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Implementation of a macro model to predict seismic response of RC structural walls

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Abstract. A relatively simple multiple-vertical-line-element macro model has been incorporated into a standard computer code DRAIN-2D. It was used in blind predictions of seismic response of cantilever RC walls subjected to a series of consequent earthquakes on a shaking table. The model was able to predict predominantly flexural response with relative success. It was able to predict the stiffness and the strength of the pre-cracked specimen and time-history response of the highly nonlinear wall as well as to simulate the shift of the neutral axis and corresponding varying axial force in the cantilever wall. However, failing to identify the rupture of some brittle reinforcement in the third test, the model was not able to predict post-critical, near collapse behaviour during the subsequent response to two stronger earthquakes. The analysed macro model seems to be appropriate for global analyses of complex building structures with RC structural walls subjected to moderate/strong earthquakes. However, it cannot, by definition, be used in refined research analyses monitoring local behaviour in the post critical region.

Keywords: nonlinear analysis; seismic analysis; reinforced concrete walls; macro model; benchmark study.

1. Introduction

Good behaviour of reinforced concrete structural walls during strong earthquakes has been observed in the past. This is due to their inherent strength, overstrength (additional strength in access of the strength required by the code), stiffness, and, if properly designed, ductility. Nevertheless, frames have been preferred in many regions of the world and the research of the seismic response of structural walls has been rather limited. This limited knowledge has been reflected in the design codes as well as in the analytical models used in the practice and even research. Therefore, simple beam-column element, extrapolated from the frame analysis has been frequently used to analyze seismic response of RC walls.

However, there has been growing evidence that the behaviour of structural walls near collapse is complex and difficult to predict. The problem was first discussed in detail many years ago after the well known full-scale tests of 7-storey RC frame-wall structure (US-Japan Program 1984), when simple beam elements had failed to simulate complex interaction between frame and the wall entering into

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inelastic region.

Refined FEM element models monitoring inelastic stress-strain relationship (“micro” models) offer an appropriate solution. Indeed a lot of progress has been done in this field during the last twenty years (i.e., Moazzami and Bertero 1987, Lefas and Kotsovos 1990, Inoue, *et al.* 1997). However, the implementation of more sophisticated FEM approach may still involve difficulties due to the lack of a completely reliable model and data, the complexities involved in the numerical solution and the need for expert users.

The alternative approach involves so-called “macroscopic” models. These models attempt to describe the overall behaviour by simplified idealizations based on empirical evidence as well as by direct monitoring of force-displacement relationships. As such they may be suitable for efficient use in the inelastic response analysis of complex building structures. However, due to their empirical basis, the general validity of such models may be questionable.

This paper refers to a specific macro model for the analysis of seismic response of RC structural walls – multiple-vertical-line-element model (MVLEM). The model, which is described in more details in the following sections, was originally proposed by Japanese researchers (Kabeyasawa, *et al.* 1983) and later modified by Vulcano and Bertero (1989), as well as by the first author of this paper (Fischinger, *et al.* 1992).

In early nineties the model was successfully applied in several analyses (Fischinger, *et al*. 1992). However, all of them were post-experimental. Recently, there has been a new opportunity for testing and further development of the model in the context of a series of benchmark “CAMUS” (Conception et Analyse des Murs sous Séisme) studies performed in France in support of the development of Eurocode 8 (EC8). CAMUS benchmark studies were organized by Commissariat à l’Energie Atomique, Electricité de France and GEO research network under auspices of the French Association for Earthquake Engineering. CAMUS 1 analysing seismic response of lightly reinforced cantilever structural walls, designed according to the French practice, was performed in 1997 (Combescure and Chaudat 2000). A large-scale cantilever wall specimen was tested on the Azalée shaking table at the CEA (Commissariat a l’Energie Atomique – Centre d’Etudes de Saclay, Laboratoire d’Etudes de Mécanique Sismique). CAMUS 3 followed the same experimental set-up, but this time the ductile wall was designed according to the EC8 standard (Eurocode 8). CAMUS 2000, testing 3D response of structural walls, was completed by summer 2003. The paper is concentrated on the results of the CAMUS 3 benchmark.

2. ‘CAMUS 3’ test

2.1. Specimen description

The 1/3rd scaled CAMUS 3 model consisted of two parallel cantilever RC walls without openings linked by RC floors (Fig. 1). The reinforcement was designed according to the EC8 design and detailing rules (Figs. 2a and 2b). The floors were linked by six square floors 1.7 × 1.7 m (including the floor connected to the footing). Each part of the structure (wall, floor, basement) was cast separately. The parts were assembled (bolted) on the shaking table. Bolts connecting walls and floors are visible in Fig. 1. The stiffness and the strength in the direction perpendicular to the planes of the walls were increased by lateral triangular bracing. The bracing was such that the two walls carried the entire vertical load. More details are given in Combescure and Chaudat (2000). The reinforcement of the model approximately corresponded to the design ground acceleration

Fig. 1 CAMUS test setup

0.75g and behaviour factor (seismic force reduction factor) $q = q_0 \times k_D \times k_w = 4 \times 0.75 \times 0.85 = 2.55$ (q_0 is basic value of the behaviour factor for uncoupled RC walls, k_D is ductility factor for DC"M" and k_w reflects prevailing failure mode of the relatively low wall). CAMUS 3 has not established clear relation between the model wall and actual prototype structures. Therefore, the subsequent comments relate predominantly to the behaviour of the model. It is believed, however, that they can be generalized to the prototype structures.

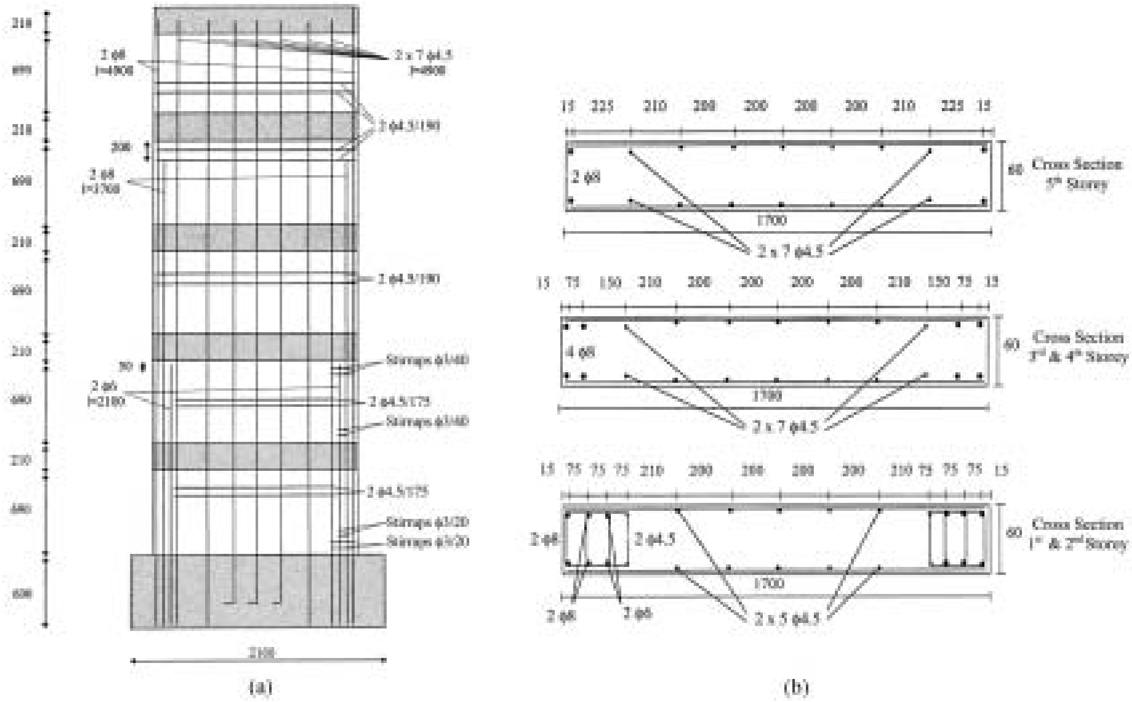


Fig. 2 (a) Reinforcement front view (b) Reinforcement cross section (dimensions are in millimeters)

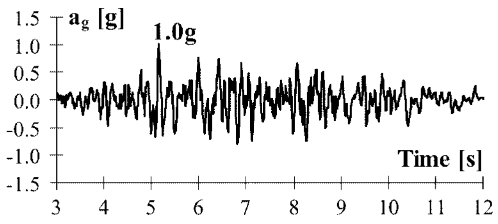


Fig. 3 Accelerogram Nice 1.0g

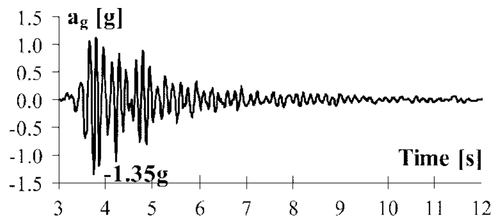


Fig. 4 Accelerogram Melendy Ranch 1.35g

It is important to note that steel bars with substantially different properties were used for reinforcement. Bars with smaller diameter (3.0 mm to 6.0 mm) had high strength (593 MPa – 814 MPa) but very low ultimate strain capability (0.9 % – 3.5%), while larger bars (diameter of 8.0 mm) exhibited large ultimate strain (16.8 %). Average compressive strength of the concrete was 39.6 MPa and Young's modulus E = 31139 MPa.

2.2. Loading program

The generated accelerogram "Nice" was used as the main input signal (Fig. 3). An intermediate test was performed with the Melendy Ranch accelerogram (Fig. 4). This signal has represented a "near-fault earthquake". This is a short signal, but it has high value of acceleration and its frequency content corresponded to the natural frequency of the CAMUS 3 cracked specimen ($T_{1,cracked}$ = 0.155 s; Fig. 5). The loading is described in more detail in CAMUS 3 (1999).

Acceleration histories used in the analyses were measured on the shaking table. Maximum accelerations for the main tests are given in Table 1. The sequence of the loading was Nice 0.42g, Nice 0.22 g, Melendy Ranch 1.35 g, Nice 0.64 g and finally Nice 1.0 g.

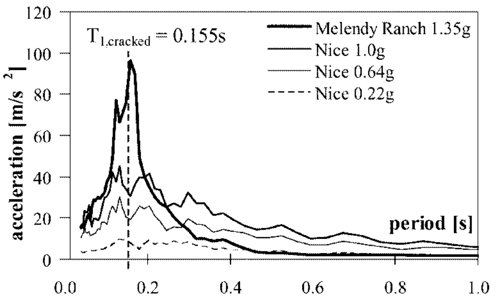


Fig. 5 Acceleration spectra for 2% damping

Table 1 Maximum acceleration of the table for the main seismic tests

Test	Nice	Nice	Melendy Ranch	Nice	Nice
Acceleration	0.42g	0.22g	1.35g	0.64g	1.0g

For better understanding of the results, it is important to note that due to the problems in the control of the shaking table:

- relatively strong accelerogram Nice 0.42 g was applied at the very beginning of the loading program before the Nice 0.22 g, which had been supposed to be the first.
- Melendy Ranch 1.35g, was originally planned much weaker (0.90 g). In fact, it was 50% stronger, resulting in the application of the near collapse earthquake in the middle of the testing program (note again that the initial period of the specimen was close to the extreme peak in the spectrum - Fig. 5).

2.3. Summary of the 'CAMUS 3' experimental results

The EC8 designed wall behaved well. The wall survived much stronger loading than it was designed for. Capacity design worked as expected. Inelastic flexural deformations were confined to the first floor. Shear failure was precluded even in the case of severe loading. Confining of the edges of the wall was adequate to prevent complete failure up to the end of the test program.

The benchmark organizers reported (Comberscure and Chaudat 2000) that the wall had been pre-cracked before the test when the walls and floors were bolted together. Visible cracks were documented on the level of the second and the fourth floor. Due to the error in the control of the shaking table during the calibration phase the already pre-cracked wall was further subjected to quite strong earthquake Nice 0.42 g. There was no visible additional damage after this loading. The natural frequency was also not changed significantly (from 6.88 to 6.44 Hz). However, the frequency that corresponded to the peak of the spectrum of the measured signal changed much more – to 5.5Hz (according to Fourier analysis done by Comberscure and Chaudat).

The Melendy Ranch seismic input motion caused important damage to the mock-up with extensive cracking and beginning of crushing at the wall edges. Permanent displacements were observed at the end of the sequence, residual cracks and significant yielding of the reinforcement bars. A large crack appeared throughout the base of each wall.

After the failure test (Nice 1.0g) almost all the vertical steel reinforcement bars were broken and buckled just above the level of the 1st construction joint. The zone, where rupture of the bars took place, followed the main cracks at the base (Fig. 6).

The displacement transducer at the top 6th storey (height: 5 m) gave few reliable results. Therefore the values measured at the 5th storey (height: 4.1m) are discussed in the paper.

Fig. 6 Specimen at the end of the test (right wall)

Significant axial force variations appeared at the medium and high level accelerations. The weight was sometimes doubled or reduced to zero. Even a net tension was observed during the Melendy 1.35g test. Ultimate bending capacity about 420 kNm was foreseen. The wall entered well into the inelastic range during the Melendy 1.35 g loading. The higher value, reached during the response (510 kNm), can be explained by the influence of the varying axial force.

3. Multiple-vertical-line-element model (MVLEM)

The MVLEM element consists of several vertical springs connected by the rigid beams at the top and bottom level (see Fig. 7). It has 3 degrees of freedom in local coordinate system. First one represents the axial deformation of the element, while second and third represents element's flexural deformations.

The stiffness matrix of the element in the local coordinate system is:

$$[\bar{K}] = \begin{bmatrix} \sum_{i=1}^{N} k_i & -\sum_{i=1}^{N} k_i z_i & \sum_{i=1}^{N} k_i z_i \\ & k_H c^2 L^2 + \sum_{i=1}^{N} k_i z_i^2 & k_H c(1-c)L^2 - \sum_{i=1}^{N} k_i z_i^2 \\ sym. & & k_H (1-c)^2 L^2 + \sum_{i=1}^{N} k_i z_i^2 \end{bmatrix}$$

The axial stiffness of the element is calculated as the sum of the stiffness of all springs. The axial stiffness of each spring is determined by hysteretic rules (see Fig. 8a). The flexural stiffness is also determined based on the axial stiffness of single springs, using Steiner's rule. The element takes

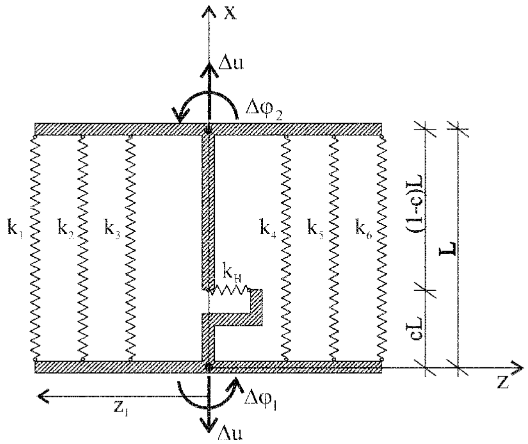
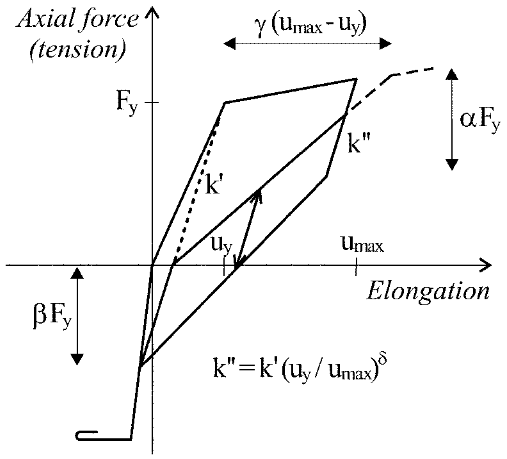
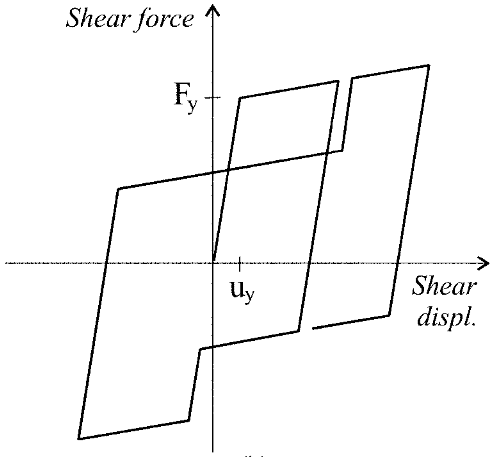


Fig. 7 Multiple-vertical-line-element model (MVLEM)



(a)

Fig. 8a Vertical spring properties



(b)

Fig. 8b Horizontal spring properties

into account shear deformations, too. The horizontal spring (see Fig. 7) of the model accounts for shear deformations. The stiffness of the horizontal spring is also determined by the hysteretic rules (see Fig. 8b).

Standard values for parameters α = 1.0, β = 1.5, γ = 1.05, δ = 0.5, and c = 0.3, as had been proposed in Fischinger, *et al*. (1992), were used in the presented studies.

4. Blind prediction of the 'CAMUS 3' response

4.1. Basic assumptions and research approach

The main purpose of the research was to check the ability of a relatively simple macro model and standard computer code to predict a predominantly flexural inelastic response of a RC structural wall.

A standard computer code for inelastic time history analysis of 2D structures DRAIN-2D, which has been frequently used by the earthquake engineering community for the last three decades (Kanaan and Powell 1973), was used. It was modified to include MVLEM. The original version of the program uses relatively simple numerical procedures, based on the constant acceleration within the time interval. There is no iteration within the integration time step and the out of balance forces are applied at the beginning of the next interval.

The sequence of all four (five) accelerograms was used in the analysis instead of separate individual records. 2% of critical damping was considered. Standard parameters (Fischinger, *et al.* 1992) were used to define hysteretic rules for vertical springs. However, Melendy Ranch as well as Nice 3 signals imposed a severe demand on the wall, causing quite an unstable response of the chosen model. DRAIN-2D program was not able to cope with this problem. Therefore, the authors were forced to use 5% hardening ratio of the vertical springs to stabilize response.

4.2. Modeling of the wall

The wall was modelled as a stack of MVLEs. According to the EC8 design procedure inelastic deformation of reinforcement should be confined into the first storey. Therefore, 9 unevenly distributed MVLE were used in the first storey to assure relatively constant curvature distribution along the height of each element. The horizontal spring was located at 30% of the height of the element. This choice was made based on the previous experience to account for the variation of the curvature along the height of the element. According to previous experience, 6 vertical springs were chosen for each MVLE.

4.3. Modeling of vertical springs

Each vertical spring was modelled as a RC truss element. Only the contribution of concrete was considered to determine the strength and stiffness in compression. Only the contribution of reinforcement was considered to determine the strength in tension. This supposition was considered acceptable at later phases of response. The contribution of both, concrete and reinforcement, was considered to determine the stiffness in tension. To determine this average stiffness, the energy criteria was used for outer springs and the geometric average for the central springs. Namely, the model used for vertical springs in CAMUS 3 was bilinear in tension (tri-linear model has been only recently developed). Obviously, it is difficult to model RC truss element with bi-linear envelope. Before cracking the stiffness is close to the stiffness of concrete ($k_c = A_c E_c / L$) and after cracking close to the stiffness of the reinforcement bars only ($k_s = A_s E_s / L$). In the case of outer, heavily reinforced springs the yield force of the reinforcement was larger than the cracking force of the concrete. In such a case the tri-linear curve is relatively easily replaced by an equivalent bi-linear curve based on the equal area (energy) under both curves. For lightly reinforced central springs the yield force is typically less than the cracking force. DRAIN was not able to simulate the drop of the force after cracking (stabilised by neighbouring springs). It has been empirically observed that the geometric average of k_c and k_s yields adequate equivalent stiffness in such situation.

4.4. Modeling of horizontal springs

It was expected that the EC8 design should preclude shear failure. Nevertheless, a non-parametric multidimensional regression method (neural network) was used to predict the seismic capacity of the wall (Peruš, Fajfar, Grabec 1994). The method is using a database available from laboratory tests of 262 structural walls. According to this prediction, the failure mode should be flexural with slight elements of shear damage. Therefore, elastic shear model was used in the blind prediction.

5. Comparison of the predicted and experimental results

5.1. Low-level event (Nice 0.22 g)

Since the model had been designed for ground acceleration 0.75g / 2.55 = 0.3 g no yielding of reinforcement was expected at Nice 0.22 g test. However, due to the error in the control of the shaking table during the calibrating phase of the test, a relatively strong test (Nice 0.42 g) preceded low level Nice 0.22 g test. Benchmark participants did not get information about Nice 0.42 g in the

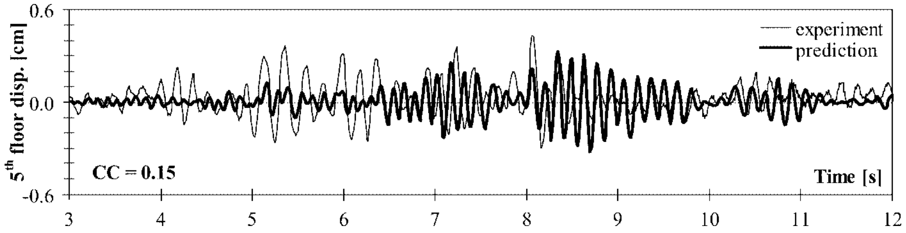


Fig. 9 5th floor displacement time-history (Nice 0.22g): comparison of the predicted and experimental results (preceding Nice 0.42g test was not considered in the analysis)

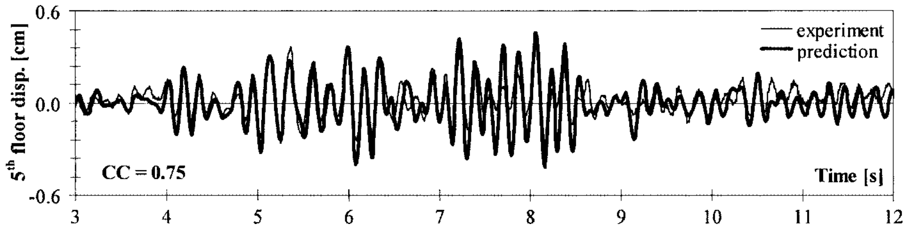


Fig. 10 5th floor displacement time-history (Nice 0.22g): comparison of the predicted and experimental results (preceding Nice 0.42g test was considered in the analysis)

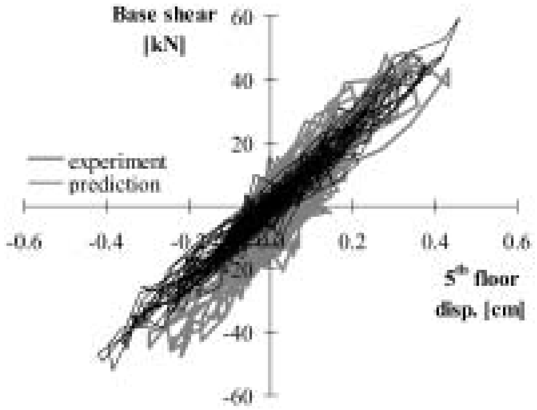


Fig. 11 Base shear - 5th floor displacement relationship (Nice 0.22g): comparison of the predicted and experimental results

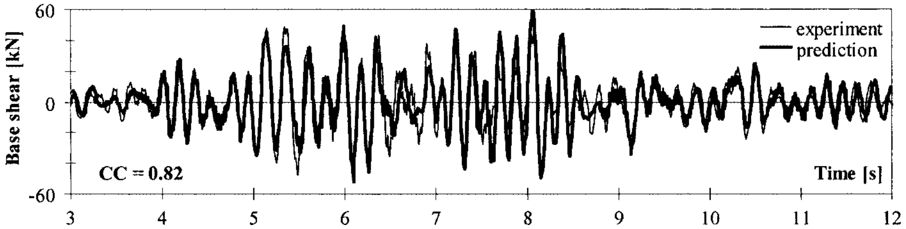


Fig. 12 Base shear time-history(Nice 0.22g): comparison of the predicted and experimental results

blind prediction phase of the project. In fact, initial natural frequency of the wall (6.88 Hz) did not change much after the Nice 0.42 g test (6.44 Hz). This probably led the organizers to the conclusion that Nice 0.42 g test could be neglected. However, the apparent frequency, estimated from the response (Combescure and Chaudat 2000) indicates significant drop of stiffness (estimated frequency is 5.0 Hz). This suggests that considerable cracks were opened during the 0.42 g event. It seems that these cracks closed by the end of the test, leading to the apparently undamaged structure.

In fact, this unintentional sequence of testing has provided an opportunity to study the influence of pre-cracking on the response of the RC wall. The 5th floor displacement time history for Nice 0.22 g is shown in Figs. 9 and 10 for two cases. Fig. 9 shows the case neglecting the preceding 0.42 g test. The predicted response has much higher frequency and smaller displacements in comparison with the test results. The correlation between experimental and predicted displacements is quite low (correlation coefficient is only CC = 0.15). However, if Nice 0.22 g is analysed in sequence with the preceding Nice 0.42 g test, correlation is much better (Fig. 10). The correlation coefficient is increased to CC = 0.75.

Acceptable correlation was also confirmed by base shear – 5th floor displacement relationship in Fig. 11. Considering a relatively crude bilinear model, the stiffness has been reasonably well predicted. Good results were also obtained within the CAMUS 1 project (Fischinger and Isaković 2000).

The prediction of the base shear time history was very close (Fig. 12). The correlation coefficient CC = 0.82.

5.2. Strong near fault event (Melendy Ranch 1.35g)

In general, the prediction was quite good as indicated by base shear - 5th floor displacement relationship in Fig. 13. The prediction of the displacement time history was good (Fig. 14), with the exception of the permanent offset, which is not shown in the analytic results (CC = 0.66). The reason for this discrepancy is the already mentioned unrealistic strain hardening (5%) of vertical springs, which was chosen in the model to stabilize numerical response. But this is not a conceptual problem of the model and it could be solved using a more advanced computer code as it was demonstrated by parallel analyses and using OpenSees code (McKenna and Fenves 2000a, 2000b).

What should be of greater concern is the fact that the experimental response indicates softening

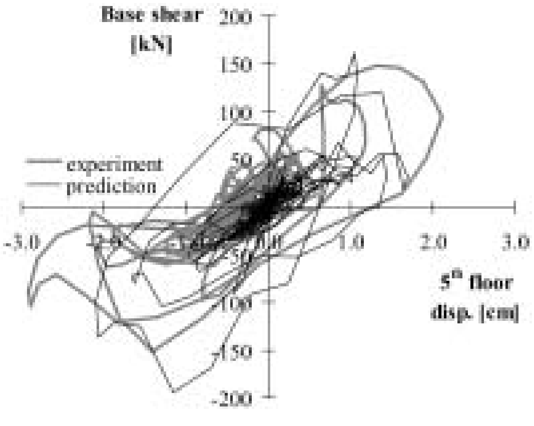


Fig. 13 Base shear - 5th floor displacement relationship (Melendy Ranch 1.35g): comparison of the predicted and experimental results

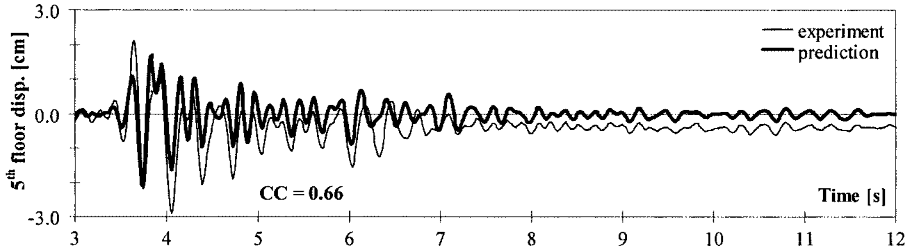


Fig. 14 5th floor displacement time-history (Melendy Ranch 1.35g): comparison of the predicted and experimental results

after the first strong cycle, which is not observed in the analytical prediction (see also hysteresis in Fig. 13). There has been strong experimental evidence that a short strong pulse of the Melendy Ranch signal heavily damaged the specimen at the very beginning of testing. The numerical model was not able to detect the local failure(s) in the wall automatically. In addition, the authors did not check the deformations at the first storey, which should indicate the failure of the brittle reinforcement in the boundary columns. The consequences of this failure were demonstrated in full extent during subsequent strong tests Nice 0.64g and Nice 1.00g. Therefore, they are discussed in more details in the following section and in the frame of the section dealing with the post-experimental analysis.

5.3. Near collapse behavior (Nice 1.0 g)

The 5th floor displacement time histories are shown in Figs. 15 and 16 for Nice 0.64 g and Nice 1.0 g response. Base shear – 5th floor displacement relationship for Nice 1.0g is shown in Fig. 17. The authors did not predict the response to Nice 0.64g and Nice 1.0g with acceptable accuracy. As

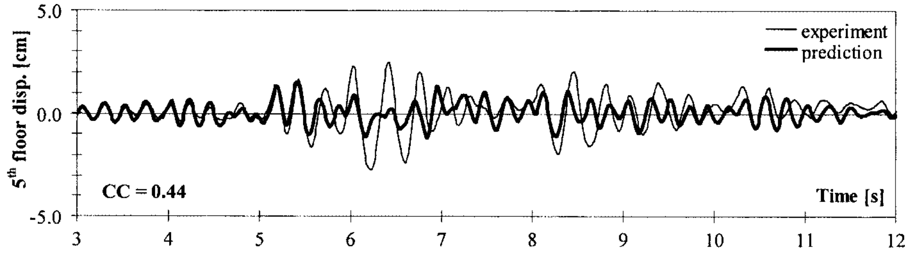


Fig. 15 5th floor displacement time-history (Nice 0.64g): comparison of the predicted and experimental results

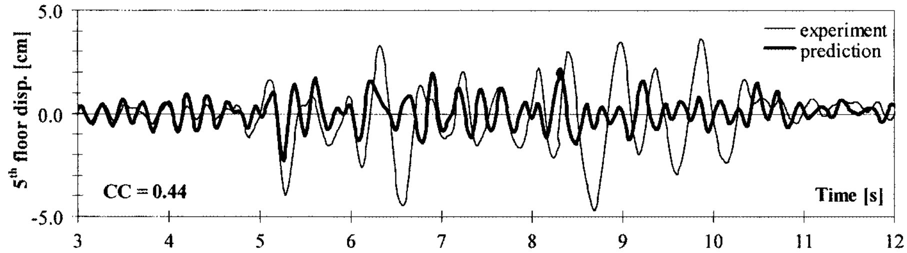


Fig. 16 5th floor displacement time-history (Nice 1.0g): comparison of the predicted and experimental results

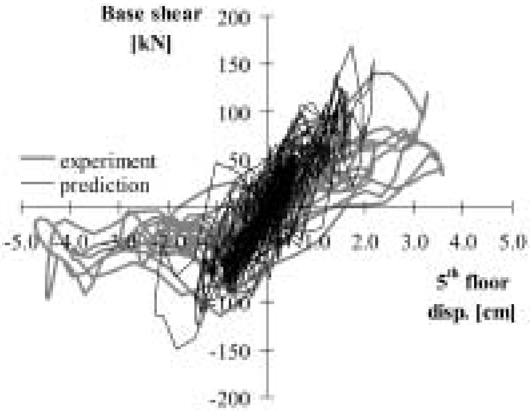


Fig. 17 Base shear - 5th floor displacement relationship (Nice 1.0g): comparison of the predicted and experimental results

mentioned in the previous section, the specimen was heavily damaged during Melendy Ranch test. From the report of experimental results (Combescure and Chaudat 2000) it is evident that most of the reinforcement bars failed at the end of the experimental program. However, there is a lot of evidence (strain gages at the base indicated values beyond measurement capacity, large cracks were opened) that some local failures (rupture of reinforcement) occurred already during the early Melendy Ranch test.

The authors were aware of this problem even at the time of the blind prediction. In the conclusions of the benchmark report they wrote: “It seems that the model was close to collapse during unusually strong excitations of Melendy Ranch and Nice 1.0g. For Melendy Ranch, the values in the acceleration spectra (for 2% damping) in the range of the predominant period of the wall were more than 10 times larger than design acceleration. If standard values 1-3% for hardening ratio (the ratio between post-yield and yield stiffness) were used, unstable response (very large displacements) of the model was observed. This was due to out-date numerical procedures used in DRAIN-2D. When the hardening ratio was increased to 5%, the response was stabilized. If then the authors checked the local deformation of the vertical springs in the first storey, they would notice that the strains are far beyond the ultimate strain capacity of the brittle reinforcement in the boundary columns. However, the authors did not make this check.

5.4. Varying axial force prediction

MVLEM has ability to model the rocking of the wall with the associated vertical displacement at the centre of the wall. This has always been considered as an important advantage in comparison with the conventional beam elements. The rocking results in vertical accelerations associated with varying axial force in cantilever wall, which was demonstrated in the benchmark study (Fig. 18).

However, the predicted effect was smaller than experimentally observed (in the experiment the axial force sometimes doubled and sometimes was reduced to zero or even tension). Post-experimental analytical studies (Fischinger, *et al.* 2000) showed that this had been due to neglecting the vertical flexibility of the shaking table.

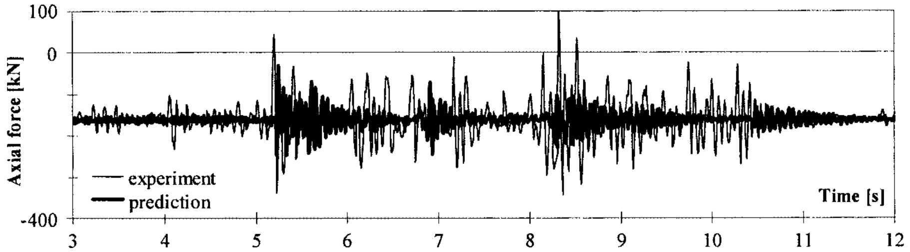


Fig. 18. Axial force time-history (Nice 1.0g): comparison of the predicted and experimental results

6. Post experimental analysis and calibration of the model

Several important lessons were learned by comparison of the predicted and experimental results. The following major sources of discrepancies between the predicted and measured results were identified and related modifications of the model were made:

· The most important source of discrepancy was the inability of the model and DRAIN-2D program to predict local failures and associated progressive collapse mechanism initiated in a relatively early stage of testing, followed by 2 additional strong events. Additionally, the authors failed to note that the predicted large deformations were greater than the deformation capacity of some brittle reinforcement bars in the boundary columns.

· Using the present version of the MVLEM incorporated into the DRAIN-2D program, it is not possible to detect local failure during the analysis. Therefore, different set of data was used after Melendy Ranch test. All brittle bars, having deformation capacity less than 3.5% were eliminated from the model.

· An unexpected deficiency in the hysteretic rules for vertical springs was identified. The Melendy Ranch signal had caused large excursion on the primary hysteretic curve. Subsequent cycles of Nice 0.64g and to large extent Nice 1.0g did not reach this point. Therefore, after Melendy Ranch the model followed hysteretic rules for “small inner cycles” instead those for very large cycles. The suitable changes of the hysteretic rules were made.

· The numerical procedures in DRAIN-2D are outdated. Therefore it was difficult to control the stability of the response. The model was simplified, assuming nearly elastic behaviour in the upper stories. Recently, the MVLE has been incorporated into the up-to-date code OpenSees (McKenna and Fenves 2000).

· The flexibility of the shaking table in the vertical direction had important influence on the fluctuating axial force as well as on the natural frequency of the wall in the vertical direction. The flexibility of the shaking table was included into the post-experiment model (rigid block with additional springs according to the plan and data provided by CEA, Saclay was used to model the shaking table).

Taking into account all these modifications, the results improved considerably. Figs. 15 and 19 as well as 16 and 20 should be compared for illustration.

The correlation of post experimental and test results has been acceptable (Table 2), except for the last part of the Nice 1.0 g, when practically all reinforcement at the base of the wall failed.

It should be noted that even some local deformations, which are typically beyond the scope of a

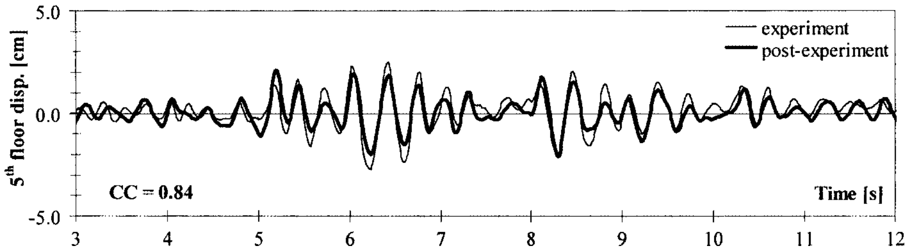


Fig. 19 5th floor displacement time-history (Nice 0.64g): comparison of the post-experiment analysis and experimental results

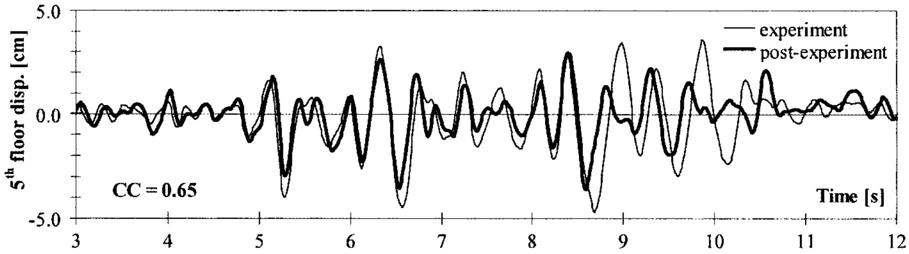


Fig. 20 5th floor displacement time-history (Nice 1.0g): comparison of the post-experiment analysis and experimental results

Table 2 Correlation coefficients between experimental and analytical results

	5th floor displ.		Base shear		Base moment	
	prediction	**post-experim**.	prediction	**post-experim**.	prediction	post-experim.
Melendy Ranch	0.66	0.78	0.63	0.56	0.67	0.67
Nice 0.64g	0.44	0.84	0.49	0.80	0.48	0.81
Nice 1.0g						
[0 sec - 20.48 sec]	0.13	0.65	0.26	0.55	0.23	0.57
[5 sec - 8.5 sec]		0.87		0.71		0.75
[8.5 sec - 20.48 sec]		0.38		0.23		0.20

macro model, were adequately modelled (Fig. 21). Again, large crack openings, measured in the second half of Nice 1.0 g response, indicate possible additional failure of the reinforcement (not included into the analysis).

7. Conclusions

A relative simple multiple-vertical-line-element macro model has been incorporated into a standard computer code DRAIN-2D. After several successful post-experimental applications in the nineties, it was now used in the blind predictions of seismic response of cantilever RC walls subjected to a series of consequent earthquakes on a shaking table.

The model was able to predict predominantly flexural response with relative success. It was able to predict the stiffness and the strength of the pre-cracked specimen and time-history response of the

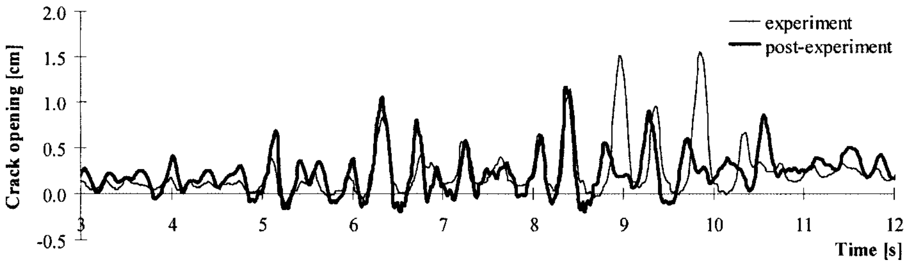


Fig. 21 Crack opening in the first story time - history (Nice 1.00g): comparison of the post-experiment analysis and experiment

highly nonlinear wall as well as to simulate the shift of the neutral axis and corresponding varying axial force in the cantilever wall. Such model seems to be appropriate for global analyses of complex building structures with RC structural walls subjected to moderate/strong earthquakes. Due to its physically clear concept and direct force-displacement monitoring, it is easy to handle and could be used by multiple levels of users.

However, although the model can provide some information on local behaviour (i.e., on deformation of boundary columns), one should be aware of limitations of empirically based macro models. By definition, they cannot be used in refined research analyses monitoring local behaviour on the micro level. This was also demonstrated by the reported benchmark study. In the early phase of testing, the wall was unintentionally subjected to extreme loading, leading to strong local degradation (seems that some brittle reinforcement in boundary columns ruptured). This was not automatically identified neither by the model integrated into the program assuming unlimited ductility, nor by the authors who did not compare the available information of the displacements (but not strains) in boundary columns with material characteristics of the (brittle) steel used. Failing to identify this damage, the model was not able to predict post-critical, near collapse behaviour during the subsequent response to two more strong earthquakes.

Eliminating the possibly fractured brittle reinforcement in the post-experimental studies, the correlation of analytical and experimental results was relatively good for the sequence of all earthquakes. This has proved the ability of the model to simulate the behaviour of highly nonlinear RC structural walls. However, the need for better numerical procedures than offered by DRAIN-2D program has been identified. OpenSees (McKenna and Fenves 2000) has been used as a promising alternative.

In parallel the study has provided very serious warning against the use of cheap, brittle reinforcement, which is nowadays often present within the European market. This is in particular valid for boundary columns of structural walls, typically experiencing very large tension deformations during the response to strong earthquakes.

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