Presentation and interpretation of SAFE tests : reinforced concrete walls subjected to shearing.

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0. INTRODUCTION

The objective of the SAFE (Structures Armées Faiblement Elancées) program is the experimental characterisation of the dynamic behaviour of braced reinforced concrete walls with low slenderness ratio. This experimental program is composed of a serie of 13 pseudo-dynamic tests performed on loaded models in pure shear and is associated with complementary tests of behaviour characterisation of the constituent materials of the models. These tests have been carried out at the ELSA (European Laboratory for Structural Assessment) laboratory at ISPRA in Italy at the request of EDF (Electricité de France) and COGEMA (Compagnie Générale pour les Matières Nucléaires), with the support of the European Commission.
Each of the thirteen models was subjected to a succession of temporal loads representative of a smooth seismic spectrum of increasing intensity. The measurements carried out were those of force, displacement and local deformations at each time interval : thus the temporal variation of each measurement could be established. The loadings were successively increased until failure of the horizontal bracing function, characterised by a loss in the horizontal rigidity and large distortions.
Tests T1 to T4 constitute the development tests of the experimental protocol : these tests have allowed the representativeness of the models to be improved in terms of the construction conditions of the walls and above all the most appropriate choice of temporal integration algorithm to control the pseudo-dynamic tests. The principal tests T5 to T12 differ in the proportion of reinforcement in the walls, the initial elastic natural frequency of the pseudo-dynamic system and the value of the average vertical load applied on top of the wall during the test. Test T13 is a replica of test T8 : this complementary test allows the use of a reinforcement on the two faces of the wall obtained with the aid of vertical and horizontal bands of the composite material CFRP (Carbon Fibre Reinforced Polymer), developed by the company FREYSSINET.
At least 3 tests are carried out on each wall by varying the intensity of the reference accelerogram.
This document describes the tests performed in the context of the SAFE program :

- The geometry of the walls, characteristics of the materials used and the experimental apparatus used are described in parts 1,2 and 3 respectively.
- The synthesis of experimental results is given in part 4.
- Part 5 involves the interpretation of these tests concerning in particular the dissipated inelastic behaviour of these walls and the quantification of margins obtained with respect to the design rules.

A description of the experimental protocol and some experimental results yielded from these tests have been presented at the fifth national AFPS seminar : only the principal elements are therefore detailed here.

1. GEOMETRIC DESCRIPTION OF THE WALLS

The tested walls have a low slenderness ratio, with a height of 1.20 m and length of 3.00 m (see Figure 1). Their thickness is 0.16 m for tests T1 to T4 and 0.20 m for tests T5 to T13. There is a bend at each end of the wall order to act as a stiffener, with the aim of eliminating the risk of bending rupture.This provides a representation of bent walls existing in a building.
The wall is integrated in a structure whose dimensions are indicated in Figure 1.
Two strongly reinforced ground beams (upper and lower) ensure the transmission of shear forces by jacks and by fixation to the ground, without the possibility of unsticking at the foot or rotation at the head.
Notches at the base of the lower ground beam allow the insertion of iron wedges to prevent sliding of the specimen. There are also notches at the extremities of the upper ground beam, which allow the lodging of vertical jacks to control rotation.

2. CHARACTERISATION OF MATERIALS

2.1 Characteristics of concrete

Two concrete compositions have been used. For tests T1 to T3, the concrete composition used by the constructor of the walls to obtain the specified compressional strength appeared to be badly adapted in term of chosen granulometry : an alternative composition was used for the successive tests. The following composition was used for tests T4 to T13, based on the compressional strength (of a cube) and tensile strength (by splitting) :

- cement: 350 kg/m^3
- 0.5 aggregate: 760 kg/m^3 ; 4.12 aggregate: 440 kg/m^3 ; 10.20 aggregate: 660 kg/m^3
- water: 175 l/m^3
- no fluidifying agent

The mechanical characteristics of this concrete are summarised in Table 1. This concrete was used in tests T4 to T13.

Number of days	7 days	14 days	28 days
Rupture in compression (cube 150 mm in length)	38.1 MPa	43.1 MPa	43.4 Mpa
Rupture in tension (cylinder f = 10 cm , H = 20 cm)	3.07 MPa	3.18 MPa	3.32 Mpa

Table 1 : Mechanical characteristics of concrete used for tests T4 to T13

2.2 Characteristics of steel reinforcements

The reinforcements were manufactured from the steel "Fe B 44 k AREX", and then hot-rolled as high-adhesion bars. The results of the standard tests for the different diameters used are presented in Table 2.

Diameter	Plasticity Limit	Loading rupture	Deformation at rupture
6 mm	579.5 Mpa 567.3 Mpa 558.7 Mpa	653.7 Mpa 677.6 Mpa 659.7 Mpa	20.8 % 21.3 % 21.2 %
8 mm	594.3 Mpa 588.0 Mpa 601.0 Mpa	682.0 Mpa 665.1 Mpa 668.8 Mpa	53.4 % 23.2 % 25.0 %
10 mm	589.2 Mpa 571.2 Mpa 558.1 Mpa	655.4 Mpa 649.4 Mpa 648.2 Mpa	26.9 % 25.0 % 28.2 %
14 mm	528.3 Mpa 531.4 Mpa 527.6 Mpa	615.8 Mpa 622.1 Mpa 618.1 Mpa	25.9 % 24.2 % 26.3 %

Table 2 : Experimental data concerning the reinforcements

The vertical r_v and horizontal r_h percentages of steel differ for each of the walls. These percentages are indicated in Table 3.

Wall	T1	T2	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12	T13
r_v	0.8	0.8	0.8	0.8	0.8	0.4	0.4	0.4	0.4	0.6	0.4	0.11	0.4
r_h	0.8	0.8	0.8	0.8	0.8	0.6	0.6	0.4	0.4	0.6	0.4	0.11	0.4

Table 3 : Percentages of metallic reinforcement inside the wall

The reinforcements are introduced in the wall using two identical layers connected several closed stirrups. The T13 wall has had extra external reinforcement on the two faces with the aid of vertical and horizontal bands of the composite material Carbon Fibre Reinforced Polymer, CFRP.

3. TEST PROCEDURE

3.1 Experimental apparatus (Figure 2)

Two sets of five jacks were attached to the tested wall by means of a bolted metallic device on the upper ground beam. The forces were applied at the level of the centre of gravity of the wall to limit the induced rotations. It is therefore possible to use only two vertical jacks of low capacity in order to obtain a zero rotation of the upper ground beam while freeing the vertical movements.

Two of the horizontal jacks (one on each side) are controlled with respect to the relative displacement of the upper ground beam. The other jacks are controlled by force in relation to the first two jacks. The vertical jacks are controlled so that their displacements are identical, and the resultant of the forces that they induce is zero. The wall is placed perpendicularly to the large wall of reaction, on the "western" side. An additional retaining wall on the "eastern" side allows the manipulation of the loading device two sides at a time, by push-pull. The maximum lateral force available is therefore 7 MN.

Finally, a passive vertical loading device has been added to impose an additional force of 0.55 MN, corresponding to an average vertical loading of 1 MPa. This device was used on walls T6, T7 and T12.

3.2 Test sequences

The reference signal used to carry out the series of tests corresponds to a synthetic accelerogram of which the response spectrum at 5% damping is the envelope of the target spectrum. The spectral response peak of this signal is obtained for a frequency of 4 Hz.

The conventional stiffness of the walls, determined from BAEL 91 and denoted K_{dim}, is 5696 MN/m for walls of 0.16 m thickness, and 7120 MN/m for walls of 0.20 m thickness.
Before each test, the effective elastic stiffness K_{el} of the walls was determined and the values of the ratio K_{el} / K_{dim} are presented in Table 4.

	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12	T13
K_{el} / K_{dim}	0.48	0.70	0.70	0.75	0.80	0.64	0.53	0.68	0.68	0.72	0.71

Table 4 : Value of K_{el} / K_{dim} during the tests performed

In the pseudo-dynamic method, the inertial effects are taken into account by numerical integration of the acceleration terms. The mass matrix is imposed : in the case of the SAFE tests, the mass in horizontal translation at the top of the wall is therefore one of the test data. Where one of the SAFE test objectives is established by dynamic margins, it was of interest to specify the different initial frequencies f_0 for the walls of identical characteristics. Table 5 gives the values of these frequencies.

Wall	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12	T13
f_0 (Hz)	4	12	8	12	4	12	4	4	4	4	12

Table 5 : Initial frequency of the tests

Remark : The initial design frequency is calculated from the conventional rigidity :

$$f_0^{\,dim} = \frac{1}{2\pi}\sqrt{\frac{K_{dim}}{M}}$$

The effective frequency under very low displacement at the start of the first test is given by the following formula :

$$f_0^{\,el} = \frac{1}{2\pi}\sqrt{\frac{K_{el}}{M}}$$

For tests T1 and T2, the effective frequency ($f_0 = f_0^{el}$) was considered when implementing the pseudo-dynamic algorithm, which is suitable in terms of effective performances but does not have great significance in terms of the project. The design frequency was therefore favoured ($f_0 = f_0^{el}$) for the successive tests to evaluate the mass of translation in question required in the PDT algorithm by inverting the formula hereafter. The reference accelerogram was transformed into a nominal accelerogram by multiplying it by a factor k, which takes into account the mass value associated with the pseudo-dynamic algorithm, to find the value of the average design shear of the model. If L and e are respectively the length (3 m) and the thickness (0.20 m) of the wall, and τ_{dim} is the average design stress, k can be found. Γ_{dim} (f) is the target design response of 7 %.

$$\frac{M.k.\Gamma_{dim}(f_0^{\,dim})}{L \times e} = \tau_{dim}$$

For each of the tests, after those at very low amplitude to determine the true initial rigidity and the experimental positioning, the nominal accelerogram is firstly applied to the wall and then the homothetic accelerograms of increasing intensity, corresponding to the nominal accelerogram multiplied by a factor α. This coefficient was specified at the time of the testing. All of the walls were tested until failure as a function of the horizontal bracing.
The coefficient k is therefore integral to the seismic level of "design" of the wall element, taking into account the mass used in the pseudo-dynamic method to secure the initial frequency. The coefficient α directly quantifies the margins (if α>1) of the design rule used for the tested element.

The method used allows the independent steadying of the level of excitation and initial global frequency. However, during the test itself, the true frequency of the structure will vary directly with the behaviour of the model.
In Table 6, the values of the coefficient k can be found for each model as well as the different intensities α applied for each of the tests. For quick reference and a summary of diverse information already given elsewhere, the values of design frequency f_0^{dim}, conventional stiffness K_{dim}, calculated mass M and the design shear τ_{dim} can also be found in the table. Tests T1 and T2 are performed with a k coefficient equal to 1, which has been used to apply a direct force greater than the design value to the structure, taking into account the mass M associated with the pseudo-dynamic algorithm : these tests are therefore carried out until failure of one sole test.
The adjustment frequency of 4 hertz has been chosen as it corresponds to the spectral amplification peak and to a plausible frequency for the structures in question. The frequency of 12 hertz corresponds to an conceivable frequency limit, associated with a low mass in the pseudo-dynamic algorithm.

mur	f 0 dim (Hz)	K dim (MN/m)	M (10 3 Kg)	t dim (MPa)	k	α
T3	4	5696	9017	4	0.0427	1, 2, 3, 5
T4	12		1002	4	0.6581	1, 1.3, 1.5
T5	8	7120	2818	4	0.2163	1, 1.3, 1.5, 2
T6	12		1252	3	0.4936	1, 1.3, 1.5, 1.8
T7	4		11272	3	0.032	1, 2, 5, 10
T8	12		1252	2	0.329	1, 1.4, 1.8
T9	4		11272	2	0.0213	1, 3, 6, 10
T10	4		11272	3	0.032	1, 3, 6, 10
T11	4		11272	2	0.0213	1, 6, 10
T12	4		11272	1.44	0.0154	1, 3, 5, 10, 15
T13	12		1252	2	0.329	1, 1.8, 2.6

Table 6 : Values of mass and coefficients k and α

4. TEST RESULTS

The principal test results are graphically displayed, showing the horizontal force applied as a function of the corresponding displacement at the top of the wall. The hysteresis loops obtained lead to the definition of the cyclic behaviour of the wall, and its equivalent viscous damping during the tests. A drop in natural frequency equally translates as a state of degradation. As an example, the results relative to wall T5 are detailed hereafter. For the other walls, the same type of detailed results are given in the appendix of this document.

4.1 Principal results for wall T5

Figure 4 shows the accelerations and displacements measured at the top of the wall for the sequence of 4 tests defined previously in Table 6 (α=1,1.3,1.5,2).
Figure 5 represents the force-displacement curves at the top of wall T5 for the 4 tests.
The degradation of the wall can be illustrated by the decrease in its natural frequency during the tests. The frequency of the wall drops rather irregularly. It passes from 7 Hz to 2.7 Hz in T5-1, stays in the 2.7/2.3 Hz interval during T5-2, then in the 2.5/2.1 Hz interval during T5-3 and falls to 1.7 Hz during T5-4.
The damping is of the order 5/6 % during T5-1, which corroborates with fairly significant cracking. The same order of magnitude is maintained during T5-2 and T5-3. During T5-4, where the cycles are more dissipative, an average of 8% is observed.
The synthesis of other results concerning the other wall can be found in the fifth national AFPS seminar.

5. INTERPRETATION OF TESTS

5.1 Introduction

The SAFE tests confirm the dynamic behaviour characteristics already identified of reinforced concrete walls : the results obtained can be compared with design rules of applicable structures of the same material. In order to be relevant, this comparison must integrate the specific elements of the test protocol, which contribute in a determinate way to the results observed.
The characterisation of the seismic response of a reinforced concrete wall according to the common rules depends essentially on :

- the parameters permitting the evaluation of the dynamic response of the element (mass, rigidity, damping),

- the parameters allowing the evaluation of the resistance capacity or the deformation of the element (adjustable limiting force, associated displacement).

The global margin with respect to design is characterised during the tests by the multiplicative factor alpha applied on the initial design spectrum, the value 1 corresponding to the process of traditional design of steel reinforcements distributed up in the wall by the rule of tie-bars applied in homogeneous sections on the raw section of the web. The value of this dynamic margin – highly variable as a function of the tests – depends on the influence of the simultaneous phenomena :

- the conservatism of the initial design,
- the adjustment of the margin of static resistance (over-resistance),
- ductility effect (important admissible distortion),
- rigidity modification effect (sliding in frequency).

These phenomena must be analysed separately, even if in the statutory approaches and their effects are generally taken into account simultaneously. The first two phenomena can be linked on the whole to the expression of the resistance under static loading of the models, the first two of dynamic character in loading.

5.2 Examination of design parameters

5.2.1 Initial rigidity

The rigidity of the elements affects the calculation of statutory excitations, generally following an inclusive elastic calculation. This statutory calculation is performed by considering the initial rigidity of the element (dynamic modulus of uncracked concrete). The statutory estimation of the initial rigidity characteristics can be compared with experimental results. Usually, there appears to be a tendency to overestimate the initial rigidity of the reinforced concrete element, even in the range of very low deformations.

Test n°	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12
Conventional rigidity	5696	5696	7120	7120	7120	7120	7120	7120	7120	7120
Initial rigidity	2613	4270	4810	4490	3770	3960	5536	5050	2640	4640

The average initial rigidity is equal to 60 % of the conventional rigidity.

In a statutory French PS92-type design process, this phenomenon does not carry a large importance when the shapes of the design spectra used are considered. It can contribute to an underestimation of the displacements, which are often marginal in practice owing to the statutory design elsewhere of the joints between neighbouring structures. It should be noted, however, that recent developments in paraseismic engineering (displacement methods) lead to a consideration of lower deformation moduli. The FEMA 273 guide considers the initial elastic energy for uncracked braced walls, and a rigidity distributed by 2 for structures that are already cracked.

5.2.2 Damping

The damping measured during the tests is the damping under lower deformations. This damping evolves with the loading history as a function of the fissuration state of the reinforced concrete. The parameter adopted for the characterisation of the history of the element is the maximum distortion already supported by the element (an indicator of the fissuration of walls working in shear).

The initial internal damping takes a low value of less than 4 %. This value cannot be directly compared with ordinary statutory values which take into account the PS92 recommendations, or under Design Earthquake for installations relevant to a particular risk (equal to 5 or 7 % of the critical damping). The values used correspond to a history of more significant deformations. This initial value is fairly close to the value generally accepted in other rules for a smaller seismic excitation.

The evolution of the damping value from the effective functioning of the wall in reinforced concrete leads to values that are highly comparable with statutory values adopted for the Design Earthquake.

A tendency of a supplementary increase in the internal damping is observed, once the element has experienced significant distortions : the values obtained are close to 10 %, the value generally considered as being in the range of large displacements.

Large deformations in the range of the maximum attainable force usually show an important deformation of the hysteresis loops, particularly after setting of the maximum force. This seems to correspond with a phenomenon that can be likened to a degree of give (displacement under zero load during the inversion of the sign of the applied force). The examination of the vertical displacements associated with the horizontal displacements supports this hypothesis.

5.2.3 Force-deformation curve

The force-deformation curve of the tested elements is non-linear, even in the case of low deformations. The fixed displacement for the maximum force is always greater than 7 mm for the tests : the corresponding distortion to the maximum force is therefore greater than 0.5 % for all the tests. It should be noted that the distortion corresponding to the use of the maximum force is considerably constant, in conformance with the traditional hypotheses of paraseismic displacement approaches.
The fissuration of elements is kept limited when open until in proximity of the ultimate force. This fissuration is associated with a drop in significant rigidity with respect to the initial rigidity. The distortion of 0.5 % can on the whole be associated with the start of severe degradation of the concrete (spalling of the cover).
Damage of the elements can be illustrated by a joint examination of the vertical and horizontal displacements in the course of the tests. The evolution of the vertical displacement shows that since the extension phase of the fissuration, softening of the wall is associated with a cumulated low vertical displacement. From the moment maximum resistance is achieved, the apparent rigidity measured on the vertical displacement does not evolve further, although a rapid increase in irreversible vertical displacement can be observed. This phenomenon is correlated with the clear deterioration of the hysteresis curve registered on the horizontal displacement.

5.3 Estimations of static resistance capacity

5.3.1 Statutory approaches
The maximum and residual forces measured during the tests can be compared with different limiting values of this horizontal force, which can be calculated as a function of the norm used and structural scheme adopted. The definition of the residual force depends on the state of degradation accepted for the element, taking into account the loss of resistance caused by the significant damage to the concrete in this configuration.
The rules relative to reinforced concrete structures allow the evaluation of the static resistance of the walls considered. By the intermediate value of the coefficient of global behaviour, the specific rules for paraseismic protection allow the transitory loading character to be taken into account : these texts also generally include the clean formulations of the usable resistance capacities, which can be used in static comparisons.
To analyse the static resistances observed, the rules considered are respectively the French codes BAEL and PS92 rules, and Eurocode 8 (in its pre-norm version). The PS92 rules are considered in the example with a ductility coefficient of 1, which expresses the static resistance.
The application of the BAEL assumes a choice of the most appropriate structural scheme : it can be considered that the most representative scheme corresponds to that of the zone of the butt of the beam or to that with a short bracket (the BAEL does not explicitly consider the case of short walls). However, the automatic reinforcement calculations are generally carried out using the simplified rules (rules of generalised tie-bars) of finite element modelisations : the comparisons therefore integrate this type of method, although their conservative character in this example may be known.
For the tests which were aimed to show the rupture schemes of short walls in shear : the bending resistance capacities of the walls allowed effective use of this type of rupture scheme (significant reinforcement of the buttresses). The statutory verifications carried out show clearly this deliberate over-design of the bending resistance in all examples for these tests. The corresponding non-significant values of this rupture mode are not detailed here.
Evaluations of the attainable limiting forces from the tie-bar rules show clearly the pessimism of this approach in the example, in particular when the shear resistance of the concrete is neglected. However, in all of the tests performed, the distributed horizontal steel reinforcements which have only a marginal role in the re-ascent of the forces in the short bracket must be considered. This arises from the reinforcement of the upper beam and the loading device of the models (existence of a tie beam in the upper part).
Generally, the different possible estimations of the attainable ultimate force are all on the side of safety as regards the maximum force as well as the residual force (for realistic ultimate displacements). It should be noted that the T12 model does not deliberately abide by the minimum requirements of the reinforced concrete rules, particularly for those which concern the distributed reinforcements in the web of the wall. The ultimate force value considered is that which has been obtained for a distortion equal to 1 % : some tests have been performed until even larger distortions are obtained, in a highly degraded state of the elements (complete expulsion of the concrete in the thickness of the element). The used forces can therefore be lower.
For test T12, the difference results in deliberately reduced reinforcement for the distributed steel reinforcements in the web.

(0) Reference design
(1) BAEL article A.5.1.2 (steel only)
(2) BAEL article A.5.1.2 (with concrete shear Vc)
(3) PS 92 (q=1)

Wall n°	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12
V dim. (0)	1920	1920	2400	1800	1800	1200	1200	1800	1200	870
BAEL (1)	1700	1730	2220	1730	1730	1110	1110		1110	250
BAEL (2)	2310	2380	3050	2560	2560	1940	1940		1940	1080
PS92 (3)	2760	2800		2450	2450	2070	2070		2070	1690
V max given	3620	5300	5540	5180	7060	3920	4170	5640	4100	3800
V ultimate given	2300	2800	4600	4400	4600	3920	3000	4600	4000	3800

5.3.2 'Best estimate' approach

In terms of the resistance of materials, the rupture values stated (maximum force) seem correlated either to a rupture mechanism in shear compression of the concrete (which is translated by spalling of the concrete), or a desolidarisation mechanism of the compressional tie-rod support of the bracket short (due to a lack of steel reinforcements in the vertical steel tie-bars).

- **Concrete rupture**

The resistance of the concrete tie-rod governs the maximum usable shear under the following usual form :

$$Tu = 0{,}5\ \nu\ (\ 0{,}85\ fc\ /\ \gamma b)$$

ν is an efficiency coefficient, of which the value can vary between 0.5 and 1. The usual statutory value is about 0.5. This leads to values of potential shear of the order of 6.4 MPa (with f_c equal to 43.5 MPa). The maximum force values lead to a shear between 6.4 MPa (minimum value) and 11.8 MPa, for all tests.

Shear (in MPa)	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12
Design	4	4	4	3	3	2	2	3	2	1.44
Maximum	7.5	11	9.2	8.6	11.8	6.5	7	9.4	6.8	6.4

The values obtained are therefore very compatible with the stated phenomena of concrete spalling (ejection of cover, then destruction of the body of the wall). The current bracket of about 7 MPa for the maximum shear observed is smaller than the values stated.

- **Rupture of the steel reinforcements**

The estimation of the maximum resistance of the vertical steel reinforcements with respect to the tie-bar of the compressed tie-rod support is dependent on the idea that one can integrate this in the participation of the vertical steel reinforcements of the compressed buttress : taking into account the entirety of the available steel sections leads to values at least equal to those corresponding to concrete rupture, considering the over-design of the buttress steel reinforcements. The statutory limitation of the participation of the buttress steel reinforcements as a function of their proximity to the butt of the web can however lead to lower limiting values than those corresponding to concrete rupture for the least reinforced walls.

The two mechanisms must therefore compete to halt the rupture and in practice be linked. This is perfectly compatible with experimental findings. Therefore, the static behaviour of the walls conforms on the whole to that one can expect in terms of material resistance and relieves the margins with respect to the statutory resistance.

5.4 Estimation of dynamic resistance

5.4.1 Force approach

The estimation of the dynamic resistance must consider the transitory character of the loading.

The values of the dynamic margin coefficients (alpha coefficient) are considerably different as a function of the initial setting of the characteristic frequency of the different walls (by the mass which has defined them since the test). These dynamic margin coefficients have been estimated with respect to the design process adopted (rules of tie-bars of distributed steel reinforcements) and not with respect to the best possible estimation of the static resistance capacity of the elements : they therefore integrate diverse contributions.

N° test	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12
Alpha	1	1	1	1	1	1	1	1	1	1
	2	1,3	1,3	1,3	3	1,4	3	3	6	3
	3	1,5	1,5	1,5	5	1,8	6	6	10	5
	5		2,0	1,8	10		10	10		10
										15

The applications of the PS92 rules or Eurocode 8 (pre-norm) type assume the definition of an overall coefficient of behaviour, which depends on the global structure. The behaviour coefficients chosen correspond to standard values for structures braced by walls (q=2 or q=3). The elastic design (q=1) leads to an assimilation of the dynamic and static resistances, which is clearly unfavourable.
The following values originate from reference (6).

N° test	T3	T4	T6	T7	T12	T13
PS 92	2761	2804	2447	2447	1695	2072
EC8	990	901	1060	1060	552	878
DAN EC8	1112	1014	1060	1060	584	983

The estimations carried out in application of Eurocode 8 are clearly very penalising : this result only confirms the analyses already concerning structures braced by walls.

5.4.2 Displacement approach
A similar approach to the one adopted in paraseismic displacement methods allows different action phenomena to be followed. In this type of approach, the seismic event is characterised by its displacement spectrum for different dampings (imposed displaced curves, imposed accelerations).
This curve has been traced for the target spectrum, in conformity with the data of the pseudo-dynamic analysis used in the tests. On this curve, the response of a simple oscillator remains defined by its natural frequency, which is associated with a straight slope from the origin for each point of the spectrum. The different settings lead to homothetic curves (factor k and α).
To compare this curve with the force-displacement curve of the tested elements, the spectral acceleration has been multiplied by the movement mass for each type of test and has taken into account the fixed factors of each test (factors k and α). It is therefore possible to create reference curves of successive tests for each model. The spectral displacement has been assimilated to the displacement of the upper cross member in the pseudo-dynamic loading. The comparison of experimental force-deformation curves with the applied displacement spectra obtained during the tests therefore shows the functioning of the dynamic system. The point of intersection of the spectral curves and those of force-displacement is the point of convergence of the system. This representation allows direct visualisation of the importance of the effects of rigidity changes. Two characteristic examples are illustrated hereafter.

- **Test T8**

Test T8 is a fixed test initially in the range of the natural frequency of the first mode shape of the wall and is considered separately. It is therefore associated with a complementary low mass and a rather elevated frequency (12 Hz). The global coefficient, alpha, leading to the use of the maximum force available is close to 1.80 in the displacement approach if the real force-displacement curve stated is taken into account. This conforms perfectly to the experimental results.

- during test T8.1, it is noted that the maximum usable force from the test is greater than the design force. The sliding frequency results from the fact that the first fissuration of the wall leads to a first phase which augments the real excitation with respect to the excitation from the theoretical elastic calculation : the maximum value attained perfectly correlates with the drop in rigidity.
- From the test T8.2, the wall makes use of the resistance capacity necessary to balance the imposed forces.
- From test T8.3, the wall makes use of the displacement necessary in the range of the maximum available force.

Taking into account the setting of the initial high frequency, the available resistance and ductility capacities have effectively served to compensate for the effects of the drop in rigidity. However, in a simplified approach of type PS92, the reference acceleration for this structure could have been fixed as a function of maximum spectral amplification.

- **Test T9**

Test T9 is a test set to the lowest frequency (4 Hz), at the peak of the design spectrum : the wall is therefore associated with a more significant mass. The maximum alpha coefficient considered in the tests (α=10) leads to a distortion generally greater than 0.5% in this type of displacement approach, which has been certified experimentally :

- during the T9.1 test, the wall does not use the resistance capacity in reinforced concrete (very low deformations) : there is a good correlation between the ideal elastic model and the test results, if the initial discrepancy in frequency is integrated ;

- during test T9.2, the force applied remains less than the theoretical force, arising from the modification of the rigidity. The theoretical elastic force calculated from the initial rigidity is close to the maximum attainable force for the wall ;
- during test T9.3, the theoretical elastic force is largely greater than the actual attainable static resistance force, which is close to the maximum force of attainable static resistance. This leads to the acceptance of a behavioural coefficient for the tested wall of the order of 1.5 with respect to the maximum attainable force (by neutralising the effect of over-resistance and considering the actual initial rigidity) ;
- during test T9.4, the element uses its ultimate displacement at a constant force. Its ductility is important, to the extent of a degradation of the concrete of the wall.

The coefficient 6 on the initial design obtained for test T9.3 can on the whole be decomposed as follows :
- conservatism of the rule of tie-bars 1.8
- - fixed over-resistance 2.2
- - dynamic effect 1.5

The values given above can only be considered as approximations.
In the different examples, the drop in experimental frequency can be correlated with the force-displacement curve. The values of maximum force attainable during each elementary test are well correlated with the adopted approach.
It should be noted that in this displacement approach, the alpha factor is in good relation with the maximum attainable displacement, taking into account the actual attainable resistance capacities which are nearly identical for models T8 and T9.

5.5 Estimation of displacements

The maximum displacement calculated from the elastic modelling of the seismic behaviour of structures is assumed to be equal to the real seismic displacement (PS92 rules).
This displacement can be calculated as a function of the initial frequency, the design spectrum used and the load factors of the test considered (alpha and k). An important discrepancy in the maximum displacements stated (line 4) is noted with respect to the elastic design hypotheses (line 1), as well as the correction of the effects linked to the difference between the initial rigidity given and the theoretical rigidity taken into account during design (line 2). This discrepancy increases with the frequency of the initial setting of the test : the linear elastic model is therefore inappropriate in this example. The correction of the elastic displacement to take into account the maximum spectral amplification of the target spectrum in the high frequency region is not enough to compensate for such a difference (line 3).
To obtain a better approximation of the displacement, it is necessary to consider the effects of the drop in rigidity during the tests.
The result obtained is not necessarily in contradiction of the usual rule of equality of the displacements, which was initially justified for non-tested structures of low frequency at the time of the SAFE tests (Newmark's rule). However, the validity of an overestimation of the rigidity of the initial structure can be legitimately questioned.

- **Test T8 : theoretical frequency of 12 Hz**

		T8.1	T8.2	T8.3
	Load coefficient alpha	1	1.4	1.8
(1)	Theoretical displacement (mm)	0.17	0.25	0.30
(2)	Displacement of real rigidity	0.38	0.53	0.68
(3)	Conventional displacement	1.2	1.7	2.2
(4)	Maximum displacement observed	2.5	5.5	12

- **Test T9 : theoretical frequency of 4 Hz**

		T9.1	T9.2	T9.3	T9.4
	Load coefficient alpha	1	3	6	10
(1)	Theoretical displacement (mm)	0.17	0.51	1.0	1.7
(2)	Displacement of real rigidity	0.21	0.64	1.3	2.1
(3)	Conventional displacement	0.17	0.51	1.0	1.7
(4)	Maximum displacement observed	0.58	2.4	6.0	16

5.6 Conclusions

The permissible distortion of a braced wall working principally in shear can be taken as equal to 0.5 %. This value is generally accepted in literature and it has been well-verified by the tests. The state of the wall for this distortion clearly shows that it is compatible with its mechanical function (limited fissuration, ensured vertical resistance, anchorage of maintained equipment and control of the risk of induced projectiles).

The rules of evaluation of the maximum resistance capacity are reasonable, as they effectively express the overall capacity of the reinforced concrete element. Approaches of the 'tie-bar rule' type to justify the distributed steel reinforcements in the bracing walls, such as those used in finite element methods, are clearly much more pessimistic for the cases considered during the tests. The evaluation of the statutory ultimate moment following an approach of reinforced concrete type is acceptable with respect to attainable residual forces in case of accidental intrusion of an element in the range of very large displacements : this guarantees a certain robustness of the system. The maximum attainable force remains largely greater than the limiting statutory ultimate force. In this way, the safety coefficient shows that varying uncertainties, which can affect the current construction of a civil engineering structure, can be freed.
A degree of caution is exercised during the rigidity evaluation of the bracing elements when examining the displacements, since a rather rigid structure functions effectively in reinforced concrete. Only the displacements of structures which have been subjected to very little excitation (and therefore very much over-designed) can be evaluated from the theoretical rigidity under low deformations. However, this is often the case with nuclear structures.
The employment of margins linked to a dynamic loading character is usually translated by an overall behavioural coefficient. The estimation of the global behaviour coefficient depends on all the characteristics of the structure : hence its value cannot be given here. On the other hand, the tests yield possible values of local ductility coefficients, which are highly important if the relation between the theoretical elastic deformation and the reasonably attainable limiting distortion (coefficient greater than 10) is considered. The variation of the relative rigidities in a real structure can lead to uncertainties in the repartition of the forces in the different bracing elements : this phenomenon is accounted for by the limitation of the global behaviour coefficients when examining the overall structure in the simplified approaches.
A realistic approach to the determination of the behaviour of concrete elements therefore implies that the non-linear character of the force-deformation curve needs to be considered. The behaviour under horizontal load for walls working principally in shear can be characterised by :

- an initial rigidity in small deformations,
- an attainable ultimate shear,
- an associated distortion of the order of 0.5 %.

The tests show the relevance of displacement approaches to the behavioural analysis of structures under seismic loads.

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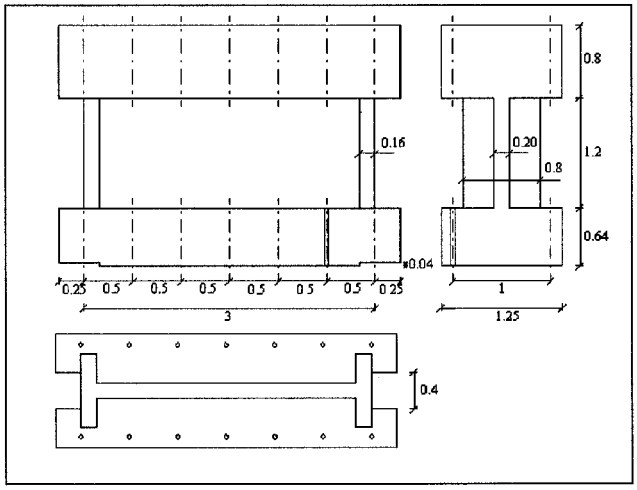

Figure 1 : Geometric description of wall T12 and the limit conditions

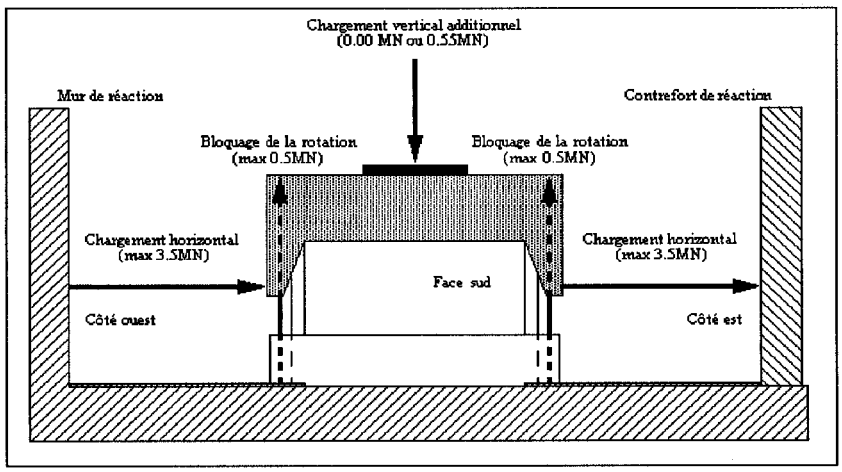

Figure 2 : Principal of the experimental apparatus

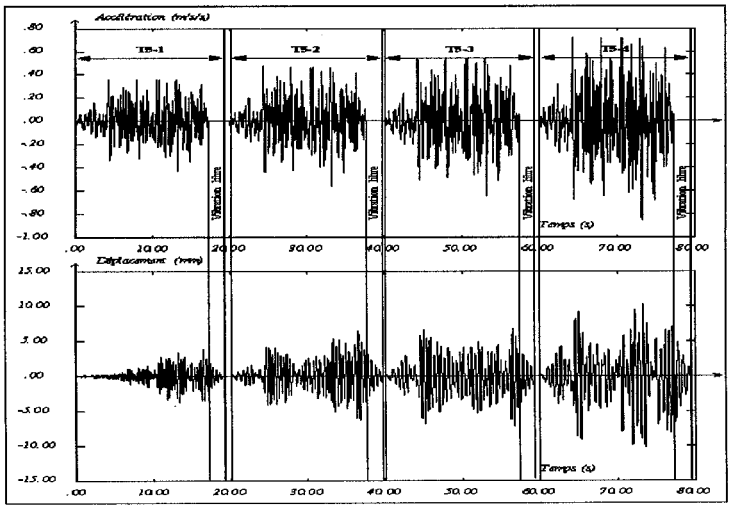

Figure 3 : Accelerations and displacements at the top of wall T5

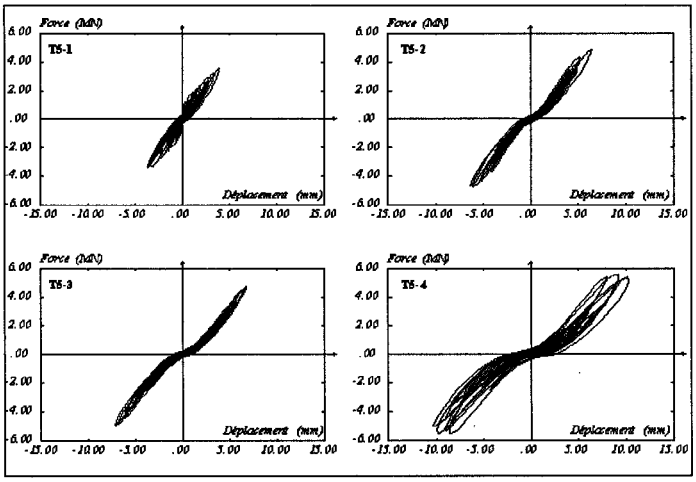

Figure 4 : Force-displacement curves at the top of wall T5

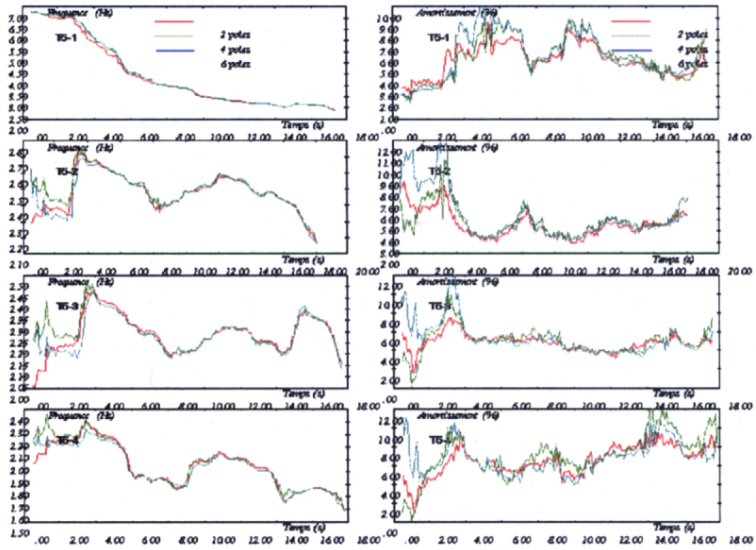

Figure 5: Identification of the natural frequency and damping of wall T5 in the course of the tests

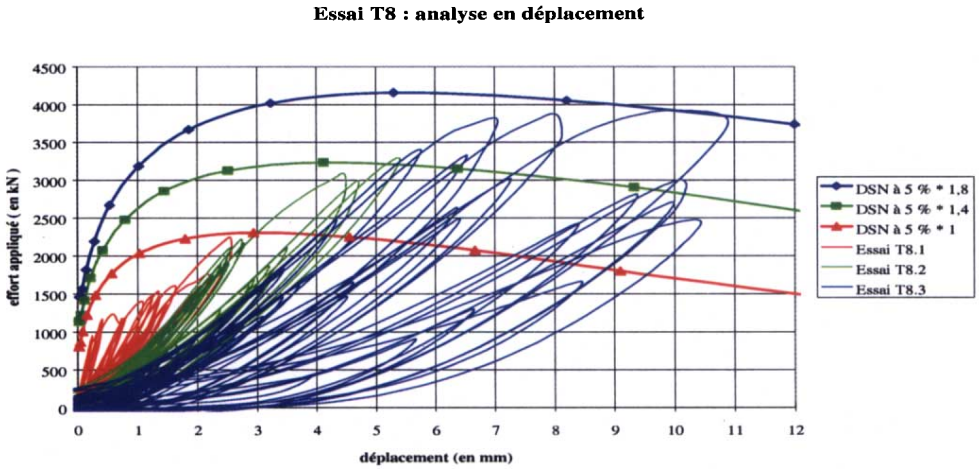

Figure 6: Displacement analysis (Test T8)