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A new simplified modelling strategy to simulate the non-linear behaviour of reinforced concrete shear walls under dynamic loading is presented. The equivalent reinforced concrete (ERC) model is derived from the framework method and uses lattice meshes for concrete and reinforcement bars and uniaxial constitutive laws based on continuum damage mechanics and plasticity. Results show the capacity of the model to analyse structures having different slenderness and boundary conditions. For low reinforcement ratios however, results are sensitive to the angle formed by the diagonals of the concrete lattice and the horizontal bars. The method is compared with the shear multi-layered beam model that uses Timoshenko multi-layered 2D beam elements and biaxial constitutive laws. Comparisons for both models with experimental results of two research programs (one organized by NUPEC and the other by COGEMA and EDF) are provided. ERC is a simplified method that intends to save computer time and allows parametrical studies.

KEY WORDS: equivalent reinforced concrete model; shear multi-layered beam model; framework method; lattice mesh; shear walls; continuum damage mechanics; dynamic loading

1. INTRODUCTION

The simulation of the non-linear behaviour of load bearing walls made of reinforced concrete under dynamic loading is an important problem for the engineering community. Recent earthquakes in Kobe (Japan), Izmit (Turkey) and Athens (Greece) have shown the key role that such structural elements play for the safety of buildings. Since they act as major earthquake resisting members, their dynamic characteristics dominate the response of the structure. It is therefore crucial to develop effective and easy to use modelling strategies for different types of reinforced concrete walls.

The Laboratoire de Mécanique et Technologie (LMT) has already proposed a simplified modelling strategy for bearing walls dominated by flexure. It consists of using multi-layered

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2D Bernoulli-beam elements coupled with continuum damage mechanics [1–3]. The problem is, however, more complicated to solve when the effects of shear deformation are prevailing, as for example the case of shear walls (bearing walls that have a small slenderness, usually less than 1.5). On this specific point two recent experimental programs, one performed by the Japanese company NUPEC (Nuclear Power Electric Corporation) [4] and the other by Eléctricité de France (EDF) and COGEMA ('SAFE' experimental program) [5, 6], gave valuable information on the seismic behaviour of shear walls with different reinforcement ratios, boundary conditions and slenderness.

The purpose of this paper is to propose an original and simplified modelling strategy to reproduce the non-linear behaviour of shear walls under dynamic loading. After a short review of relevant research work we recapitulate the mathematical tools and concepts used in a simplified analysis, an essential tool for parametric computations. We apply the shear multi-layered beam model (SMB) to mock-ups of the two experimental programs and demonstrate its limitations and the need for an alternative approach. The derivation of the new model, called equivalent reinforced concrete model (ERC) follows. The model, deduced from the framework Method [7], uses a lattice mesh and constitutive laws based on continuum damage mechanics and plasticity. Finally, a discussion on the effect of the angle of the lattice diagonals—a major parameter of the model on the structural response—and some conclusions are presented.

2. DESIGN AND MODELLING THE SHEAR BEHAVIOUR OF REINFORCED CONCRETE: PREVIOUS WORK

Since the early 1900s, truss models have been used as conceptual tools in the analysis and design of reinforced concrete beams. Ritter [8] and Mörsch [9] postulated that after a reinforced concrete beam cracks (due to a diagonal tensile stress field), it can be described by a parallel chord truss with diagonals inclined at 45° with respect to the longitudinal axis of the beam. Truss models for shear and torsion having a variable angle of inclination were presented by Kupfer [10]. Schlaich *et al.* [11] introduced the 'strut-and-tie' model. Collins and Mitchell [12] abandoned the assumption of linear elasticity and developed the 'compression field theory' (CFT) and Hsu [13] the 'rotating angle softened truss model' (RA-STM). The reader can find a detailed presentation of these procedures in the ASCE-ACI Committee on Shear and Torsion Report [14]. Different lattice models have been published recently [15–17] to analyse the shear behaviour of reinforced concrete under cyclic or dynamic loading. Our purpose is to work on the same ideas by using specific concepts developed at LMT to simulate the non-linear behaviour of reinforced concrete structures.

3. NUMERICAL TOOLS AND CONCEPTS FOR A SIMPLIFIED ANALYSIS

To perform non-linear dynamic calculations the code EFICOS was developed at LMT. EFICOS uses 2D beam elements divided into several layers. A constitutive law is chosen for each layer and a seismic load history is applied as an input motion at the base of the structure. The code combines the advantage of using structural elements with the simplicity of a true uniaxial behaviour or an enhanced uniaxial behaviour including shear [18, 19].

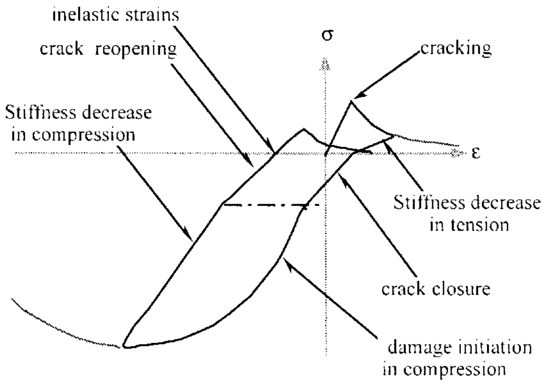


Figure 1. 1D response of the unilateral damage model for concrete.

3.1. Constitutive laws

The constitutive law used for concrete is based on the principles of continuum damage mechanics [20, 21]. Seismic loading, which includes cyclic aspects, produces micro-cracking in concrete. The major phenomena—decrease of material stiffness as the micro-cracks open, stiffness recovery as the cracks close and inelastic strains induced by damage—have to be considered. The model, called hereafter 'unilateral damage model' and presented in his 1D version, is suitable for this kind of loading and involves two damage scalar variables, one in tension D_1 and one in compression D_2 and the description of isotropic inelastic strains (Figure 1). The total strain is given by Reference [21]

$$\varepsilon = \varepsilon^{\mathrm{e}} + \varepsilon^{\mathrm{in}}$$

$$\varepsilon^{\mathrm{e}} = \frac{\sigma_+}{E(1-D_1)} + \frac{\sigma_-}{E(1-D_2)}$$

$$\varepsilon^{\mathrm{in}} = \frac{\beta_1 D_1}{E(1-D_1)} + F(\sigma)\frac{\beta_2 D_2}{E(1-D_2)}$$

where ε^{e} the elastic strain and $\varepsilon^{\mathrm{in}}$ the inelastic strain, E is the initial Young's modulus and where the positive part and negative part of the stress are expressed with

$$\sigma > 0 \rightarrow \sigma_+ = \sigma, \quad \sigma_- = 0$$

$$\sigma < 0 \rightarrow \sigma_+ = 0, \quad \sigma_- = \sigma$$

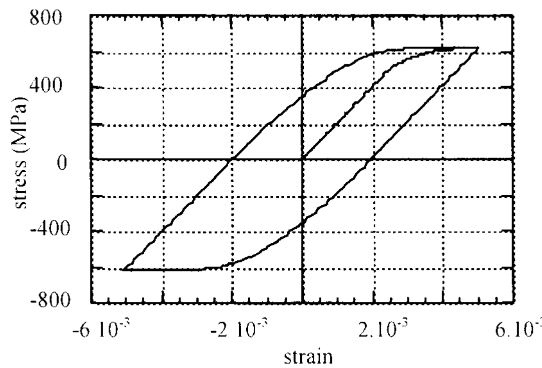


Figure 2. 1D response of the elastoplastic model with kinematic hardening for steel.

$F(\sigma)$ is the crack closure function, depending on the stress and on the material parameter σ_f (crack closure stress):

$$\sigma > 0 \rightarrow F(\sigma) = 1$$

$$-\sigma_f < \sigma < 0 \rightarrow F(\sigma) = 1 - \frac{\sigma}{\sigma_f}$$

$$\sigma < -\sigma_f \rightarrow F(\sigma) = 0$$

Damage criteria are expressed as $f_i = Y_i - Z_i$ ($i = 1$ for tension or 2 for compression, Y_i is the associated force to the damage variable D_i and Z_i a threshold dependent on the hardening variables). The evolution laws for the damage variables D_i are written as

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

where Y_{0i} is the initial elastic threshold and A_i, B_i and β_i are material constants.

A classical plasticity model with kinematic hardening is used to simulate steel (Figure 2) [22]. Reinforcement in the beams is introduced with special layers, the behaviour of which is a combination of concrete and steel by using an *ad hoc* role of mixture [1]. For dynamic calculations, damping is considered constant throughout the analysis and is a linear combination of the global stiffness matrix and the mass matrix (Rayleigh damping).

3.2. The shear multi-layered beam model

LMT has been interested in problems of shear in earthquake engineering since the early 1990s and has always worked on simplified approaches. By using the 'unilateral damage law' and the finite element code EFICOS, a modelling strategy has already been proposed [18, 19]. It consists in using multi-layered 2D Timoshenko beams with bending, shear and axial force interactions for the static and dynamic analysis of reinforced concrete structures (SMB model). The predictive capacity was proven by comparing the prediction with the results of the NUPEC seismic experimental program performed on a shaking table (Figure 3(a) and 3(b)) [19].

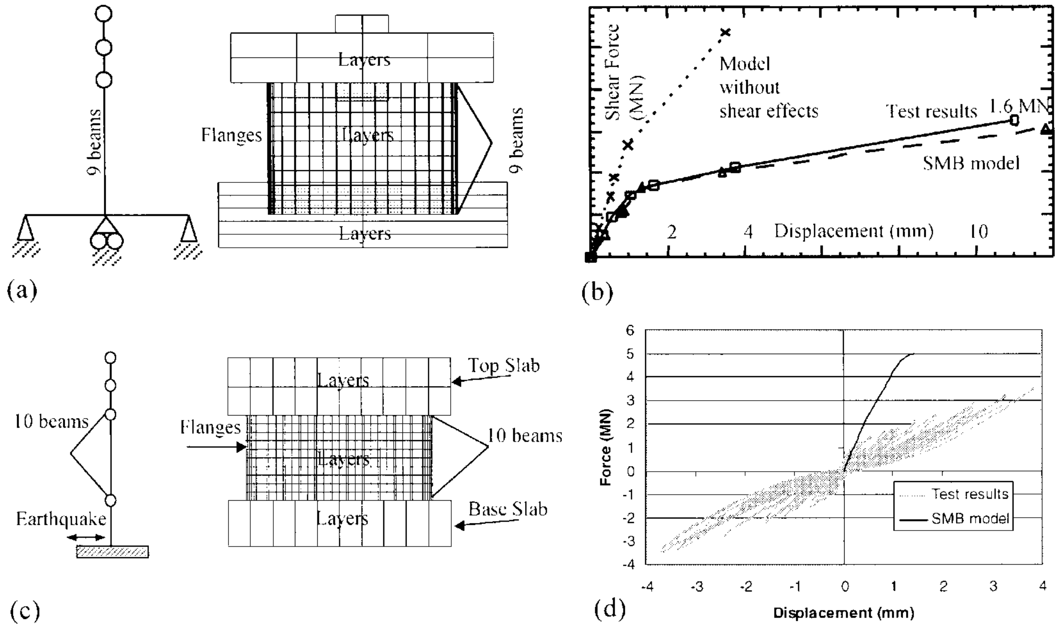


Figure 3. Mesh for the SMB model. Envelope of the top displacement versus shear force at the top of the specimens: (a, b) NUPEC specimen, (c, d) SAFE T5 specimen.

The SMB model was also used to simulate the behaviour of the SAFE specimen (EDF and COGEMA experimental program). This time however, the method failed to predict the non-linear behaviour of the structures (Figure 3(c) and 3(d)). The reason is the smaller slenderness of the tested specimens (0.4 for the SAFE mock-ups instead of 0.7 for the NUPEC specimen) and the different boundary conditions. During the NUPEC shaking table test, rotation at the top of the shear wall was free. For the SAFE experimental program, performed on a reaction wall, rotation is prevented and the specimens are dominated by shear without flexure interfering at all. A model based on a beam formulation such as the SMB model is unable to simulate this kind of behaviour. Although the linear behaviour is well simulated the model fails to reproduce the non-linear part. An original and simplified modelling strategy able to simulate the response of the NUPEC and the SAFE specimens is presented in the following.

4. EQUIVALENT REINFORCED CONCRETE MODEL (ERC)

4.1. Background

The ERC model uses a lattice mesh to predict the non-linear behaviour of shear walls and is based on the framework method [7]. The basic idea of the framework method consists in replacing the continuum material of the elastic body under investigation by a framework of bars, arranged according to a definite pattern, whose elements have suitable elastic properties. The criterion of suitability of the framework pattern under plane stress hypothesis, is an

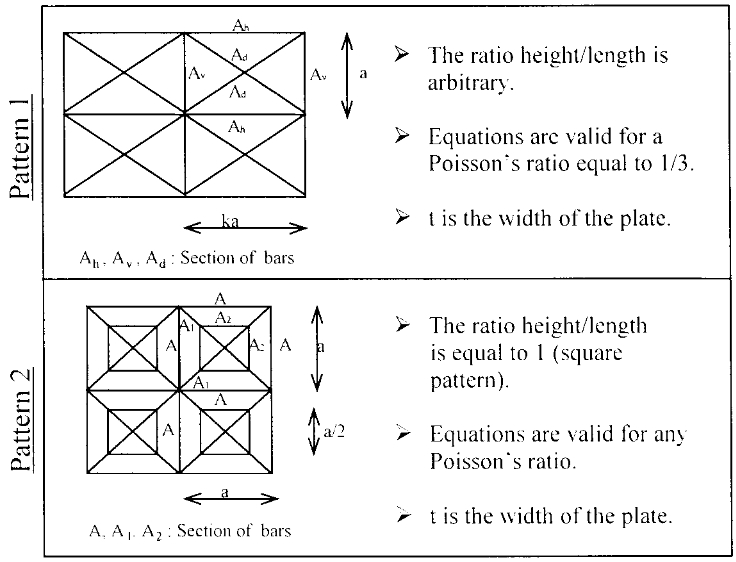


Figure 4. Two patterns of the framework method. (The first pattern is used in this paper.)

equality in deformability of the framework and the solid material in elasticity. If the size of the pattern unit of such a framework is made infinitesimal, then the latter will present a complete mechanical model of the solid prototype, with identical displacements, strains and unit stresses. Some of the patterns proposed by Hrennikoff for plane stress elastic problems of a homogenous material are shown in Figure 4. The elastic properties of the bars (sections) for the two patterns are given by

Pattern 1	Pattern 2
$A_v = \frac{3}{8}\frac{3k^2-1}{k}at$	$A = \frac{at}{1+v}$
$A_h = \frac{3}{8}(3-k^2)\,at$	$A_1 = \frac{at}{(1+v)\sqrt{2}}$
$A_d = \frac{3}{16}\frac{(1+k^2)^{3/2}}{k}at$	$A_2 = \frac{3v-1}{2(1+v)(1-2v)}at$

where v is the Poisson ratio, k the ratio between the length and the height a of the pattern.

4.2. Proposed lattice model

The idea is to use the patterns proposed by Hrennikoff in a non-linear context and for a non-heterogenous material. The new model is called the ERC model and its principles are summarized in Figure 5. The following assumptions are made:

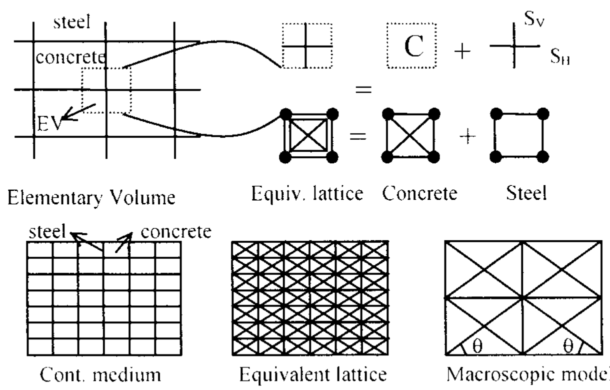


Figure 5. Reinforced concrete equivalent model (ERC)—principles of meshing.

- An elementary volume of reinforced concrete (EV) can be separated into a concrete element (C) and horizontal and vertical reinforcement bars (S_{H} and S_{V}, respectively). Concrete and steel are then modelled separately by using two different lattices.
- The sections of the bars simulating concrete have been derived directly from the framework method. The first pattern of the framework method is used because of its simplicity and the smaller number of required elements. This pattern is accurate for a Poisson's ratio equal to 1/3, obviously not the case for reinforced concrete in the elastic regime. This choice is however justified by the fact that the problems we are dealing with are highly non-linear (collapse of the specimen) and therefore the apparent Poisson's ratio is significantly changing (from 0.2 to 0.4 or even more).
- A crucial parameter for the success of the non-linear simulation is the angle θ that the diagonals of the concrete lattice form with the horizontal bars. This angle depends on the reinforcement ratios in the horizontal and vertical direction, the loading (normal compressive stress at the base of the specimens and shear stress) and the boundary conditions. It is related to the direction of the cracks in the structure (the bars are supposed to reproduce the Ritter–Mörsch scheme) and it lies between 30 and 60° to avoid negative values for the sections of 'concrete' bars calculated according to the first pattern of the framework method. For the non-linear calculations presented in this paper, the value of the angle has been calibrated to reproduce correctly the monotonic experimental curve (elastic domain and ultimate strength, see also Figure 14). A discussion on the choice of the angle is presented at the end of the paper.
- The 'unilateral damage model' in its 1D formulation is used to simulate the non-linear behaviour of concrete. Tests on reinforced concrete elements demonstrated that even after extensive cracking, tensile stresses still exist in the cracked concrete and that they significantly increase the ability of the cracked concrete to resist shear stresses [14]. Adjusting the post pick behaviour of the 'unilateral damage model' enables us to simulate this phenomenon known as 'tension-stiffening phenomenon' [23].
- A lattice composed of horizontal and vertical bars coupled with a uniaxial plasticity model simulates steel. The section and position of the bars coincide with the actual section and position of the reinforcement. To simplify the mesh, the method of distribution

is used where the sections of bars are defined proportional to a corresponding surface area. The mesh is thus independent of the geometry of the specimen.

- For at least the type of structures tested here, where the stress field is rather homogeneous, the number of elements that simulate concrete or steel do not have a great influence on the result. Two meshes with the same angle θ and different number of bars give virtually the same results [24]. A 'macro' model can be used instead of the 'equivalent lattice' (Figure 5). ERC can be used for structures where the stress field is homogeneous and the angle of the cracks does not change significantly during the loading. Otherwise, re-meshing strategies or other types of models (e.g. the 'strut-and-tie' model [11]) should be used.
- Perfect bond is assumed between concrete and steel.
- Symmetry of the pattern is required for cyclic and transient dynamic calculations.

5. MODELLING NUPEC SPECIMEN (DYNAMIC TEST)

The purpose of the test was to analyse the response characteristics of shear walls at levels ranging from the elastic state to the elasto-plastic ultimate state and to provide experimental data for computer code improvements by comparing the results of simulations with the experiments [4]. The main features of the specimen are shown in Figure 6 and Table I. The dynamic test was performed at NUPEC's large-scale shaking table at the Tadotsu Engineering Laboratory. The specimen was excited only in the x-direction.

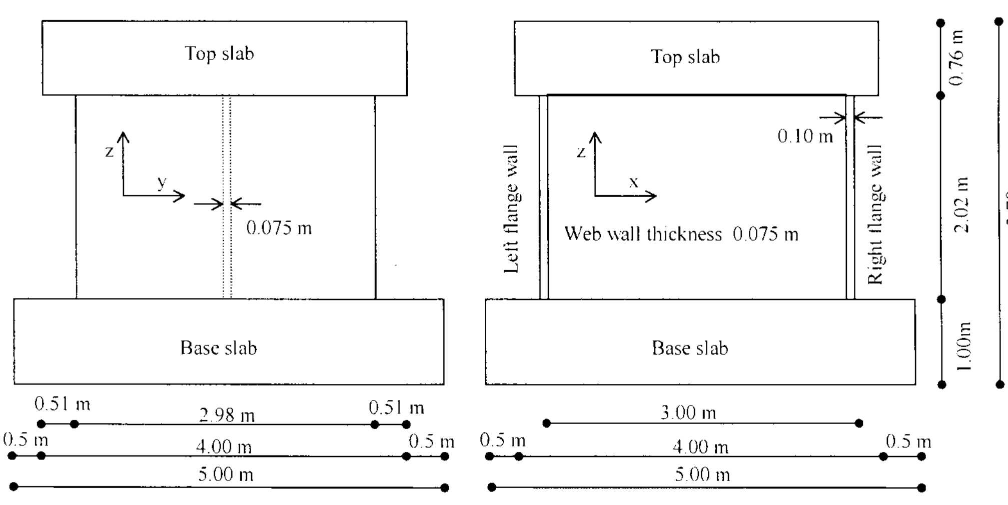


Figure 6. Geometry of the NUPEC specimen.

Table I. Main characteristics of the NUPEC specimen.

Type of test	Dynamic on a shaking table
Boundary conditions	Rotation free at the top
Height/length	≈ 0.7
Section of web wall (m^2)	0.225
Section of flanges (m^2)	0.596
Horizontal reinforcement (%)	**1.2**
Vertical reinforcement (%)	**1.2**
Compression strength of concrete (MPa)	28.6
Tensile strength of concrete (MPa)	2.3
Young's modulus of concrete (MPa)	22 960
Yield strength of steel (MPa)	384
Young's modulus of steel (MPa)	188 000
Normal stress at the base (MPa)	**1.5**
Mass (top slab + extra mass) (kg)	$(29.1 + 92.9) \times 10^3 = 122\,000$

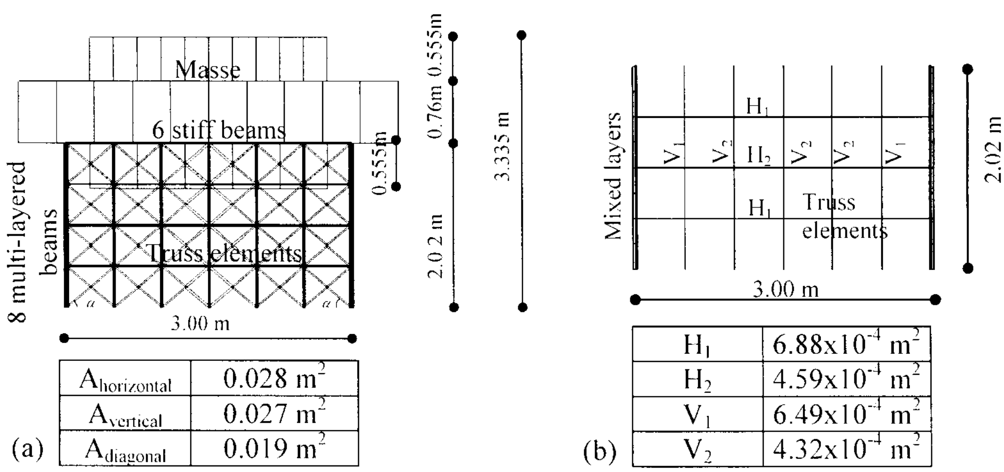


Figure 7. NUPEC test: (a) concrete mesh and section of truss elements; (b) steel mesh and section of truss elements.

5.1. Concrete and steel mesh

The angle θ has been calibrated with the experimental results and it is found approximately equal to 45° (Figure 7(a)). Each flange is described by eight multi-layered beams (Euler–Bernoulli hypothesis) to account for bending. The width of these beams equals the actual length of the flange (2.98 m). Six very stiff beams, free to rotate, simulate the top slab. Distributed masses are introduced at the top of the wall by three multi-layered beams. The base slab is not simulated. The section of the 'concrete' bars has been calculated according to the framework method. Horizontal and vertical bars simulate horizontal and vertical reinforcements (Figure 7(b)). Their sections and positions have been found by using the distribution method.

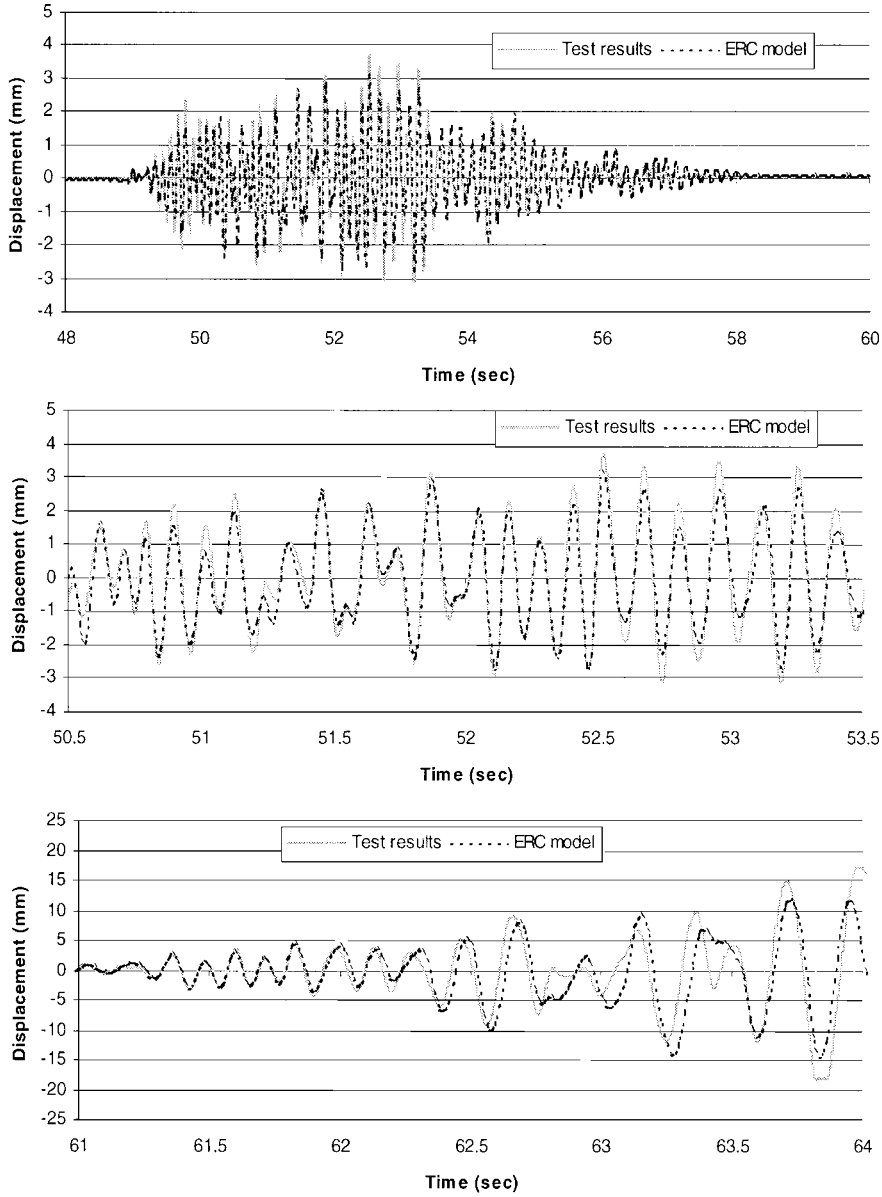


Figure 8. NUPEC: results of the dynamic transient analysis.

Reinforcement in flanges is introduced through special layers in the beams. Parameters used for the materials are the specific values already reported in Table I. 1D constitutive laws are used for concrete and steel.

5.2. *Modal and transient dynamic analysis*

The modal analysis predicted well the first frequency of the mock-up (13.1 Hz for the test and 12.9 Hz for the simulation). For the dynamic analysis the Rayleigh damping coefficients have been adjusted to ensure a 4 per cent damping on the two first modes. For the lower levels no significant differences between numerical and experimental results appear, since the structure stays nearly in the elastic domain. A zoom around the last two sequences (48–64 s) shows that the ERC model correctly predicts the global behaviour of the structure even under severe loading (just before collapse). No shifting between the numerical and experimental curves appears (Figure 8).

We have demonstrated the ability of the ERC model to successfully simulate the non-linear behaviour of shear walls having a small slenderness (equal to 0.7) when the rotation of the top is allowed, a case where our SMB model applies as well. The following section deal with the possibility of using the ERC model when the SMB model is no applicable (i.e. SAFE experimental program).

6. MODELLING SAFE SPECIMEN (PSEUDODYNAMIC TEST)

Within the SAFE research project, organized by COGEMA and EDF, 13 squat reinforced concrete walls have been tested at the Centre of Research of the European Commission at ISPRA in Italy [5, 6]. The main differences between the NUPEC and the SAFE programs are that the latter are pseudodynamic (PsD tests), the specimens are more squat (slenderness 0.46 instead of 0.7) and rotation of the upper part is not allowed. The following paragraphs describe the modelling of the specimens T5 and T12.

T5 and T12 mock-ups have the same geometric characteristics. The differences are the reinforcement ratio and the normal stress at the base of the specimens. T12 has a much more important normal stress and a smaller reinforcement ratio. Characteristics of the walls, the flanges, the top and lower slabs are shown in Figure 9 and Table II. The specimens were excited in the x-direction. The loading levels were determined to get responses ranging from elastic to elasto-plastic regimes.

6.1. *Concrete and steel mesh*

The angle θ for the concrete mesh has been calibrated with the experimental results ($\theta = 41.6°$ for T5 and $\theta = 30.1°$ for T12—Figure 10(a) and 10(b)). Each flange is described by multi-layered Bernoulli beam elements to account for bending. The width of these beam elements equals the actual length of the flange (0.80 m). Four stiff beam elements, the rotation of which is not allowed, simulate the top slab. Vertical displacement is free and the walls are fixed at the base. Horizontal and vertical reinforcement is simulated with horizontal and vertical truss elements (Figure 10(c) and 10(d)). Their section has been found by using the distribution method. Reinforcement in flanges is introduced through special mixed layers in the beam elements. Specific values used for the materials are the ones already reported in Table II.

Owing to its nature, a PsD test can be simulated by static calculations by using the displacements applied to the structure by the servo-controlled hydraulic actuators. To check our modelling strategy we will however also proceed with dynamic calculations for the T5 mock-

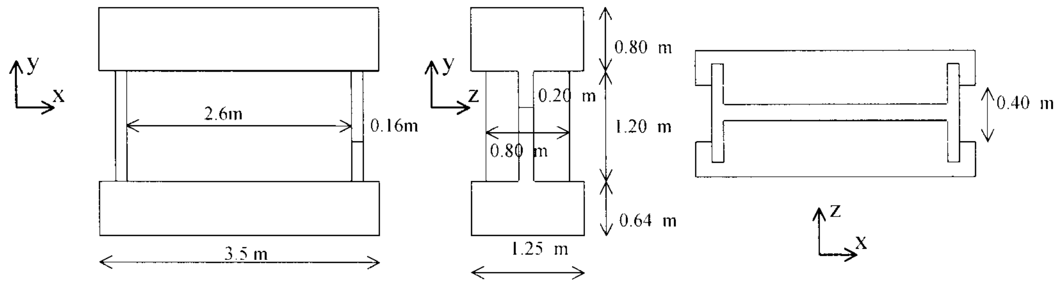


Figure 9. SAFE: geometry of the T5 and T12 specimen.

up, by using the artificially generated earthquake ground acceleration histogram given as an input to the simulation. Attention has to be paid to the fictitious mass used during the pseudodynamic test. The latter influences only the translation mode of the specimen and does not change its normal stress at the base. It is simulated as an extra mass linked to the shear wall via a rigid bar.

6.2. Cyclic analysis

The displacements applied to the mock-ups during the experiments are used as input data for the cyclic analysis. Displacement is applied at the top of the specimens and the total horizontal reaction is monitored. Global results correctly capture the history of the horizontal reaction for both specimens (Figure 11).

6.3. Spalling of the T5 specimen

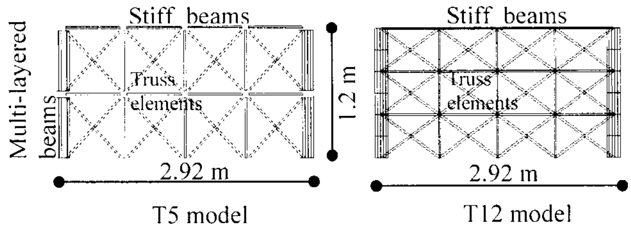
At the end of the T5 experiment an important spalling has been observed. The concrete of the cover was seriously damaged and the reinforcement bars were visible. A simplified, straightforward method to take into account this phenomenon is to change the width of the web wall in our mesh. By changing the width from 0.20 to 0.15 m (0.025 m corresponds approximately to the width of the cover) and by simulating the sections of the truss elements, numerical results are in better agreement in the non-linear regime (Figure 12).

6.4. Modal and transient dynamic analysis for the T5 mock-up

The modal analysis predicted well the fundamental frequency of the mock-up (6.7 Hz for the test and 6.8 Hz for the numerical model). Figure 13 shows the results of the dynamic calculations. The Rayleigh damping coefficients have been adjusted to ensure a 1 per cent damping on the two first modes. This small value is justified by the nature of the pseudodynamic test, which is carried out quasi-statically. The simulation is in reasonable agreement with the experiment. Some differences appear for the last loading step, however the maximum values of the displacements are correctly reproduced.

Table II. SAFE-main characteristics of the T5 and T12 specimens.

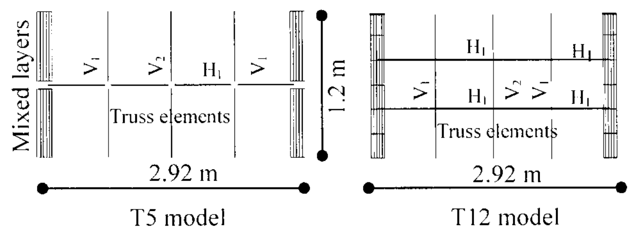
Specimen	T5	T12
Type of test	Pseudodynamic test	Pseudodynamic test
Boundary conditions	Rotation at the top not allowed	Rotation at the top not allowed
Height/length	0.46	0.46
Section of web wall (m^2)	0.52	0.52
Section of flanges (m^2)	0.128	0.128
Horizontal reinforcement (%)	**0.8**	**0.11**
Vertical reinforcement (%)	**0.8**	**0.11**
Compression strength of concrete (MPa)	34.7	34.7
Tensile strength of concrete (MPa)	3.0	3.0
Young's modulus of concrete (MPa)	30 000	30 000
Yield strength of steel (MPa)	500	400
Young's modulus of steel (MPa)	200 000	200 000
Normal stress at the base (MPa)	**0.34**	**1.0**
Mass (top slab + extra mass) (Kg)	$(29 + 0) \times 10^3 = 29\,000$	$(29 + 56.06) \times 10^3 = 85\,065$



$A_{horizontal}$	0.075 m^2
$A_{vertical}$	0.116 m^2
$A_{diagonal}$	0.069 m^2

$A_{horizontal}$	0.07 m^2
$A_{vertical}$	0.138 m^2
$A_{diagonal}$	0.069 m^2

(a) (b)



H_1	$17.59x10^{-4}$ m^2
V_1	$14.96x10^{-4}$ m^2
V_2	$9.97x10^{-4}$ m^2

H_1	$1.41x10^{-4}$ m^2
V_1	$2.12x10^{-4}$ m^2
V_2	$1.41x10^{-4}$ m^2

(c) (d)

Figure 10. SAFE: concrete mesh and section of truss elements for (a) T5 specimen and (b) T12 specimen: steel mesh and section of truss elements for (c) T5 specimen and (d) T12 specimen.

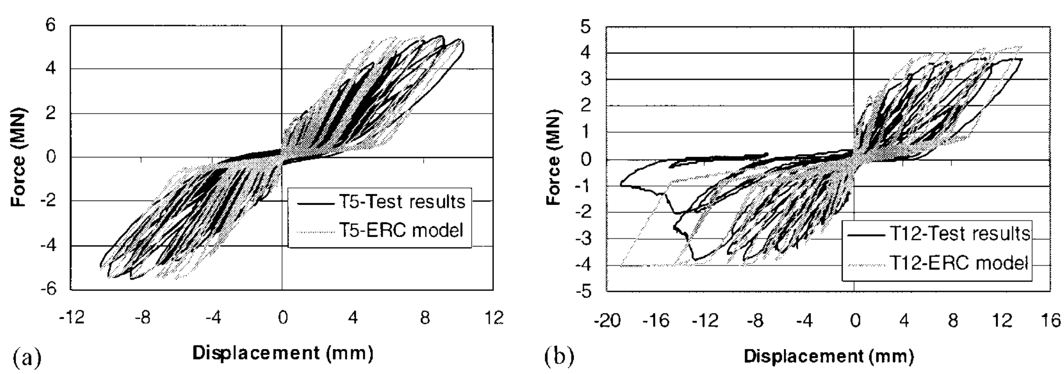


Figure 11. SAFE: results of the cyclic calculations for the: (a) T5 specimen and (b) T12 specimen.

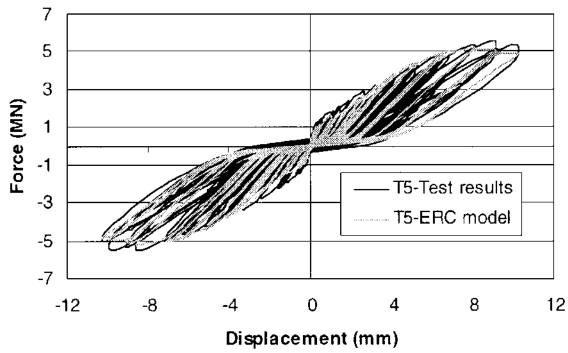


Figure 12. SAFE: results of the cyclic calculations for the T5 specimen taking into account the spalling.

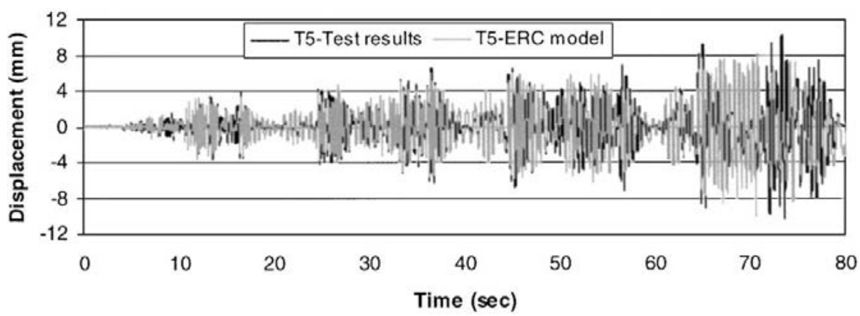


Figure 13. SAFE: results of the transient dynamic analysis for the T5 specimen.

7. DETERMINATION OF THE ANGLE θ

The angle θ between the diagonal and the horizontal bars of the mesh is crucial to the success of the simulations. In this paper, its value has been calibrated by using monotonic experiments. To study its influence, a sensitivity analysis is presented in Figure 14(a)–14(c). For normally reinforced concrete structures (e.g. NUPEC and the T5 specimen), the results of the simulation do not change significantly with the angle. A value between 35 and 45° correctly reproduces the global behaviour of the T5 and NUPEC mock-ups. A way to calculate approximately the angle θ would be to consider it equal to the direction ϕ of the principal stress at the end of the linear regime. However, for the specific case of lightly reinforced structures with an important normal stress, the value of the angle significantly influences the results. For the T12 specimen for example, it has to be limited between 30 and 33°. An accurate estimation of the angle is then necessary by using non-linear methods to calculate the variation of the angle ϕ during the load history.

This variation can be calculated by using simplified approaches. By assuming that the model reproduces the Ritter–Mörsch scheme, ϕ is derived from the equilibrium equations of

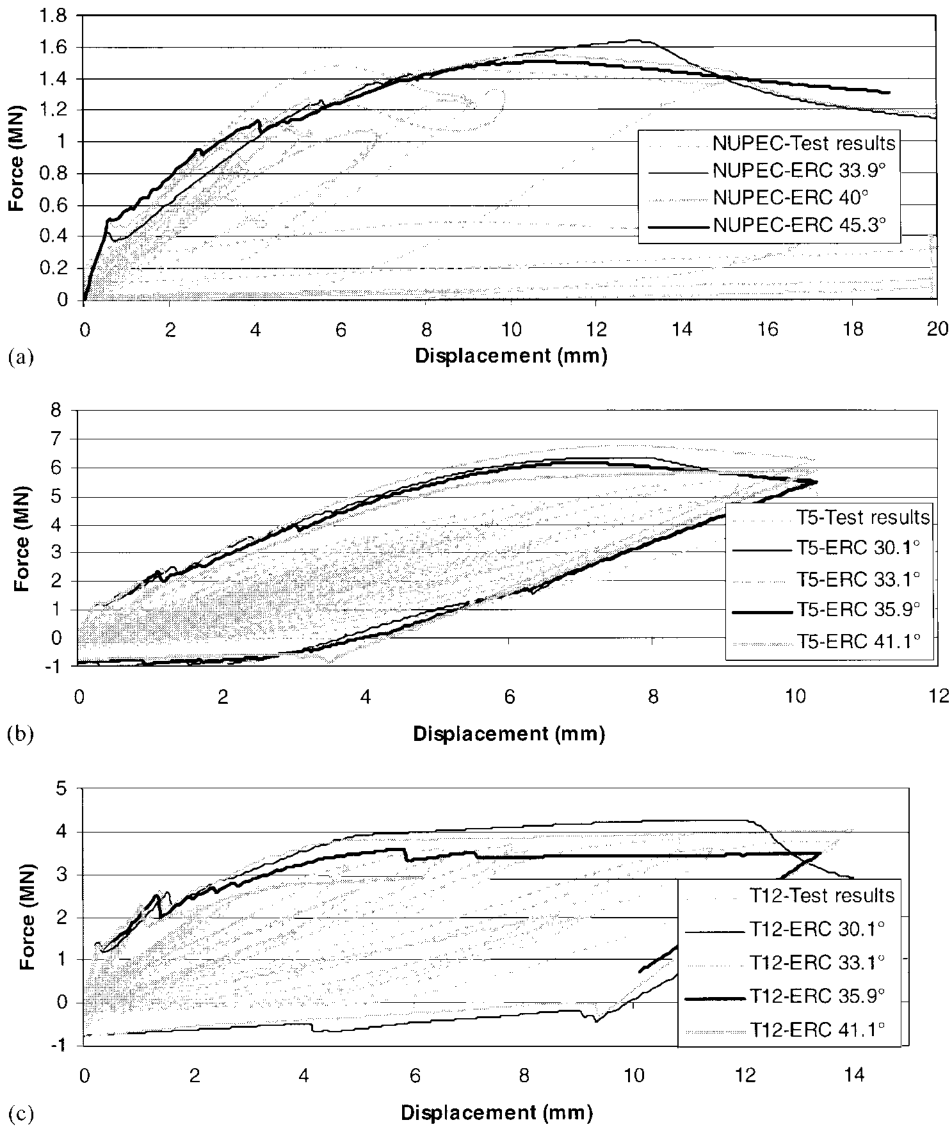


Figure 14. Sensibility study: (a) NUPEC specimen; (b) T5 specimen; (c) T12 specimen.

the corresponding truss and the stress–strain relationship of concrete and steel [12]

$$\tan^2 \phi = \frac{\varepsilon_l + \varepsilon_d}{\varepsilon_t + \varepsilon_d}$$

where ε_l is the strain of the horizontal reinforcement, ε_t the strain of the vertical reinforcement and ε_d the strain of the concrete strut. The stress–strain relationships for the materials are given from the compression field theory [12] or the rotating angle softened truss model theory [13].

However, these relationships are based on experimental results where the rotation of the tested specimens was free. It is interesting to test the validity of such constitutive laws for specific boundary conditions such as those of the T12 mock-up [24].

8. CONCLUSIONS

An original simplified modelling strategy to simulate the behaviour of shear walls under dynamic loading has been discussed. The ERC is based upon the framework method and uses a lattice mesh to simulate the walls. The use of simplified structural elements and 1D constitutive laws for the materials makes the method straightforward and cost effective, allowing for parametric studies. The ERC model has been validated with the NUPEC and SAFE experimental data. We were able to reproduce the global behaviour of three shear walls having various reinforcement ratios and geometric characteristics, even for the specific case where rotation of the upper part was prohibited. Static and dynamic calculations were presented and comparisons were made with the results of dynamic and pseudodynamic tests. The phenomenon of spalling occurred in one test and has been successfully simulated by a reduction of the initial width of the web wall.

The success of the simulation depends on the value of the angle θ that the diagonal compressive trusses form with the horizontal ones. This is particularly true for lightly reinforced structures. Some ways to estimate this value have been discussed, but need to be further studied. The proposed lattice model is very promising and could be used to simulate the non-linear behaviour of plastic zones developed in beam–column joints, base of bearing walls and ends of beams. It can be introduced in commercial codes used by civil engineers that are more familiar with elements such as trusses and beams. Last, 3D applications of the model considering out of plane flexure and torsion phenomena are also possible.

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