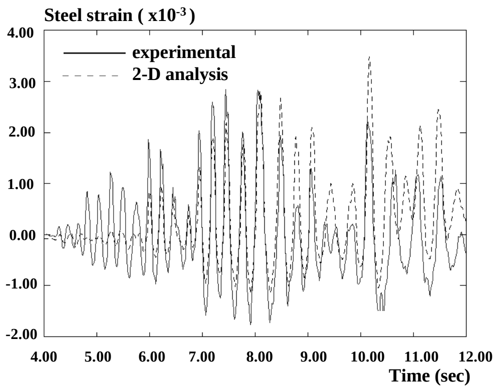
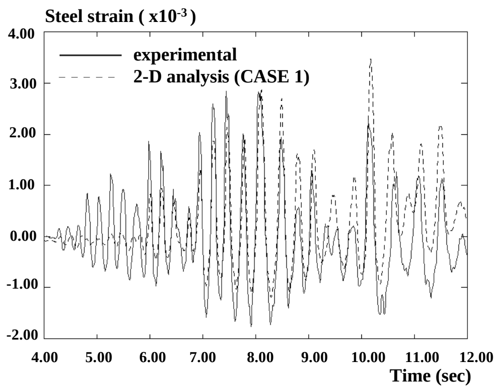


Figure 7: Moment-curvature relationship at 3rd floor level (perfect bond assumption)-CAMUS19 input motion

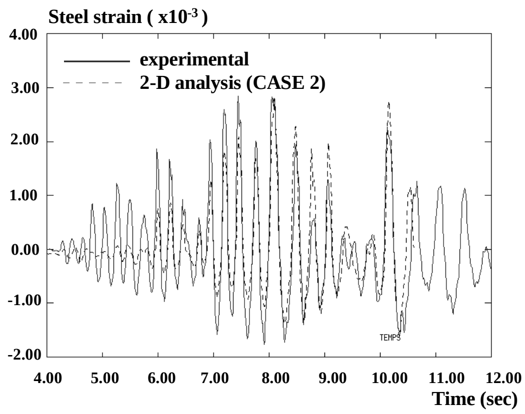
As a general rule, maximum strain distribution in steel over the height of the wall conforms to that experimentally measured. However, assumption of perfect bond between steel and concrete leads to strain overestimation in the lateral steel located at the construction joints of the 1st and 2nd floor. As shown in Figure 8, bond interaction reduces the steel strain values because of the slippage occurring at the connection points between steel and concrete. The maximum strain reduction is obtained in CASE 2 (weak link). These results indicate that local behaviour prediction may be improved if steel-concrete bond is taken into account and suitable values for the parameters affecting the bond-slip low are selected. At this stage, it is believed that this type of structure needs further experimental studies to improve the knowledge of steel-concrete bond, and to help in selecting better parameters for the bond-slip constitutive law.



Perfect bond



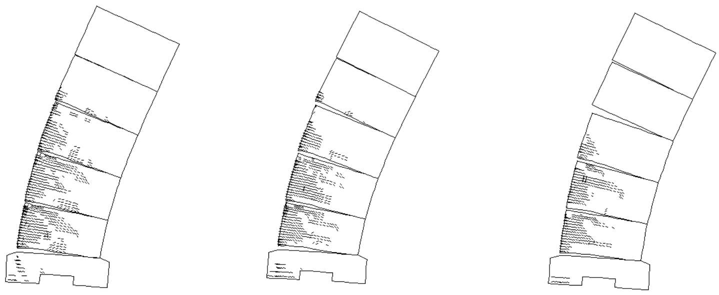
Code prescribed bond (CASE 1)



Weak bond (CASE 2)

Figure 8. Time-history of steel strain (1st floor construction joint) corresponding to CAMUS19 input motion

The cracking distribution corresponding to the positive displacement peak during the test at failure is shown in Figure 9. Predicted crack pattern indicates a more pronounced crack opening at the level of construction joints than along the interstorey height. Concentration of cracking at the level of the construction joints was equally experimentally detected, but it is to be noted that interstorey cracking is somewhat overestimated by the 2-D FEM calculations. Nevertheless, as shown in Figure 9 the CASE 2 (weak bond model) leads to reduced interstorey cracking due to partially energy dissipation through steel-concrete friction. It is to be remembered that construction joints were represented in the present calculations by unilateral imposed cinematic conditions between pairs of nodes at opposite sides of the construction joints. When major discontinuities predetermine the location of dominant cracks (which is the case of CAMUS shear wall) the best way of modelling the problem seems to be to combine the discrete cracks at the predefined location with a smeared crack approach for the rest of the structure. However, as present calculations have not taken into account properly the interface behaviour, an interface element capable of expressing the dependence of the shear stress along the crack on shear slip and of cohesive stress across the crack on crack with, has to be developed.

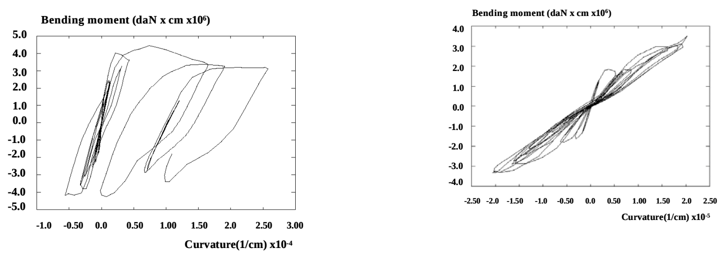


Perfect bond Code prescribed bond (CASE 1) Weak bond (CASE 2)

Figure 9: Calculated crack pattern corresponding to camus19 input motion

NONLINEAR DYNAMIC ANALYSIS OF EC8 DESIGNED STRUCTURE

The geometry of CAMUS III is the same as that corresponding to CAMUS I specimen, but the reinforcement is designed according to EC8 in order to have the same ultimate bending moment at the base as CAMUS I. Following the capacity design principles, the critical region is the "plastic hinge region" at the base of the wall, this region being carefully detailed for inelastic flexural actions. In all storeys above the ground storey, the wall is then overdesigned in flexure and shear to ensure an elastic behaviour of the upper regions. Predictive results of CAMUS III behaviour are based on a dynamic analysis which is similar to that performed for CAMUS I. However, only the case corresponding to perfect bond between steel and concrete is considered in the analysis. Opposite to CAMUS I, failure is expected to happen at the base of the wall. This can be visualised on Figure 10 where the moment-curvature relationships at the base of the first and second storey are represented. The predicted crack pattern also indicate that opposite to CAMUS I the expected damage is almost entirely concentrated near the base (Figure 11).



1st floor level 2nd floor level

Figure 10: Moment-curvature relationships for CAMUS III at different levels-CAMUS19 input motion

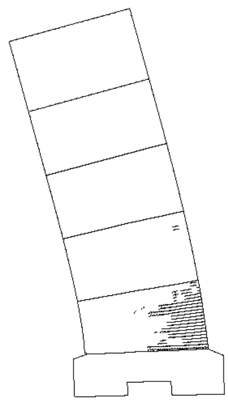


Figure 11. Calculated crack pattern corresponding to CAMUS III - CAMUS19 input motion

CONCLUSIONS

The numerical results indicate that a refined 2-D finite element model can provide a correct description of the measured dynamic behaviour of a slightly reinforced shear wall structure. The global response of the mock-up and the location of failure were simulated with sufficient accuracy. Construction joints were represented by unilateral imposed cinematic conditions between pairs of nodes at opposite sides of the construction joints. When major discontinuities predetermine the location of dominant cracks (which is the case of CAMUS I shear wall) the best way of modelling the problem seems to be to combine the discrete cracks at the predefined location with a smeared crack approach for the rest of the structure. However, as present calculations have not taken into account properly the interface behaviour, an interface element capable of expressing the dependence of the shear stress along the crack on shear slip and of cohesive stress across the crack on crack with, needs to be developed. The accuracy in the description of the measured local response can be improved by selecting better steel-concrete bond characteristics for the constitutive laws to be used in the finite element calculations. The database for this evaluation is actually still too limited to allow their determination with enough precision and further studies are badly needed. Opposite to a slightly reinforced shear wall structure, failure of a shear wall structure designed according to EC8 is predicted at the base of the wall. Load and displacement capacity predicted are similar to those observed for CAMUS I, but is seems that reinforcement ratio in upper stories, adopted to control higher order mode effects, are rather conservative.

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