CRACKING IN SHEAR WALLS:

EXPERIMENTAL RESULTS AND ANALYTICAL PREDICTIONS

Philippe BISCH\textsuperscript{1}, Silvano ERLICHER\textsuperscript{2} and Miquel HUGUET\textsuperscript{3}

ABSTRACT

This paper is a contribution to the analysis of cracking (crack width, spacing and orientation) in reinforced concrete shear walls under seismic loading. First, the experimental campaign of the French national project CEOS.fr and several test results obtained by different measurement techniques applied on the test specimens are presented. These results are compared to those obtained by the application of design codes formulas. For assessing the crack width, it is necessary to perform a structural analysis; the reinforced concrete shear wall specimen is modelled by the Finite Element (FE) software \textit{Code\_Aster} using three different approaches: (i) linear elastic behaviour, (ii) \textit{GLRC\_DM} global homogenized nonlinear constitutive law for reinforced concrete and (iii) \textit{ENDO\_ISOT\_BETON} nonlinear concrete law for a semi-global approach (multilayer element). The results of the FE analysis are then used to compute the crack width. The structural modelling is highlighted as a major parameter for a good assessment. Procedures to calculate crack width based on the \textit{fib} Model Code 2010 approach are proposed and a good agreement to the experimental results is obtained. Finally, the effects of the cyclic loading at Ultimate Limit State, as the values of residual crack width after an earthquake, are presented and discussed.

1. INTRODUCTION

The evaluation and control of cracking in Reinforced Concrete (RC) elements of industrial buildings is an important design aspect. Most of construction design codes, like e.g. the Model Code 2010 (\textit{fib}, 2010) and Eurocode 2 (CEN, 2005), require cracking control against the Serviceability Limit State (SLS). However, in the particular case of nuclear power plants, the limitation of cracking should be assured throughout extreme events as earthquakes in order to preserve the tightness, durability and confinement issues against the Ultimate Limit State (ULS) under cyclic loadings. Moreover, the formulas provided by these documents are semi-empirical and mainly developed for beams and relatively thin elements. Thus, their reliability is questionable for thick RC walls subjected to high shear stresses, which are the structural elements that govern the response of nuclear power plants buildings to a seismic excitation. Prediction and control of skew cracking of thick walls is a key issue that should be improved. Important contributions in this direction can be found for instance in the works of Vecchio and Collins (1986), Kaufmann and Marti (1998) and Pimentel et al. (2010).

In this paper, the experimental campaign of the French national research project CEOS.fr (IREX, 2008) is briefly presented. The tests, the different measuring techniques employed and the most representative results are presented in § 2. Then, some important results, mainly related to crack

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spacings, are presented and discussed in § 3. The assessment of crack width is presented in § 5; it is obtained from the strains and stresses distributions computed via the structural analysis presented in § 4. Finally, results related to the cyclic loading cracking at ULS are given in § 6.

2. THE CEOS.fr FRENCH NATIONAL PROJECT

The experimental results considered in this work have been obtained in the framework of the French national research project CEOS.fr. Shear-wall specimens were designed and tested in order to analyse the shear cracking in RC walls under cyclic loading.

The test campaign concerned four specimens representing a reduced-scale model of RC shear-walls commonly employed in nuclear facilities to resist horizontal seismic loadings. The applied loading history on three of these tests was cyclic and reversing, while a non-reversing loading history (without reversal of the applied force sign) was applied to wall number 3. No vertical load was applied except the self-weight, which is small compared to the horizontal loadings.

The specimens measure 4200 mm × 1050 mm × 150 mm (Fig. 1) and, as a result of their geometrical scale of 1/3, they are equivalent to a “real” 12.6 m × 3.15 m × 0.45 m wall. Indeed, these dimensions are representative of a structural wall employed in nuclear facilities and they ensure a slenderness ratio (1/4) that guarantees the prevailing of the diagonal shear cracking development over the bending one. This is also attained in the specimen by several vertical rebars with 25 mm and 32 mm diameters and placed at the extremities of the wall, which connect two horizontal concrete beams with a high reinforcement ratio. These beams, connected to the shear wall in its upper and bottom part, allow a better distribution of the shear force in the wall.

The wall number 3 is made of a C40 concrete mix and is reinforced with two identical steel layers on each face of the wall, each one with horizontal and vertical rebars of 10 mm diameter and spaced by 100 mm. The cover is 10 mm for the horizontal rebars and 20 mm for the vertical ones. The load is applied by a hydraulic actuator placed 100 mm over the top of the wall at one side of the upper thick beam by means of 300 kN force increments (Fig. 2). The specimen is installed in a steel frame to avoid large reactions on the laboratory slab (Fig. 4). This also allows a better control of the force
application and of the boundary conditions. Fig. 3 shows the force exchange between the wall and the steel frame: they constitute together a self-balanced system.

The other three walls (number 1, 2 and 4) are submitted to reversing cyclic loadings, with an actuator placed on each side of the top beam; so the loading history is similar to that presented in Fig. 2 but with negative and positive values. The design of wall 2 is the same as that of wall 3, but with a different loading history. Wall 1 is basically the same as 2 and 3, but it is cast with a different concrete class C25, in order to assess the effect of concrete strength on cracking. Wall 4 is also similar, but with a different reinforcement: all rebars have a 8 mm diameter and they are spaced by 80 mm, with a 0.8% total steel ratio in each direction, compared to 1% for the other walls (Table 1).

Some different approaches have been used to measure the wall displacements, strains and crack pattern. Three types of sensors have been used: (i) several Linear Voltage Differential Transformers (LVDT) measuring the relative displacement between the upper and the bottom parts of the wall and the cracks width evolution during the experiment; (ii) Bragg optic fibre sensors to measure the steel bars strains and (iii) resistive strain gauges, for concrete surface strains. The external temperature was also measured at several points. Cracks were carefully identified at each stage of the loading and LVDTs were installed when they appeared to measure their widths.

Digital Image Correlation (DIC) gives a more exhaustive characterisation of the global crack pattern. This technique uses images from the rear face of the wall, captured by three digital cameras that cover the whole surface of the wall for all the test duration. The displacement field \( u \) between two different loading times (the first one taken as reference) is then obtained by correlating the grayscale levels of the two corresponding images within each pixel. The differentiation of the displacement field \( u \) gives the strain field that can be post-processed to measure crack widths, spacings and orientations (Ruocci et al., 2012).

![Figure 5](image1.jpg)  
**Figure 5.** (a) Steel and concrete horizontal and vertical strain evolution at sensors V2, H2 and 12; (b) crack widths measured by LVDT 5 and 7

![Figure 6](image2.jpg)  
**Figure 6.** Comparison between crack pattern identification by DIC analysis (left) and visual detection (right) at the central portion of the specimen for the last loading stage (4,2 MN)

To show the extent of the results obtained, the case of the shear wall number 3 is examined in more detail. Some sensors installed on the specimen recorded the evolution of the concrete strains on the wall surface and steel bars strains during the experiment. An example of these measurements is presented in Fig. 5 (a), which shows the measured horizontal and vertical concrete and steel strains at the centre of the wall. The arrows show the instant when the crack formation causes a sudden change in steel strain and the vertical line marks the instant of the crack visual detection; a good correlation is observed. As expected, before the crack formation, concrete and steel vertical strains are both positive.
(tension), while the horizontal ones are negative (compression). Concrete and steel have the same strains at low loadings, but slip appears when the crack forms. At this instant, the steel vertical strain jumps to a higher positive value while the horizontal one becomes positive. Finally, concrete turns from tension to compression in the vertical direction, while the concrete horizontal strain is not affected by the crack apparition.

LVDT extensometers have recorded the crack widths. Fig. 5(b) shows the measurements of sensors D5 and D7, placed across two different cracks (after their visual detection) at the centre of the specimen. They are fixed orthogonally to their respective cracks to record the relative normal displacement.

The DIC analysis has recorded the crack patterns at all loading stages. The crack pattern detected at the last load cycle for the central part of the specimen is presented in Fig. 6 and compared with the visual crack identification, showing a good agreement.

3. MAIN OBSERVATIONS FROM THE EXPERIMENTAL RESULTS

The large number of measurements obtained by image correlation allows a true statistical cracking assessment, especially for the mean and characteristic values of both crack spacing and width. In order to make sense in the case of walls, the stress field is needed to be roughly homogeneous on the entire area of analysis. Some preliminary analyses showed that the stress distribution is highly dependent on the wall structural behaviour (particularly on the boundary conditions), and that accordingly the stress field is not homogeneous in the wall. Therefore, the exploitation of the DIC has been limited to the central area of the wall, which is mainly subjected to a homogeneous state of shear and horizontal compression. The statistical analysis in this area provides a relevant comparison with the formulas given by standards, in this case the fib Model Code 2010 (MC10) and Eurocode 2 Part 1 (EC2).

Table 1 summarizes the mean spacings $s_{rm}$ between cracks obtained by image correlation in the central area of the four walls. The geometrical projections $s_{rmx}$ and $s_{rmy}$ defined in Fig. 7 are calculated from the observed $\theta$ angle, by the equations:

$$s_{rm} = s_{rmy} \cos \theta = s_{rmx} \sin \theta$$

![Figure 7. Crack spacing](image)

<table>
<thead>
<tr>
<th>Test n°</th>
<th>Specimen</th>
<th>Angle [°]</th>
<th>Mean spacing [mm]</th>
</tr>
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<td></td>
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<td>Right force</td>
</tr>
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<td></td>
<td></td>
<td>$s_{rm}$</td>
<td>$s_{rmx}$</td>
</tr>
<tr>
<td>1</td>
<td>Concrete C25</td>
<td>29,5</td>
<td>88,4</td>
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<tr>
<td></td>
<td>HA10 @ 100×100mm</td>
<td>33,2</td>
<td>180</td>
</tr>
<tr>
<td>2</td>
<td>Concrete C40</td>
<td>30,1</td>
<td>98,6</td>
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<tr>
<td></td>
<td>HA10 @ 100×100mm</td>
<td>29,6</td>
<td>114</td>
</tr>
<tr>
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<td>Concrete C40</td>
<td>28,4</td>
<td>98,0</td>
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<tr>
<td></td>
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<tr>
<td></td>
<td>HA8 @ 80×80mm</td>
<td>30,5</td>
<td>140</td>
</tr>
</tbody>
</table>

The review of Table 1 can be summarised in the following observations:

a) The mean value of spacing (accounting for the two loading directions) for tests 2 and 3 is 98,4 mm, while it is 80,5 mm for test 1, i.e. a relative difference of 22%. This highlights the influence of the concrete strength, which is not taken into account by the formulas of the codes.

b) Note the remarkable stability of the values obtained for tests 2 and 3.

c) The mean value is 119,9 mm for test n° 4, i.e. 22% more than for tests 2 and 3.
These results are compared in Table 2 (average values of the two loading directions) with those given by the formulas of the fib Model Code 2010. In order to compare the mean crack spacing and width experimental values with the higher characteristic values of MC10 formulas, the latter are divided by 1.5, a commonly accepted ratio. The calculations are performed with the average crack angle between the two loading directions. Both calculations using the different covers for the two rebars layers and taking the average cover are performed.

Table 2. Comparison of crack spacing in horizontal and vertical directions

<table>
<thead>
<tr>
<th>Test n°</th>
<th>Mock up</th>
<th>Angle [°]</th>
<th>Mean spacing [mm]</th>
<th>Angle [°]</th>
<th>Mean spacing [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean value</td>
<td>Experimental</td>
<td>Theory (MC10)/1,5</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Concrete C25 HA10 @ 100×100mm</td>
<td>31</td>
<td>156 95 80</td>
<td>101 174 137 101</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Concrete C40 HA10 @ 100×100mm</td>
<td>30</td>
<td>198 114 99</td>
<td>101 174 137 101</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Concrete C40 HA10 @ 100×100mm</td>
<td>28</td>
<td>206 111 98</td>
<td>101 174 137 101</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Concrete C40 HA8 @ 80×80mm</td>
<td>30</td>
<td>238 139 120</td>
<td>96 153 125 92</td>
<td></td>
</tr>
</tbody>
</table>

Formulas assume that $s_{rmax}$ and $s_{rmy}$ are the spacings between cracks in two virtual tie beams in horizontal and vertical directions respectively, taking into account the reinforcement present in each direction; this is a strong physical hypothesis (Vecchio and Collins, 1986).

In the MC10 formula, the effective steel ratio is involved in the term $\phi/\rho$. Taking the mean cover between each layer of rebars or the cover of each layer separately leads to the same mean spacing $s_{rm}$ when the formulas are applied, so both approaches are equivalent. The obtained values are 101 mm for $\phi$ 10 bars and 92 mm for $\phi$ 8 bars, i.e. a decrease of 8.7% instead of the observed increase.

Thus, the MC10 formula gives a lower spacing in the case of wall 4 than in the other cases, but this is contrary to experimental evidence. Other assumptions being laid down, it appears that the expression of the effective thickness is the cause of this contradiction. Taking the total thickness as the effective thickness does not solve the problem, since the calculation shows that the spacing should be unchanged between the walls with $\phi$ 10 rebars and the wall with $\phi$ 8 rebars, which is better but still not satisfying. Therefore, it should be necessary to consider the tri-dimensional stress field, which is at the origin of the definition of the effective area, in a different manner.

The experimental values given in Table 2 are the geometrical components of the spacing, as shown in Fig. 7. The angle $\theta$ being significantly lower than 45°, the geometrical projection $s_{rmax}$ may be greater than the crack spacing in this direction, and this fact is observed here. Then, it would be reasonable only in the $y$ direction to assume that the geometrical projection $s_{rmy}$, and the spacing of the vertical virtual tie beam are comparable, which is not the case since the values from the formulas are higher than observed. Note that taking account of the mean cover gives a better result in the $y$ direction. Thus, the hypothesis of horizontal and vertical virtual tie beams is not verified, but finally, the calculated values of $s_{rm}$ fit much better with the experimental values than for the components.

As already mentioned, the DIC allows a good evaluation of the crack width mean value and the associated standard deviation (Fig. 8), for a single crack or a set of cracks. The obtained values can be compared to those provided by the LVDTs. There must be differences between these two types of measure in reason of:

- from observation, it can be assumed that cracks are through cracks, but they are not measured on the same face of the wall;
- when a LVDT is installed, the crack is already open, because it is visible; the initial value of the width remains within the range determined by the precision of the measurements (50 $\mu$m);
- the LVDT measure corresponds to a particular point along the crack.

Fig. 8 shows the comparison of the evolution of crack widths in the wall 3, obtained by DIC and LVDT. The blue continuous line indicates the average of the DIC results and the dotted lines indicate
the dispersion of measurements along the crack. The correlation between these two types of measures is excellent in the case of sensor 5, but less good in the case of sensor 7, especially at the beginning of the measurements.

In order to compare the results with the formulas of the standards, it is necessary to make a structural analysis to obtain the evolution of strains and stresses.

4. STRUCTURAL ANALYSIS

The shear wall specimen is modelled (Ruocci et al., 2013) by shell elements (Fig. 9) in the FE software Code_Aster developed by EDF R&D. The mortar layer which connects the lower beam with the steel frame is modelled by several spring elements in order to represent the real boundary conditions. Two couples of opposite forces are applied on two steel plates which redistribute the load over the extremities of the lower beam, in order to reproduce the post-tensioned bars effect, which fix the beam to the steel frame in the vertical direction. The horizontal external load is distributed to four nodes to give a resultant applied at the same height as the real load.

First, a linear elastic analysis is performed. Then, to perform nonlinear analyses based on a realistic behaviour of the wall, a global and a semi-global modelling approaches are used. In the global modelling, the macroscopic constitutive law $GLRC_{DM}$ (Markovic et al., 2007) is used to model the reinforced concrete behaviour. Material properties are directly described by homogenised parameters that take into account both steel bars and concrete behaviours. This nonlinear constitutive law is used for RC shell structures and models the damage due to membrane stresses (the case of the test specimen) and to the bending ones. The global RC material is isotropic elastic before the damage threshold. The post-elastic behaviour of $GLRC_{DM}$ is modelled according to a linear law.

In the semi-global approach, steel and concrete are modelled separately, with several layers modelling the concrete. The stress integration is performed throughout these layers. In Code_Aster, the concrete local nonlinear constitutive law called ENDO_ISOT_BETON ($EIB$) is used for each of the 11 concrete layers considered, and an isotropic hardening plasticity constitutive law is used for
reinforcing steel layers. The concrete nonlinear constitutive law EIB describes a concrete local damage mechanism with stiffness restoration effect and aims at describing concrete fracture in tension (Badel et al., 2007).

A comparison of these two constitutive laws in a pure tension numerical test is presented in Fig. 10. The models are tested under monotonic loading, which can be compared to the envelope of cycles of wall 3. Fig. 11 shows the global force-displacement curves obtained with the two nonlinear constitutive laws in the FE model and their comparison with experimental results. GLRC_DM shows a good agreement with test results when compression damage coefficient is set to $\gamma_c = 0.28$, although this value implies a compression damage influence that seems too much important. EIB law does not present such a good agreement; a reduction of the $f_{ct}$ value in the FE computations, consistent with the observed “early cracking”, may allow a better accordance between numerical and experimental results.

Crack widths obtained by using GLRC_DM, EIB and linear elastic constitutive models are compared to experiment. In the last case, values are taken at the peaks of the cycles. Stresses obtained from the three models are used, together with the MC10 formula (see § 5), with $\beta = 0.35$, which has been found as the better value for the tension stiffening effect. Fig. 12 shows the evolution of crack width vs. force for two sensors in the central zone (see location in Fig. 6).

For each sensor, the experimental spacing is taken into account in the calculations, so that the comparison highlights the difference of mean deformations.

This comparison leads to the following conclusions:

- The different models provide the right order of magnitude of the angle of the cracks, compatible with the required accuracy.
- The quality of the results for crack width is highly variable depending on the used constitutive model; non-linear models underestimate it more frequently. Taking into account precisely the boundary conditions plays also a considerable role. This highlights the fact that the result essentially depends on structural modelling and on the choice of the model parameters. The linear elastic model gives acceptable results. A reduced concrete Young modulus (which takes into account concrete damage due to cracking) gives better results, reflecting that the interactions with the steel frame and the boundary conditions are strong and that a lower Young modulus represents the relative flexibility of the wall with respect to the steel frame.
5. CRACK WIDTH ASSESSMENT

The mean crack width is determined as the product of the average crack spacing $s_{crm}$ and of the difference between mean steel strain $\varepsilon_{sm}$ and mean concrete strain $\varepsilon_{cm}$.

The Cracked Membrane Model (CMM) proposed by Kaufmann et al. (1998) is a theory for evaluating cracks in RC membrane elements subjected to shear stresses, based on a macroscopic approach. It incorporates some elements from the Modified Compression Field Theory (Vecchio and Collins, 1986) and the Tension Chord Model (Marti et al., 1998). Four main assumptions are adopted: (i) the crack is stress free and it is able to rotate; (ii) concrete principal stress and strain directions are coincident; (iii) the crack direction is perpendicular to the principal tensile direction of average strains; and (iv) the bond shear stress between concrete and steel bars is constant in space and time. For the sake of brevity, the details of the method are not recalled here.

According to MC10, crack width is obtained from the bar transmission length $l_{s,max,\theta}$ and the mean differential deformation between concrete and steel, in the direction perpendicular to the crack:

$$w_d = 2 \cdot l_{s,max,\theta}(\varepsilon_{sm\perp} - \varepsilon_{cm\perp})$$  \hspace{1cm} (2)

$\theta$ can be taken as the angle of the direction perpendicular to the principal direction of tension if the considered combination of loads corresponds to that which creates the crack (this hypothesis is assumed hereafter); if not, the angle is already known and the procedure should be adapted for considering the force tangential to the crack.

However, MC10 does not give precise guidance to calculate the second term of (2), since it only refers to CMM (or equivalent). In the present paper, the following method is proposed.

Once the angle of the crack is determined, the force $N_r$ applied on a unit length of crack is obtained from the FE analysis. This force is balanced by the tensions in the reinforcement (Fig. 13). The mean deformation of each layer of bars is then calculated by applying the MC10 equations, taking into account the force transferred to concrete by tension stiffening.

Thus, the mean deformations $\varepsilon_{s,xx}$ and $\varepsilon_{s,yy}$ of the rebars are obtained. The distortion between orthogonal bars is non-zero, but it can be neglected between cracks, because there it is due to the distortion of the concrete. Thus, the mean deformation due to reinforcement in the direction orthogonal to the crack is obtained by the usual tensorial relationships.

$$\varepsilon_{sm\perp} - \varepsilon_{cm\perp} = \frac{1}{E_s} \left( \frac{N_r}{2h_{eff}} - \beta f_{ct} \right) \left( \frac{\sin^2 \theta}{\rho_{xx,eff}} + \frac{\cos^2 \theta}{\rho_{yy,eff}} \right) - \beta \alpha_f f_{ct} + \frac{\alpha_e N_{r//}}{h}$$ \hspace{1cm} (3)

where $h_{eff}$ is the effective thickness, as defined in MC10.
Equation (3), although it seems physically sound, can be improved, because it does not insure the compatibility of deformations. Indeed, in the physical model proposed by MC10, when the crack pattern is established, there is no slip between the bars and the concrete in points O, Mx, and My (Fig. 14), which leads to an alternative expression (“MC10 modified”) for the crack width:

\[ w = \frac{E_r}{2} \left( \epsilon_{s,xx} + \epsilon_{s,yy} - \epsilon_{c//} - \epsilon_{c\perp} \right) \]  

(4)

where

\[ \epsilon_{c//} + \epsilon_{c\perp} = \frac{1 - \nu}{E_\text{ch}} \left( N_{ij} + 2\beta f_{ct} \bar{h}_{\text{eff}} \right) \]  

(5)

Fig. 15 shows a comparison between the experimental results from the CEOS.fr test 3 and the numerical post-processing ones. For each sensor, the same crack spacing (experimental) is used for the different curves, so in fact only the differential deformations are compared. The numerical results are obtained from the linear elastic analysis with a reduced stiffness, and deformations are obtained using CMM, MC10 and MC10 modified. The location of the sensors is given in Fig. 6.

Figure 15. Crack width assessment: comparison of different formulation of the differential deformation with experimental results

The elastic model used with the “MC10 modified” formula gives reasonable results, except in the crack formation domain, which is logical. However, it is noted that this encouraging result is obtained for the CEOS.fr walls where reinforcement is the same in both directions.

In addition, it is observed that the strain compatibility also imposes a variation of distortion along the crack: referring to Fig. 16, it is noted that the distance between Mx and My remains constant if the concrete deformation is neglected. This leads to the following relationship between the distortion and the crack width:

\[ \alpha = 2\epsilon_{r,xy} = 2 \frac{w}{s_f} \sin \theta \cos \theta \]  

(6)

Figure 16: Distortion in the vicinity of the crack due to compatibility of deformation when the crack opens

In Fig. 16, the distortion obtained by dividing the relative horizontal displacement between the top and the bottom of the wall by the height is reduced according to Eq. 6, so that it can be compared with the mean crack width. The two curves fit well in the middle of the test loading. At the beginning,
cracking is not stabilised and the crack width is wider. At the end, the influence of the distortion in the concrete between cracks is greater because the increase of its damage.

6. CRACKING DUE TO CYCLIC LOADING

In one hand, the fact that the considered limit state is an ULS and not a SLS should not, in principle, change the spacing between cracks. Indeed, the only parameter that can be affected by the level of stress is the bond stress $f_{hm}$ between concrete and steel bars. If the load increases, the bond stress increases, but there is a full slip between bars and concrete and the spacing is therefore not affected.

In the other hand, the mean differential deformation is affected by the value of the tension stiffening which has to be reduced by correcting the MC10 $\beta$ factor, given for a SLS situation, with a reduction factor $\alpha$ ($\alpha \leq 1$), which is of the order of 2/3 at ULS:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - \alpha \beta f_{cm} \rho_{s,eff} (1 + \alpha \rho_{s,eff})}{E_s} \quad (6)$$

In the case of the walls subjected to shear under the seismic action, each cycle causes inclined cracking depending on the direction of seismic forces, and when the applied force direction is reversed this set of cracks closes and another family of cracks opens symmetrically to the vertical. Walls 2 (cyclic reversing) and 3 (cyclic non-reversing) are identical and the results may only differ due to the reversing character or not of the applied force. Crack pattern of walls 2 and 3 are shown in Fig. 16. The analysis of the results shows that the inclination angle and the spacing between cracks are not significantly affected by the direction of thrust.

![Figure 16. Stabilised cracking pattern at 4200 KN for walls 2 (left) and 3 (right) – origin CSTB](image)

The global behaviour is apprehended by the horizontal displacement vs. applied force curves, shown in Fig. 17 for walls 2 and 3.

![Figure 17. Global behaviour of wall 2 (reversing cycles) and wall 3 (non-reversing cycles) – CSTB](image)

These curves show a fairly classical behaviour, with including:

- a progressive softening with the development of cracking;
- pinched hysteresis cycles reflecting the predominance of cracking in damage, with the materials remaining quasi-elastic;
- at each level of loading, three cycles are applied; they are almost superimposed, which means that the concrete is mainly damaged by cracking, except when near to rupture;
- for the non-reversing loading (wall 3), cracks do not close completely when unloading;
- for the reversing loading (wall 2), the shape of the cycles at small loading shows a full crack re-closing when the sense of the thrust is reversed.
The ultimate strength is not reached by a rupture in the central zone but by a sliding along the plane between the wall and the upper beam, extended by a diagonal plane joining the horizontal bottom support. This mode has been perfectly reproduced for the four walls. The ultimate strength is higher than expected, and in the central part the yield stress is not attained in the steel bars.

Fig. 18 shows the envelope curves of the cycles for the four walls. The resistance is essentially determined by the concrete strength: with a lower concrete strength value, the wall 1 appears as less resistant, softer and more ductile than walls 2 and 3, although the reinforcement is the same. This shows, if a proof was needed, that the concrete plays an important role in the resistance to shear. Wall 4, less reinforced than the others, shows a particular behaviour with a rapid decrease of resistance after the peak followed by a stabilisation.

![Figure 18. Envelop curves of cycles for walls 1 to 4](image1)

![Figure 19. Cycles of cracks width in wall 2](image2)

In order to judge the reversing cycling effects, the curves of walls 2 and 3 are compared. It appears that reversing cycling has an effect on the stiffness and the strength of the wall, which are lower in the case of wall 2. The physical explanation is that the existence of two symmetrical crack families increases the concrete damage, notably in the cycles of crack opening and re-closing, which are not completely reversible. This difference is not very important (of the order of 10%) and can therefore be taken into account as a first approximation by a reducing factor.

In the case of wall 3, it is observed that the residual width increases when increasing the applied load. In the case of wall 2, the cracks close at each cycle when the thrust is reversed.

Fig. 19 shows the evolution of cracks width in wall 2 within a window of time. Sensors D2 and D4 measure crack widths (negative) due to the right force, while D10 and D11 measure crack widths due to the left one. No significant difference appears between these two sets of cycles.

<table>
<thead>
<tr>
<th>No test</th>
<th>Mean width [mm]</th>
<th>Maximum width [mm]</th>
<th>Quantile 95% of width [mm]</th>
<th>Standard deviation of width [mm]</th>
<th>Residual width after unloading [mm]</th>
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<tr>
<td></td>
<td>Left force</td>
<td>Right force</td>
<td>Left force</td>
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<tr>
<td>3</td>
<td>0.118</td>
<td>-</td>
<td>0.364</td>
<td>-</td>
<td>0.222</td>
</tr>
<tr>
<td>4</td>
<td>0.147</td>
<td>0.207</td>
<td>0.377</td>
<td>0.475</td>
<td>0.263</td>
</tr>
</tbody>
</table>

Table 3 shows the values obtained by image correlation that characterise the cracking at the last cycle for the four walls. In the table, the average ratio of the residual to the mean width is 43% while the ratio to the characteristic width is 24%, but this is only indicative since it appears that the residual width is obtained early in the test loading and it is probably due to the aggregate interlock (as the tangential displacement at the crack is of the same order as the normal one) and the local concrete damage. However, the CEOS.fr experiments do not fully conclude on the residual width after an earthquake loading, since ULS is not reached in the central part (it has been observed that the level of
stresses corresponds to a SLS)). At this level of load, except for wall 3 with non-reversing loading, there is no evidence that the residual width increases with the maximum crack width (Fig. 19).

7. CONCLUSIONS

The tests of RC walls performed in the frame of the French national project CEOS.fr have provided interesting results for the assessment of cracking in both normal and seismic situations. Among these results, those presented in the present paper lead to the conclusions below:

- the Vecchio and Collins assumption of virtual horizontal and vertical tie beams is questionable, but, under some conditions, it gives acceptable values for the cracks spacing;
- the tensile strength of concrete has an influence on crack spacing which should be considered;
- the definition of the effective area as in MC10 and EC2 is not appropriate;
- taking the mean value of covers of the two layers of bars is acceptable to calculate the spacing;
- the quality of the numerical results regarding the crack width is highly dependent on the structural model, particularly on the boundary conditions and the constitutive law used;
- two formulas are proposed to assess the mean differential deformations between rebars and concrete, which fit well with the CEOS.fr walls experimental results;
- reversing cycling creates a symmetrical bidirectional crack pattern, which slightly affects the stiffness and the maximum strength of the wall;
- using the previous results, the crack pattern and crack width in case of earthquake can be assessed, taking into account a decrease of the tension stiffening effect;
- the magnitude of the residual crack width is not in proportion to the maximum crack width attained during the movement; in the CEOS.fr test it remains below 0.1 mm.

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REFERENCES

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