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TESTS ON MASONRY INFILLED R/C FRAMES AND ITS NUMERICAL INTERPRETATION

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ABSTRACT

This paper aims at presenting a testing program carried out at the National Laboratory for Civil Engineering of Portugal (LNEC - Lisbon - Portugal) and the associated numerical studies performed at the European Laboratory for Structural Assessment (ELSA- Ispra- Italy). The extensive experimental study concerns a serie of one-bay reinforced concrete frames without and with masonry panels under cyclic horizontal loading. Associated to these tests, some calculations with the same cyclic loading have been performed using refined material models (fixed crack and plasticity-based models) under the plane stress assumption. Particular attention is paid to the interface between the masonry infill and the reinforced concrete frame which is modelled by a Mohr-Coulomb type joint model. The numerical and experimental results show quite good agreement: the use of these material models can give some information on both the failure pattern and the global characteristics of this type of structure (maximum strength, stiffness, hysteretic behaviour...).

KEYWORDS

Infilled frames, Masonry, Static test, Cyclic loading, Numerical studies, Plasticity, Joint model

INTRODUCTION

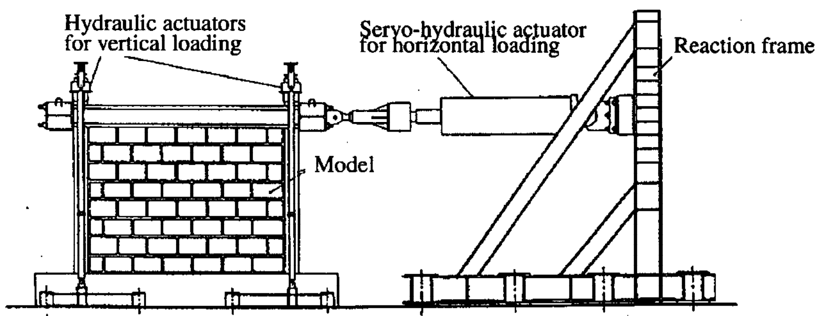
Masonry infills have a significant effect on the global seismic response of R/C frame structures (Fardis & Calvi 1994, Meharbi et al. 1994). On the one hand, they improve the global resistance to lateral loads and the energy dissipation capacity. On the other hand, they also increase the lateral stiffness of the structure and, thus, the seismic forces. Furthermore, they may also affect the initial collapse mechanism of the bare frame (short column effect due to openings, torsional or soft storey effect due to irregular arrangements or panel failure at only one floor).

Before studying ca omplete structure, the structural behaviour of these "non-structural" components has to be, in a first stage, better understood and, in a second stage, quantified. Because the structural characteristics of a single panel are very important and reasonably representative of the ultimate resistance of the buildings, an extensive campaign of test is being performed at LNEC (Portugal) on simple structures made of a reinforced concrete frame and a masonry infill panel using a cyclic static loading. Associated to the experimental tests, the list of refined material models available in the F.E.M code CASTEM 2000 (CEA 1990) can be completed and used in order to understand the resistance and failure mechanisms of this type of structure.

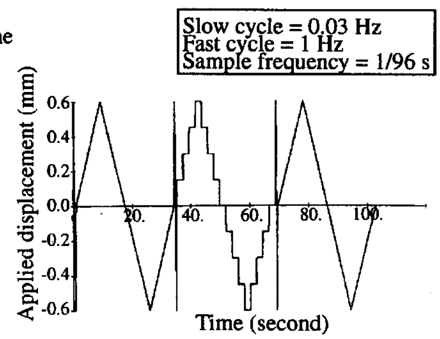
EXPERIMENTAL RESULTS

Models description:

Fourteen models in a 2:3 scale are being tested, namely: two bare frames, eight frames infilled with a full masonry panel and four frames infilled with a masonry panel with a window opening. The two models MD3 and MD4 considered in this paper are constituted by a one-story, one-bay reinforced concrete frame completely infilled with a hollow-brick masonry wall (C3ES reports, 1995). The model had an eight of 1.80 m and a length of 2.40 m. The columns and the beam have respectively 15 by 15cm^2 and 20 by 15cm^2 sections and are reinforced with 8 longitudinal 10mm bars and with 6 longitudinal 8mm bars. The infill wall was built with 0.30m x 0.20m x 0.15m horizontally hollow bricks, usual in Portugal, bedded using mortars.



a)



b)

Stages	Displacement (mm)	Stages	Displacement (mm)	Stages	Displacement (mm)
1	0.6 (0.1)	6	4.5 (2.2)	11	34.5 (31.5)
2	0.9 (0.2)	7	6.8 (3.5)	12	51.8 (47.0)
3	1.4 (0.4)	8	10.2 (7.0)	13	77.7 (76.0)
4	2.0 (0.6)	9	15.3 (12.0)	14	105.5 (102.0)
5	3.0 (1.5)	10	23.0 (21.0)		

c)

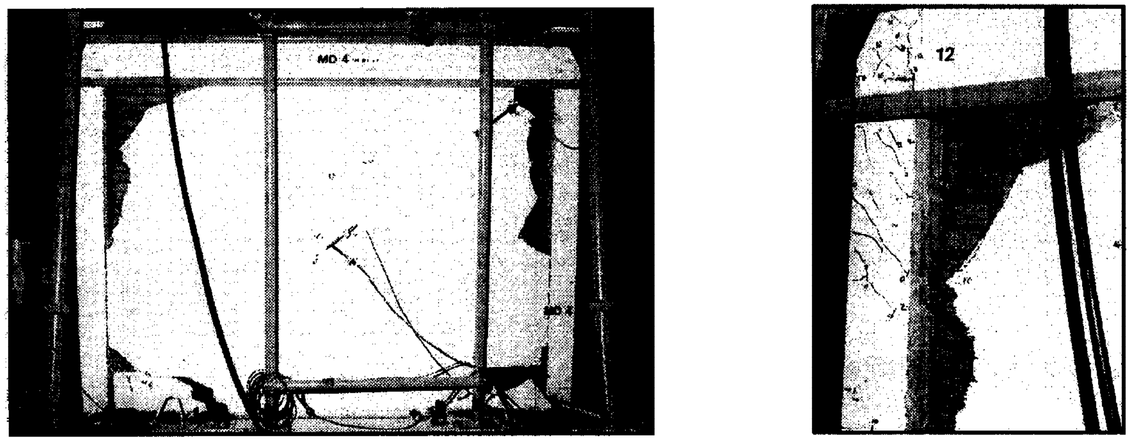
Fig. 1. Tests description: a) Experimental set-up, b) Loading history for the 1st stage and c) Amplitude of each stage

Loading description:

An horizontal cyclic loading has been applied to the two specimens using a reaction steel frame and a 400 kN actuator (Fig. 1-a). Two values of vertical loading have been considered: 100 kN per column for the MD3 model and 20 kN for the MD4 model. The loading history is composed by 14 stages. Since these tests aim also at evaluating the strain rate effect on the global response of the structure, each stage consisted of 6 cycles of three different types: fast cycles, slow no-stepped cycles and slow stepped cycles. Fig. 1b-c illustrates the 6 cycles of the first stage and gives the amplitude at each stage. The first displacement corresponds to the transducer connected to the reaction frame and the second one to the transducer attached to the base of the infilled frame. The second amplitude value takes into account the stiffness of the reaction frame and its motion relative to the base of the model and represents so the real drift of the infilled frame.

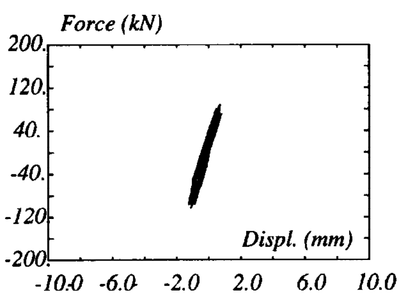
Experimental results:

The two specimens had similar failure pattern and maximum strength. The visual analysis of the models at the end of each stage shows the masonry crushing at the four corners of the panel (Fig. 2). For the model MD4, cracking has appeared clearly at the interface between the panel and the frame before crushing. Cracking appears also in the surrounding frame close of the beam-column joint but the frame failure is not reached.
During the first stages, the force-displacement curve is quite linear although some energy dissipation can be observed. For these stages, the stiffness values calculated with the force-displacement curves are presented in Fig. 3. When the masonry begins to crush (stage 8 for the model MD3 and stage 7 for the model MD4), pinching appears in the force-displacement curves (Fig. 3). The importance of this effect increases up to the ultimate stage.

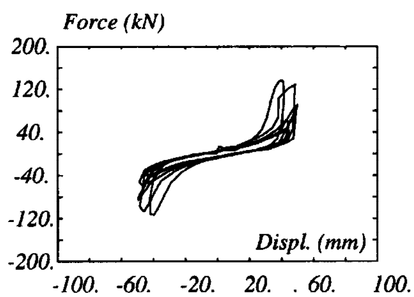


a) b)

Fig. 2. Failure pattern: a) Damage State at the stage 12 and b) Details (model MD4)



a)

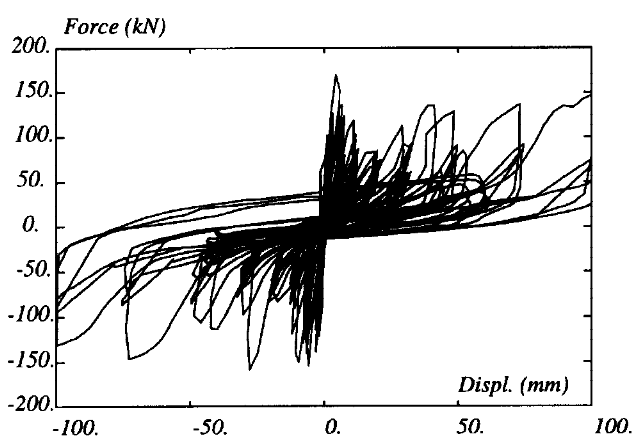


b)

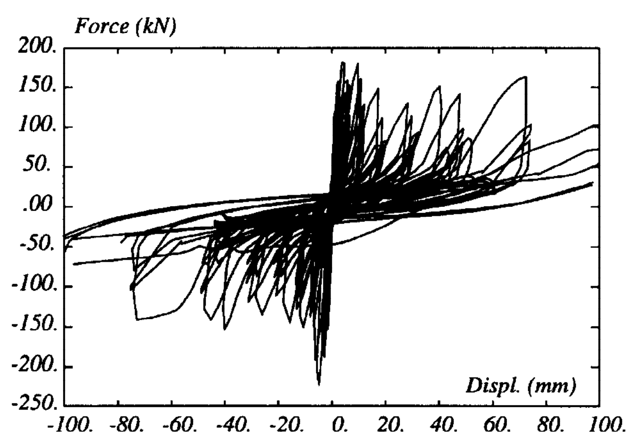
Model	MD3	MD4
Stage 1	209.	187.0
Stage 2	180.	182.
Stage 3	159.	181.
Stage 4	135.	161.
Stage 5	126. (86.)	132.

Stiffness values (kN/mm)

c)



d)



e)

Fig. 3. Force displacement relationship: a) MD3 (stages 1-5) MD3 (Stage 11) c) Stiffness values deduced from the curves d) MD3 and e) MD4 (stages 1-14)

MATERIAL MODELS

Generality

Two modelling approaches, global and local, are classically used for analysing the infilled frame structures under horizontal seismic loading. In the global approach, each masonry panel is replaced by one or several macro-elements, for instance two trusses with an uniaxial behaviour. The complexity of the behaviour law depends on the various phenomena taken into account by the model (pinching due to the crack closure, crushing at the corners, decrease of stiffness due to cracking, slippage of a horizontal bed joint...). The frame is modelled with beam and column elements with moment-curvature relationships. This approach allows one to perform a large number of computations with dynamic loading but the identification of the truss parameters is often based on empirical rules (Fardis and Calvi 1994). The formulae used to identify the infill stiffness and strength have been validated on campaigns of tests concerning particular types of infilled frames (steel or reinforced concrete frame with different stiffness values, various scale factors and masonry types) but they are extrapolated to the other cases.
Thus, the local approach, where each part of the structure (masonry, brick, mortar or frame) is discretized, can be useful, in a first stage, to understand the behaviour of the infilled frame under horizontal loading and verify the diagonal strut mechanism, and, in a second stage, to identify the parameters of the equivalent diagonal strut. In this approach, both materials -masonry and concrete- are considered as homogeneous media with an elastic or a non-linear, isotropic or anisotropic behaviour law. In this type of modelling, the hypothesis made on the contact between frame and infill panel becomes important since the global stiffness and strength are highly dependant on the presence of cracks at this interface. This approach has ever been used to investigate the interaction between different infills (Combescure et al., 1995) and will be considered in this study.

Concrete and steel models

The surrounding frame is modelled using bidimensional elements for concrete and truss elements for steel under the plane stress assumptions. The Ottosen-type fixed crack model available in CASTEM 2000 is considered for the concrete modelling. The utilization of a tensile fracture energy and an equivalent length linked to the element and to the crack orientation allows one to obtain global results quite independent on the mesh size. The model is linear elastic in compression and takes into account the interaction between shear stiffness and crack opening. An uniaxial Menegotto Pinto-type law is used for the steel truss elements.

Masonry model

The experimental results and some previous studies have shown that the specimen failure is reached when the masonry crushes. Furthermore, the ultimate strength depends on the number of cycles applied to the specimen. In order to take into account the former property, a plasticity-based model with two yield surfaces has been developed and introduced in CASTEM 2000.

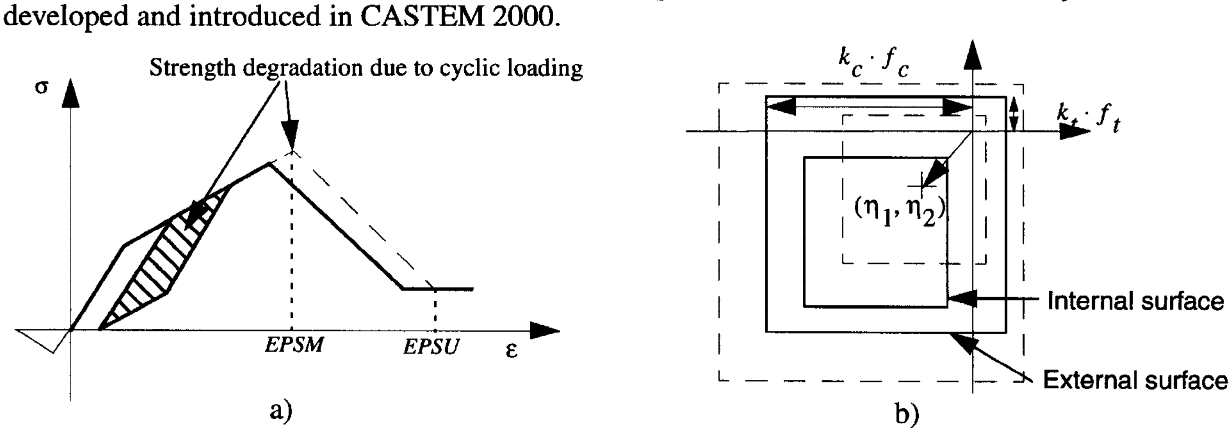


Fig. 4. Masonry model: a) Model response b) Yield surfaces in the principal stress directions

Each yield surface is defined by two Rankyne-type criteria. The tension and compression mechanisms of the external surface have independent isotropic hardening (resp. k_t and k_c).

$$\sigma_{max} - k_t \cdot f_t < 0 \quad k_c \cdot f_c - \sigma_{min} < 0 \tag{1}$$

For the internal surface, a kinematic hardening is defined for each principal direction (η_1 and η_2).

$$\begin{aligned} \sigma_1 - \eta_1 - f_t' < 0 \quad & f_c' - (\sigma_1 - \eta_1) < 0 \\ \sigma_2 - \eta_2 - f_t' < 0 \quad & f_c' - (\sigma_2 - \eta_2) < 0 \end{aligned} \tag{2}$$

The return-mapping to the yield surface is performed with an implicit Euler-backward algorithm close of the return-mapping technics used by Feenstra (1993).

The hardening work of the small surface is scaled by the cyclic strength degradation parameter EPSO, the isotropic hardening of the compression mechanism of the external surface. Consequently, the compression masonry strength can be a decreasing function of the cyclic work.

The multilinear hardening laws are identified with the strain-stress curves under monotonic uniaxial loading. The Young modulus and the compression strength are identified with the results of the diagonal test ($E \approx 4 \cdot G$ for $\nu = 0.2$ if G is calculated with the RILEM rules) and the compression tests (perpendicular to the holes) respectively. For the studied cases, these parameters are respectively equal to 1800 MPa and 2.2MPa. Masonry has a softening behaviour and localization may occur. In order to avoid the mesh dependency of the global results, the softening characteristic (EPSU) has to depend on the mesh size. The traction and compression curves are defined for a certain length characterizing the specimen used for the compression or tensile uniaxial tests. The original EPSU is multiplied for each element by the ratio of this length by the square root of the element superficy which characterizes the local mesh size.

As for the classical plasticity-based models, cracking and unilateral phenomena are not considered. This fact has a minor importance since the main cracking is assumed localized at the interface between frame and infill and is modelled by a joint/interface element.

Contact modelling

The contact modelling of the interface has a major influence on the initial stiffness and global strength. In a previous study, unilateral frictionless and perfect contacts were considered (Combescure et al., 1995). Since the frictionless contact underestimates the horizontal shear force transmitted by the masonry panel, a plasticity-type joint model with a Mohr-Coulomb yield surface is used (Snyman et al., 1991). Although the sliding behaviour is governed by plasticity rules, the unilateral phenomenon is reproduced in tension: the joint opens without creating plastic strain. The plastic flow can be or not associated and the dilatancy phenomenon can be considered. In our case, the dilatancy angle is assumed equal to zero. Since the model has no softening, a null tensile strength is considered: the interface is ever cracked at the beginning of the test. A classical value of 40 degrees taken from the results of the tests performed by Mehrabi on mortar-brick interfaces is taken for the Mohr-Coulomb failure surface.

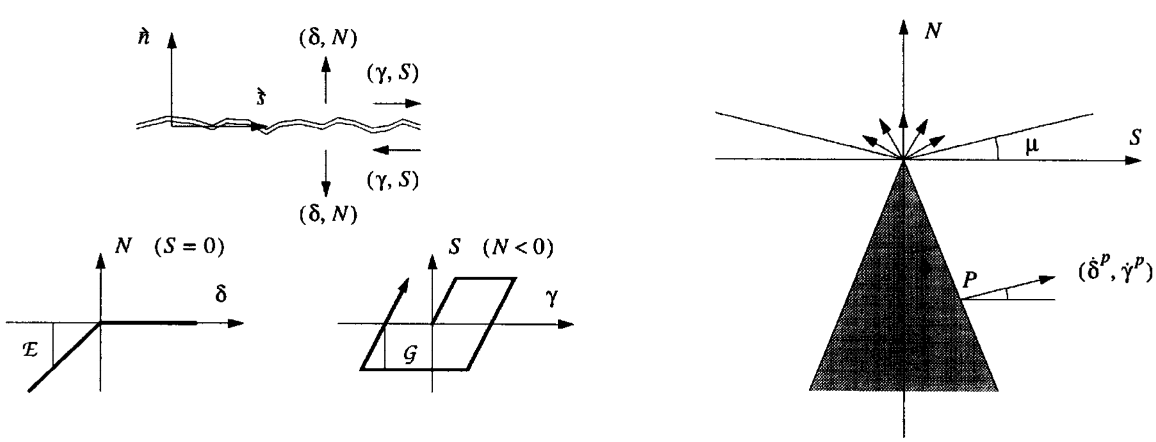


Fig. 6. Description of the joint model

NUMERICAL RESULTS

The models tested at LNEC-Lisbon have been used to validate the numerical material models. The difference between the cases MD3 and MD4 is only the value of the vertical loading (respectively 100 kN and 20kN) In a first time, a monotonic loading is considered. The second part of the study focuses on the cyclic behaviour. The masonry strength degradation has been neglected for the first computations. The last results focus on the strength degradation due to the cyclic loading.

Response of the infilled frame under monotonic behaviour

The analyse with a monotonic loading can help one to understand the resistance mechanism of the infilled frame under horizontal loading and give an estimation of the maximum strength. The failure pattern observed during the tests is well captured by the refined modelling: an equivalent diagonal strut appears between two opposite corners and the maximum strength is reached when masonry begins to crush at the corners (Fig. 7). Failure is also characterized by the motion of the diagonal strut down to the base of the windward column. As part of the horizontal force is also transmitted at the upper side of the infill panel, a secondary diagonal appears. This interpretation is given by F. Guerreiro Pires (1990) for a previous campaign of tests on similar infilled frames.

In order to determine the parameters influencing the global response and, thus, requiring a detailed identification, an extensive parametric study has been performed considering a monotonic loading. The predominant parameters are the compression parameters of masonry (Young modulus, the compressive strength and the softening slope in compression) and the tensile strength and the angle defining the Mohr-Coulomb surface for the joint model. The fact that the elastic parameters of the interface model have few influence on the global response is very useful since a decrease of their values reduces the numerical problems.

A first computation has been performed with a softening behaviour too brittle (EPSU = 10*EPSM) and the ultimate strain has been increased up to 50*EPSM (Fig. 4-a). This value is also considered for the cyclic computation.

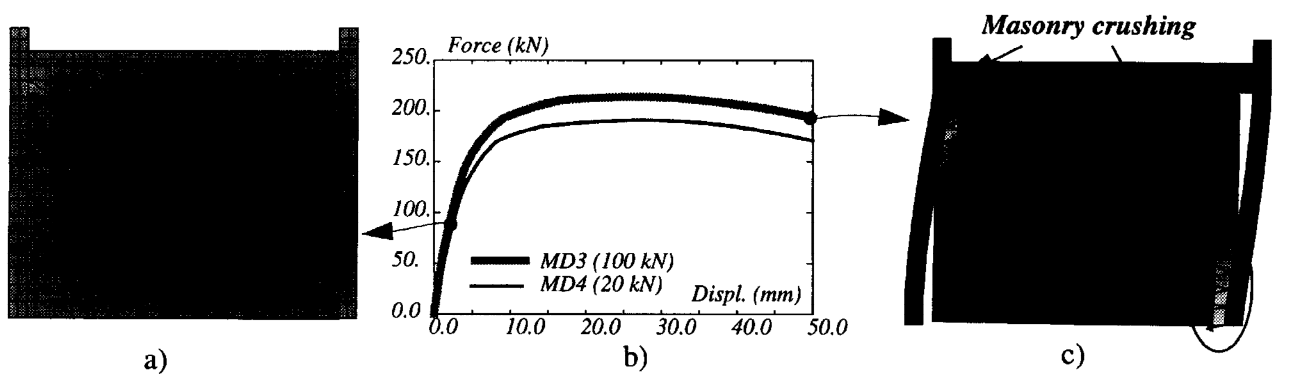


Fig. 7. Monotonic behaviour: a) Resistance mechanism b) Global curves and c) Failure pattern (MD3)

Response under cyclic loading

A first study has been performed neglecting the cyclic degradation of the masonry strength.

The previous conclusions can be extended to the cyclic case: the panel acts as two diagonal struts which fail in compression. The medium and the large amplitude cycles are characterized by an elastic unloading followed by a sliding phase and the reloading of the second diagonal. The pinching effect increases with the infill damage. The differences between the hysteretic curves of the models MD3 and MD4 can be explained by the influence of the vertical loading on the material behaviour. When this force is low (MD4 model), the joint element dissipates few energy and the hysteretic curves remain linear up to the masonry crushing. An increase of its value allows a larger contact between the frame and the infill panel and the structure begins to dissipate energy by friction in the joint element at the beginning of the loading history. For the cycles with a larger amplitude, the same remarks can be made. Sliding is also characterized by a S-shaped curve due to the vertical loading partly transmitted by the infill panel (Fig. 8-a).

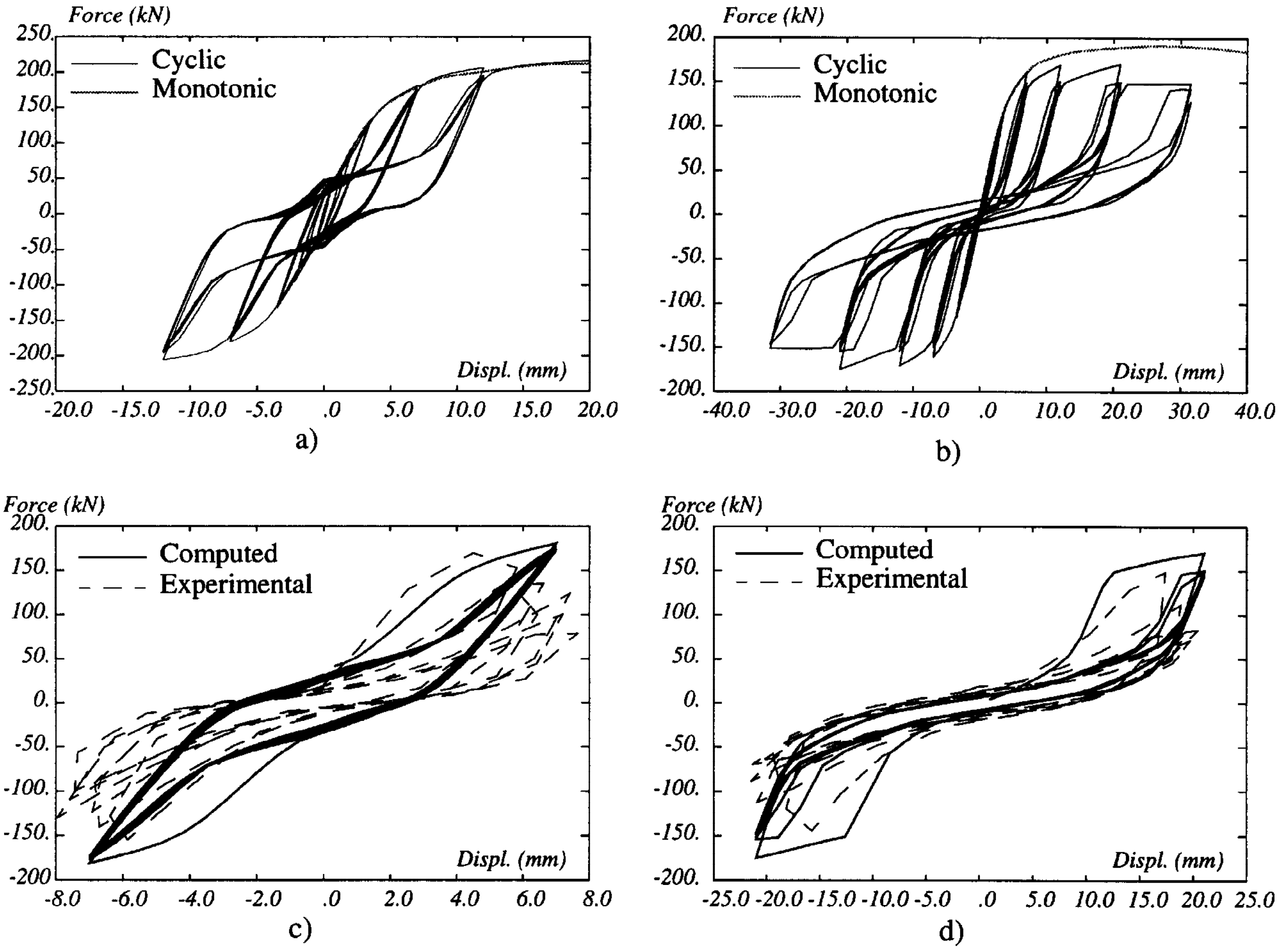


Fig. 8. Cyclic behaviour: a) MD3 b) MD4 c) MD3 (Stage 8) d) MD4 (Stage 10)

The comparison between experimental and numerical results can be performed on base of the global curves. This comparison shows a good estimation of the maximum strength of the models and a satisfying cyclic behaviour, but also an underestimation of the stiffness. The calculated initial stiffness after cracking is equal to 89 kN/mm and 78 kN/mm for the models MD3 and MD4. These values have to be compared with Fig. 3-c. During the cyles, the unloading stiffness seems also to be underestimated. This difference can be explained by the contact assumption since the tensile strength of the joint is considered null from the beginning.

Discussion about the strength degradation

It is well known that the behaviour of the unreinforced masonry structure under monotonic and cyclic loading are quite different but the quantification and the origin of this phenomenon remain a problem to be solved for the masonry studied structures. The former curves show a strength degradation for the larger amplitudes when the force is close to the maximum strength Fig. 8-b. Since, in that case, this phenomenon was not taken into account by the masonry constitutive law (EPSO=0), it can only be explained by the interaction between the interface condition and the masonry softening behaviour and the concrete softening.
The last numerical results shows the influence of the masonry strength degradation parameter on the global curve and maximum strength (Fig. 9). Thus, the refined modelling can reproduce the strength degradation due to cyclic loading by considering, first, the interaction between the contact condition and the masonry softening behaviour and, second, the degradation for the masonry material.

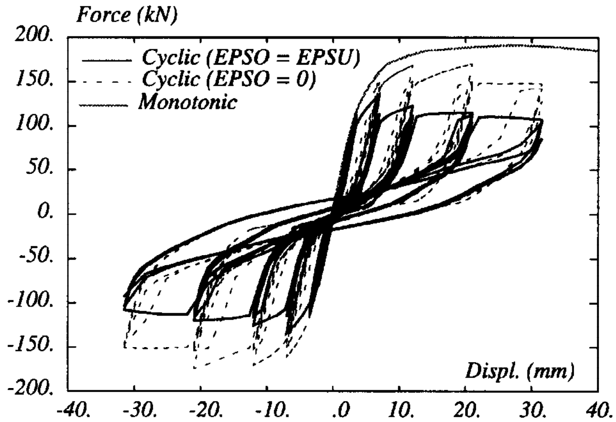


Fig. 9. Influence of the strength degradation parameter EPSO

CONCLUSION

The experiments on masonry infilled frames presented in the first part of the paper have been modelled by using refined constitutive laws for each material. This type of material models can reproduce the failure mechanisms and the main aspects of the cyclic behaviour. They represent a powerful tool, not only to improve our knowledge about the complex phenomena occurring in these structures but also to highlight the main characteristics influencing the global results.

The validation of these tools has to be continued considering tests on infilled frames with simpler loading history and can be extended to the cases with masonry panel with opening and different geometry. After this step, the identification of the parameters of the global model used for the dynamic studies can be made with small tests on the basic components (wallettes, joint, concrete...) and "numerical experiments".

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