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Seismic behaviour of mixed RC-URM wall structures: comparison between numerical results and experimental evidence

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Abstract. Although mixed reinforced concrete (RC) - unreinforced masonry (URM) wall structures are often used, experimental and numerical studies on their seismic behaviour are scarce. Previous studies pointed out that the obtained results from numerical simulations are strongly dependant on the modelling assumptions. Two quasi-static cyclic tests on mixed RC-URM wall structures were recently completed at EPFL: the tests were carried out using a novel set up capable of measuring the reaction forces (axial force, bending moment, shear force) at the base of the URM wall and allowing to back-calculate the reaction forces at the base of the RC wall. A further objective of the research programme is to provide general guidelines for the analysis of mixed RC-URM wall structures using different numerical approaches. In the paper, a micro-modelling / shell element approach was adopted to study the seismic behaviour of such mixed structures; the numerical results – in terms of reaction forces, inter-storey drifts and deformed shapes – are discussed and compared against the obtained experimental results.

Keywords: Seismic Behaviour; Mixed RC-URM Wall Structures; Pushover Analyses.

1 INTRODUCTION

Many residential buildings are constructed using both reinforced concrete (RC) and unreinforced masonry (URM) walls that are connected at each floor through RC slabs. For the seismic design of such structures, often only the lateral stiffness and strength of the RC walls is considered. Nevertheless, during an earthquake the URM walls are subjected at the same drift demands as the RC walls and generally they attain axial load failure before the RC walls are severely damaged. The URM walls trigger therefore the failure of the complete structure. In addition, the interaction of RC slender walls, which deform primarily in flexure, with URM walls, which behaviour is governed by shear deformations, yields a structural behaviour that differs significantly from that of buildings with URM walls only (Paparo and Beyer 2012). For example *(i)* the variation of the inter-storey drift profile over the height of the structure and *(ii)* the distribution of the deformations of the URM wall over the height of the structure differs significantly for URM wall buildings and mixed RC-URM wall buildings. For this reason, a research programme was initiated at the Swiss Federal Institute of Technology of Lausanne (EPFL) with the objective to contribute to the understanding of the seismic behaviour of such mixed structures: large scale investigations were carried out and numerical and mechanical studies are performed to derive engineering models and design recommendations for such structures.

Since it was shown that numerical analyses on mixed RC-URM walls are very sensitive to the modelling assumptions, this paper focuses on the validation of a numerical finite element model against two quasi static cyclic tests on two-storey mixed structures; each test unit is composed of a URM wall coupled to a RC wall by means of two RC beams. The structures are analysed using the finite element code “Atena” (Cervenka 2007): the URM walls are modelled using a micro-modelling approach (Lourenco 1996), while the reinforced concrete members are represented through shell and truss elements.

To put the analyses into context, the article describes briefly in Section 2 the performed tests on two mixed RC-URM wall structures. Section 3 summarises the modelling approach chosen, stressing its advantages and drawbacks; in addition, the mechanical properties adopted after calibration and sensitivity analysis are presented. In Section 4 the numerical results in terms of distribution of base shear, axial load and base moment as well as the crack pattern for selected drift demands and drift profiles are compared against the experimental evidences. The article closes with a brief summary of the results, possible further implementations and outlook on future research applications.

2 EXPERIMENTAL CAMPAIGN: EPFL TESTS

Two two-third scale models, representing the most interesting parts of a four storey mixed RC-URM wall structure, were tested under quasi-static cyclic loading regime. The geometry of the two specimens (TU1 and TU2) was identical. The two tests differed only with respect to the vertical load that was applied at the top of the walls: for TU1 the axial load applied at the top of the URM wall was 400 kN and led to a shear dominant behaviour of the URM wall; for the second test (TU2) the axial load was decreased to 200 kN in order to achieve a prevalent flexural behaviour of the URM wall. The axial load applied at the top of the RC wall was 125 kN for TU1 and 0 kN for TU2. The cross sections of the concrete members and the reinforcement layouts are represented in Figures 1a and 1b while Figure 1c represents the dimensions of the specimen; more information on the test can be found in (Paparo and Beyer 2013).

The loading protocols are represented in Figure 2: during the quasi-static cyclic tests, the second storey actuator applied a sequence of cyclic lateral displacements and the actuator of the first storey was slaved to the other and applied the same force as the actuator of the second storey. A novel test set up allowed measuring the reaction forces at the base of the URM wall; knowing the external forces, the reaction forces at the base of the RC wall could be back-calculated.

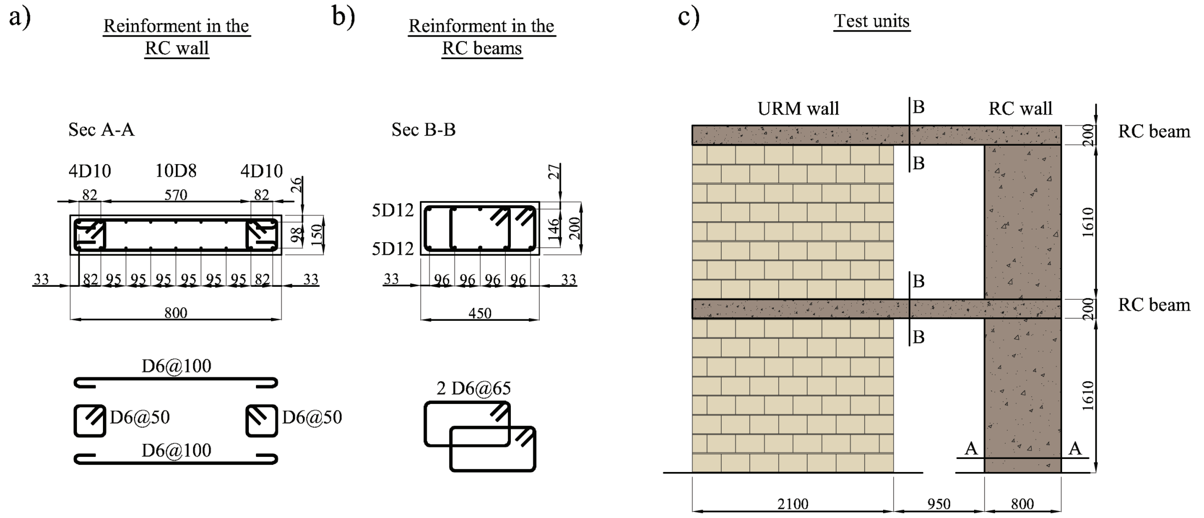

Figure 1. (a) Reinforcement layout of RC walls; (b) Reinforcement layout of RC beams; (c) Drawing and dimensions of the test units. URM and RC wall's thickness: 150 mm. All dimensions in mm.

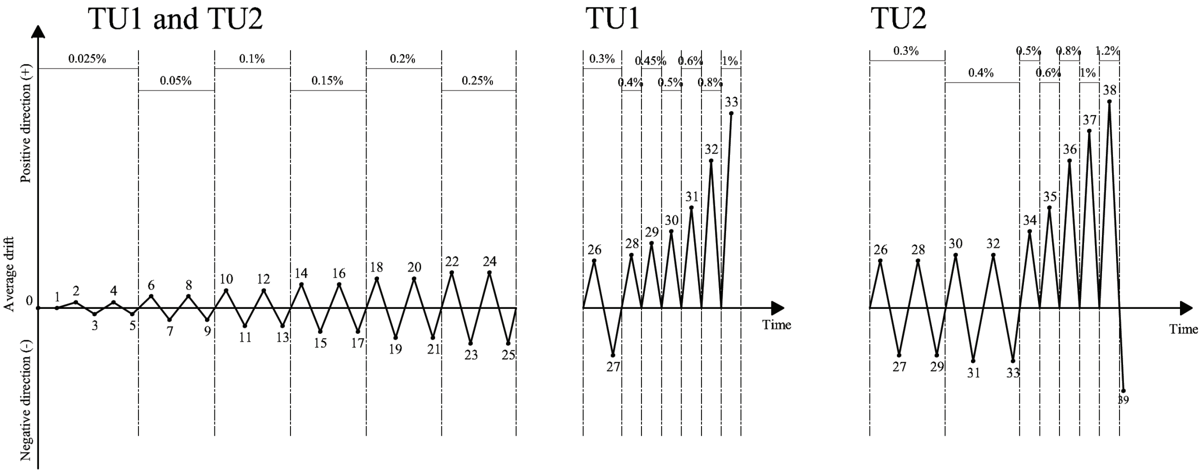

Figure 2. Loading history for TU1 and TU2

3 SIMPLIFIED MICRO-MODELLING / SHELL ELEMENT APPROACH

3.1 Modelling assumptions

The simplified micro-modelling / shell element approach represents the concrete members by means of shell elements with a bi-directional concrete model capable of accounting for tension and compression softening. Longitudinal reinforcing bars are modelled by means of bilinear truss elements while the shear reinforcement is represented as smeared reinforcement.

The URM walls are modelled using a simplified micro-modelling approach: dimensionless contact interfaces, with a Mohr-Coulomb failure criterion, tension and cohesion softening, represent the mortar joints; expanded bricks are modelled by using a concrete material model with smeared cracks. This allows to account for the cracking of the bricks in order to avoid an overly stiff response of the URM wall after reaching its peak strength. The compression strength of the bricks was set equal to 20 MPa, value which is around 2 times the measured compressive strength of the masonry. The most important material properties used in the simulations are listed and compared to the experimentally determined material properties in Table 1.

3.2 Benefits and drawbacks of the modelling assumptions

The simplified micro-modelling approach satisfies the needs for a detailed analysis of mixed RC-URM wall structures: as it will be shown in the following, the numerical results, at least until the peak response of the URM wall, match the experimental results well with regard to the distribution of the reaction forces between the two walls (base shear force, axial force and base moment), deformed shape and inter-storey drift profile. The approach is able to account for the extent of cracking of the RC beams. This feature is important since it influences the distribution of the axial forces between the two walls.

Although the modelling approach is relatively detailed, not all effects can be captured. First of all, the anisotropic stiffness of the masonry wall since the finite element program can model only isotropic elements. In order to obtain a repartition of the horizontal loads similar to the one found from the experiments, it was chosen to assign to the URM walls their lateral stiffness. As a consequence, the vertical stiffness of the masonry walls used for the analyses is smaller than in the tests; this feature leads to a slight underestimation (less than 4%) of the axial load absorbed by the URM wall in the finite element program.

Furthermore, the contact interfaces are infinitely thin and they cannot capture the transversal stresses developed by the different Poisson's ratios. The different Poisson's ratios of bricks and mortar decreases the compression strength of the masonry; this can be accounted for in the model by assigning to the bricks the compression strength of the masonry which was found to be around 8 MPa from the material tests. In the numerical study presented in this paper, the compression strength was assumed to be 20 MPa. As a consequence, the crushing of the bricks was neglected and the softening of the URM wall was not completely accounted for. In addition, it is expected that with such a simplification it is not possible to predict accurately the axial load failure of the structure. On-going studies are aiming to find a simple but at the same time reliable procedure to account for the axial load failure of URM walls. For example, provided good estimate of the softening branch of the URM wall, the failure of the masonry can be accounted for by checking a threshold compression stress of the bricks at the bottom corners of the masonry wall.

The cracking of the units are modelled using a smeared crack approach. According to Lourenco, a discrete cracking model for the units in the middle of each brick is preferable: this approach should avoid mesh dependency of the results. Parametric analyses carried out by varying the mesh size in the bricks have shown not mesh dependency of the results for TU1 and TU2. On-going studies are comparing analyses with smeared and lumped cracks in the bricks.

Table 1. Material properties: comparison between the selected values for the numerical study and the test results

Materials	Material properties	Finite element approach	Material test
Bricks	E-modulus (E_b)	5.6 [GPa]	3.1 – 5.6 [GPa]
	Tensile strength (f_t)	0.8 [MPa]	-
	Compressive strength (f_{cbx})	20 [MPa]	23 [MPa]
Masonry	Compressive strength (f_{cM})	-	8 [MPa]
Mortar joints	Friction (μ)	0.63 [/]	0.60 - 0.67 [/]
	Cohesion (c)	0.38 [MPa]	0.35 – 0.41 [MPa]
	Tensile strength (f_{tm})	0.5 [Mpa]	-
	Normal stiffness (K_{nn})	6 x 10^4 [MN/m^3]	-
	Tangent stiffness (K_{tt})	2 x 10^4 [MN/m^3]	-
	Mode I fracture energy (G^I_f)	0.05 [kN/m]	-
	Mode II fracture energy (G^{II}_f)	0.1 [kN/m]	-
Concrete	E-modulus (E_c)	33 – 36 [GPa]	31 – 35 [GPa]
	Tensile strength (f_{tc})	3.6 – 4.5 [MPa]	3.6 – 4.5 [MPa]
Steel	Tensile strength (f_y)	527 [MPa]	527 – 550 [MPa]

E_b: E-modulus of bricks
f_t: Tensile strength for loading along the brick's length
f_{cbx}: Compressive strength for loading along the brick's length
f_{cM}: Compressive strength of masonry panel subjected to compression orthogonal to bed-joints
μ, c: Friction and cohesion for peak strength of mortar-brick interfaces of bed-joints
f_{lm}: Tensile strength of mortar-brick interfaces of bed-joints
K_{nn}, K_{tt}: Normal and tangent stiffness of mortar-brick interfaces of bed-joints
E_c: E-modulus of the concrete
f_{tc}: Concrete tensile strength
f_y: Yielding strength of the reinforcement steel

G^I_f: Mode I fracture energy
G^{II}_f: Mode II fracture energy

4 COMPARISON AND VALIDATION OF THE ANALYSES AGAINST THE EXPERIMENTAL RESULTS

4.1 Reaction forces

4.1.1 Distribution of the base shear forces

The distribution of the shear forces between the two walls that is obtained from the numerical model is compared against the experimental results in Figures 3a and 3b. In TU1 the URM wall deformed primarily in shear (Figure 3a) and the finite element program was able to capture its behaviour as well as its shear strength. In TU2 the response of the URM (Figure 3b) was dominated by the rocking behaviour; the finite element program predicted the flexural response but, for the positive direction of loading, it underestimated the peak shear strength of the masonry wall by about 10%. Regardless the test unit and the loading direction, the numerical model was not able to represent the shear strength degradation of the URM wall due to the crushing of the compressed toes, which triggered the horizontal load failure of both specimens.

In TU1 the shear strength of the RC wall was well predicted; this means that both the moment capacity of the RC wall and the coupling effect due to the RC beams are correctly estimated. For TU2 the difference between the test results and the analyses is slightly bigger: for the positive direction of loading the shear taken by the RC wall was overestimated, while for the negative one it was underestimated. Regarding the global response of the structure, for both test units and directions of loading, the total base shear was well predicted when compared with the test results.

4.1.2 Distribution of the axial forces

The comparison between the estimated variation of the axial forces and the relative test results is presented in Figures 3c and 3d. Since it was chosen to assign to the bricks their lateral stiffness, the distribution of the axial load between the two walls is not totally consistent with the distribution found from the experiments. The variation in axial load at the base of the RC and the URM walls results from the shear forces transmitted by the RC beams. In order to avoid an overly stiff response of the structure and an overestimation of the forces transmitted to the walls, the RC beams have to be modelled with a sufficient number of elements, e.g. eight elements over the brick height.

The variation of the axial load predicted with the analyses reflects rather well the experimental results. The largest difference between numerical analyses and experiments is obtained for the positive direction of loading of TU1. During the test, between drifts of +0.05% and +0.8%, the total axial load applied at the top of the walls increased. This increase in external axial load was not modelled in the finite element program and led to the discrepancy between numerical results and experiments.

4.1.3 Distribution of the base moment

The finite element program predicted well the base moment of the URM walls (Figures 3e and 3f). Concerning the predictions of the moment at the base of the RC walls, good agreement was obtained for TU1; similarly to the discrepancies found for the base shear, some differences were observed for the base moment of the RC wall in TU2 (overestimation for the positive direction of loading and underestimation for the negative one). For both test units the total overturning moments (OTM) were, at least before the onset of the softening of the URM walls, well predicted.

A parametric study pointed out that the response of the RC wall is highly sensitive to the adopted concrete tensile strength (f_{ct}). On the other hand, the compressive strength of the concrete as well as the strain hardening behaviour of the reinforcement is not very significant for the correct evaluation of the reaction forces at the base of the RC wall. This is because RC walls in mixed RC-URM wall structures generally do not undergo important plastic deformations.

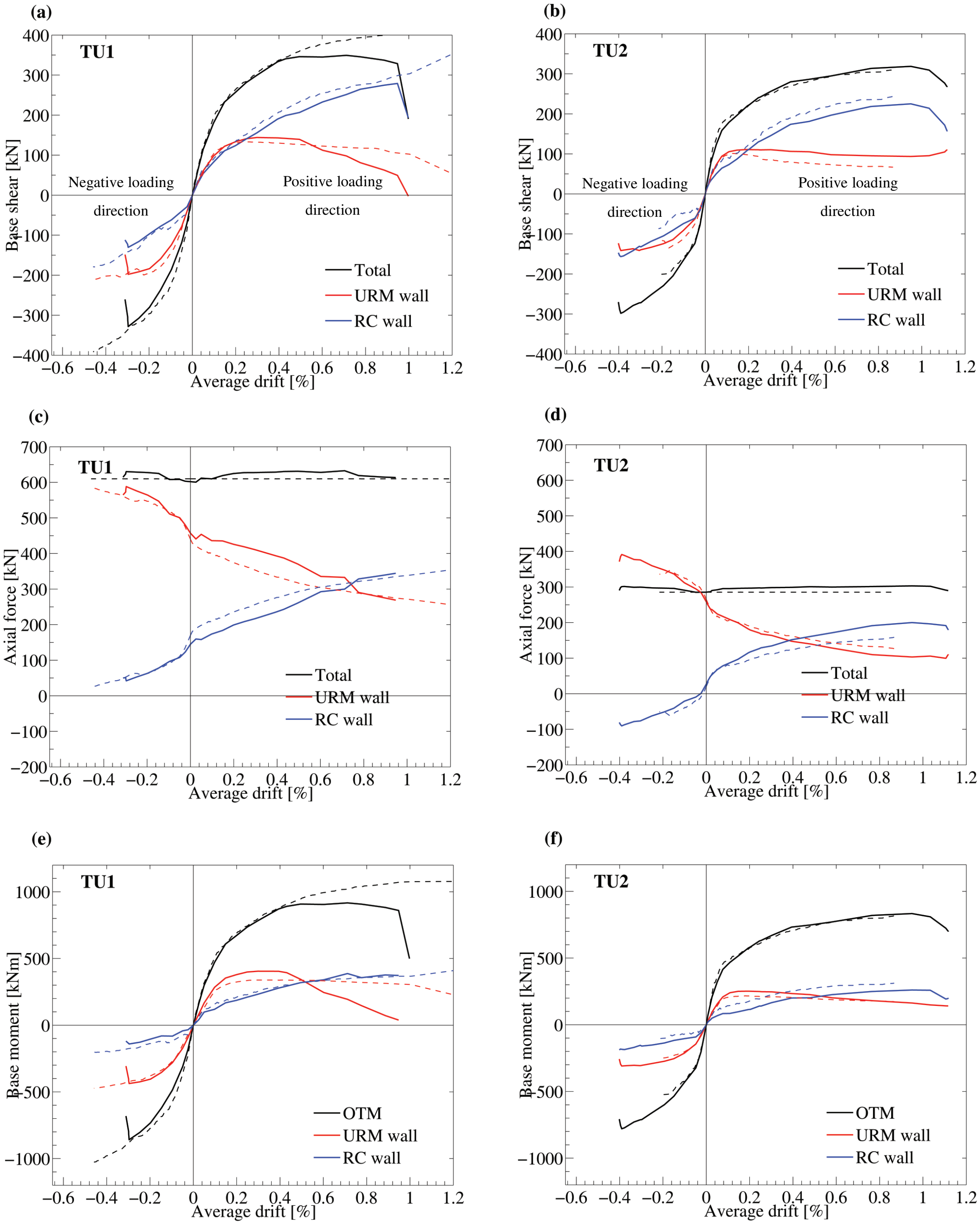

Figure 3. Variation of the reaction forces at the base of the two walls: comparison between the experimental results (solid lines) and the numerical results (dotted lines).

4.2 Inter-storey drifts, deformed shapes and crack patterns

4.2.1 Inter-storey drifts

Good agreement between experiments and analyses were obtained with regard to the inter-storey drift profiles (Figure 4). For TU2 in the positive loading direction, for drift demands bigger than +0.1%, the numerical inter-storey drift of the second storey is slightly (around 10%) larger than the one measured during the test. This is probably due because in the numerical model the mode II fracture energy of the URM wall (G^{II}_f) was underestimated: the cracks in the second storey opened up at an average drift of +0.1% and increased in width and number rapidly. In the experiment instead the cracks started opening an average drift of +0.2% and increased in width and number only for larger drifts (+0.5%). As a consequence, the stiffness of the URM wall decreased and the RC wall influenced more the evolution of the inter-storey drift, leading the structure to a more flexural dominated drift profile, with higher horizontal displacements of the second storey than of the first one. It was observed that, with a 10 times larger mode II fracture energy (G^{II}_f), the calculated inter-storey drift of the second storey is more similar to the one measured during the test.

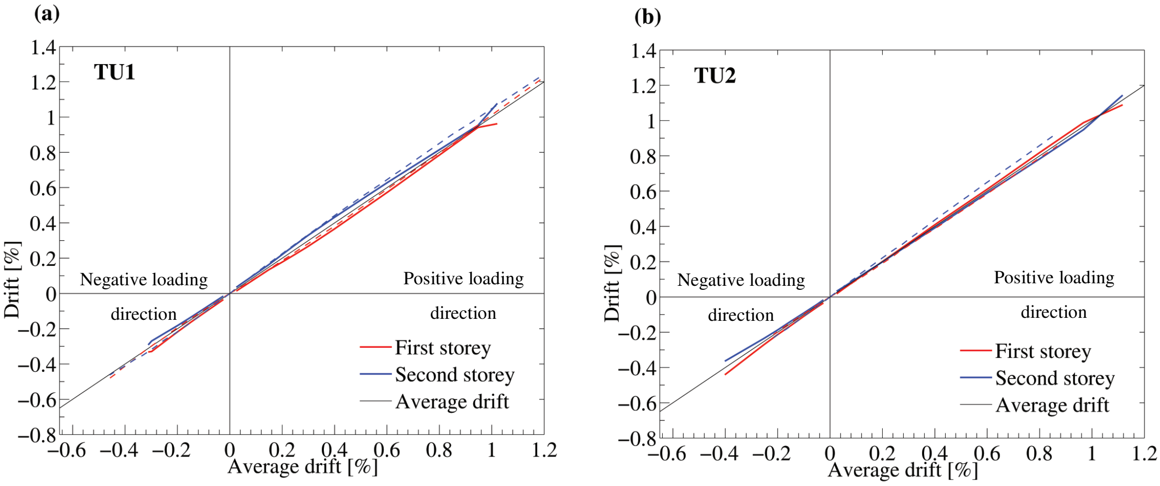

Figure 4. Inter-storey drifts: comparison between the experimental evidences (solid lines) and the results obtained with the finite element model (dotted lines).

4.2.2 Deformed shapes and crack patterns

Figures 5 and 6 show the comparison of the displaced shapes obtained with the finite element program against the experimental results. The overall crack pattern is quite well reflected by the analyses: the URM walls of both test units featured cracks distributed over the height of the two storeys, while in the RC walls the cracks are mainly concentrated in the first storey. Also the inclination of the shear cracks in the URM walls is consistent with the test results (e.g. the shear cracks in the second storey of TU2 are steeper than in the second storey of TU1).

5 CONCLUSIONS AND OUTLOOK

The article presents a comparison of experimental evidence and numerical analyses for the seismic response of two mixed RC-URM wall structures. The numerical analyses were carried out by means of a finite element program in which a simplified micro-modelling / shell element approach was adopted. Previous studies have shown that the numerical results are very sensitive to the mechanical and numerical modelling assumptions and, so far, no validations against experimental results were done. The objective of the paper is to provide some insights into the effect of certain modelling assumptions for the analysis of mixed RC-URM wall structures using a simplified micro-modelling / shell element approach.

From the comparison it was found that the response of the RC wall was highly dependent on the assumed concrete tensile strength. Both the flexural and the shear responses of the URM wall were well predicted. The numerical model underestimated the peak shear strength by about 10% when the URM wall was subjected to a dominant rocking behaviour. Also the variation of the axial forces between the walls was relatively well predicted. The distribution of the vertical load between the two walls was somewhat critical since the finite element program can model only isotropic elements. Furthermore, the variation of the axial load under seismic excitation is highly dependent on the modelling of the RC beams, which have to be modelled accurately in order to avoid an overly stiff response of the structure and an overestimation of the forces transmitted to the walls. Concerning the displacement profiles, the shapes of the test units were well predicted; also the distribution of the damages in the URM wall over the height of the structure was respected and the calculated inter-storey drift profile matches the one measured during the experiments.

Although the results are rather satisfactory, developments and implementations of the model are required: further studies will also focus on identifying suitable failure criteria for the numerical model of the URM walls. From the experiments, it was found that the horizontal failure of mixed RC-URM wall structures is mainly dominated by the horizontal and vertical load failure of the URM walls. In order to obtain a comprehensive numerical model able to capture the deformation at failure of the structure, corresponding limit strains or stresses have to be defined. Once the numerical model will give satisfactory responses also for the post peak behaviour of the URM walls, a parametric study on different configurations of mixed RC-URM wall structures will be carried out.

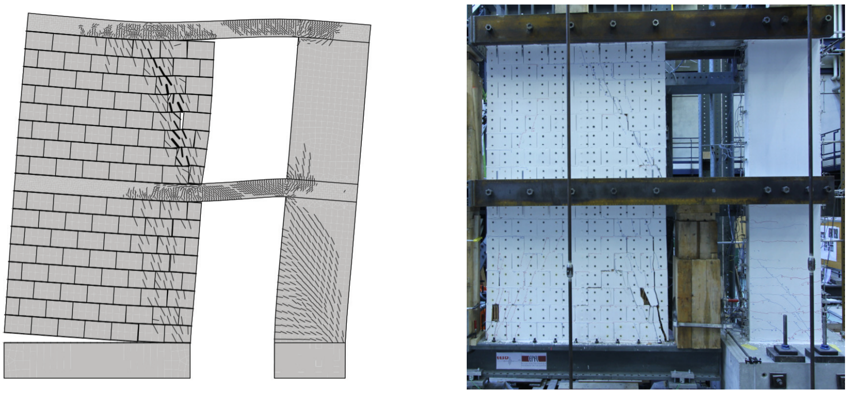

Figure 5. TU1: comparison of the displacement shapes at approximately +0.8% drift (magnifying factor: 10)

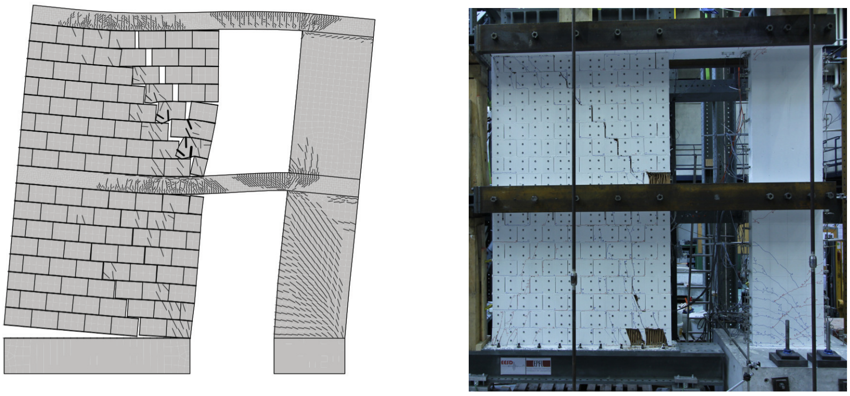

Figure 6. TU2: comparison of the displacement shapes at approximately +1.1% drift (magnifying factor: 10)

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