ON SEISMIC BEHAVIOR AND MATHEMATICAL MODELLING OF INFILLED RC FRAME STRUCTURES

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SUMMARY

The aim of the study reported in the paper is to investigate the seismic response of infilled RC frames and its mathematical modelling. Several variants of a four-story and a three-story reinforced concrete (RC) building, tested pseudodynamically at the European Laboratory for Structural Assessment (ELSA) in Ispra, have been analysed. In addition, a shaking table test performed at ISMES on an asymmetric two-story RC building was simulated numerically. In the paper, the most important results obtained in tests and analyses are summarised and discussed. The results confirm that the influence of infills is important and that they should be included in mathematical models. It is shown that the nonlinear seismic response of RC frames with masonry infill can be adequately simulated by a combination of conventional nonlinear elements, i.e. beam elements with concentrated plasticity for beams and columns, and equivalent strut elements for infill panels. The major problem represents the determination of characteristic of equivalent struts.

INTRODUCTION

Experience from earthquakes and test results suggest that »non-structural« masonry infills usually exhibit strong influence on seismic response of frame structures. This influence may be positive or negative. An extensive research program has been conducted in Europe with the aim of better understanding of the behaviour of infilled RC frame structures. Within this program, inter alia, a series of pseudodynamic tests were performed at ELSA (Ispra, Italy) on full-scale four- and three-story RC frame buildings with different, regular and irregular, arrangements of infill. In addition, shaking table tests of a two-story RC frame building with asymmetric infill were performed in ISMES (Bergamo, Italy). The aim of the study performed at the University of Ljubljana is to investigate the seismic response of infilled RC frames and its mathematical modelling by making use of test results. In the paper, the most important results obtained in tests and analyses are summarised and discussed.

FOUR STORY BUILDING

Pseudo-dynamic tests were conducted on several variants of a full-scale four-story reinforced concrete frame building without and with masonry infill. Nonlinear analyses were performed at several research institutions, including the University of Ljubljana. Here only the most important findings will be summarised. More details can be found in Fajfar and Drobnic [1998] and Zarnic and Gostic [1997].

The bare frame building was designed according to European prestandards Eurocode 2 and 8. First, pseudodynamic tests were performed on bare structure. Secondly, the structure was tested after infilling of its two exterior frames with light masonry walls in all four stories. Then, the masonry panels were demolished and replaced with new ones, leaving the first story bare. Thus, a soft-story structure was created. In the tests, the effective peak ground acceleration was 0.45 g, i.e. 50% larger than the value adopted in design. The first repair of the main RC structure was made on the ground story columns after the test on the soft-story structure. The tests have shown that the presence of infills changes the response of the structure to a large extent. In the case of the regular pattern of infills, both top displacement and story drifts were largely reduced. The test led to the complete destruction of infills at the first and second stories. The infills at the third story suffered extensive damage, and the ones in the upper story remained almost intact. In the case of the irregular pattern of infills

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(open first story) the concentration of drift was observed at the ground floor (soft story effect), and some damage was suffered by the infills of the second storey. Most ground story columns were visibly damaged.

Nonlinear dynamic analyses were performed with a modified version of the DRAIN-2DX [Prakash 1993]. All beams and columns were modelled by a perfectly elastic, massless beam element with two nonlinear rotational springs at the two ends. The moment – rotation relationship for each spring was defined by a trilinear envelope and Takeda's hysteretic rules. Asymmetric backbone curves were used for beams. In addition to these elements, simple rotational connection elements were used to model the influence of the slippage of steel bars in the joints, observed in tests. The connection elements were placed between beams and joints. A rigid connection was assumed between columns and joints. Infill panels were modelled with equivalent diagonal struts, which carry loads only in compression. A simple element provided in DRAIN-2DX (Compression-tension link element – Type 09), was applied. The original element was modified. The unloading path was always directed towards origin. The force – deformation envelope and the hysteretic behaviour of the element are shown in Figures 1 and 3, respectively. The determination of the properties of the envelope relies largely on a trial and error procedure and will be discussed in next chapter.

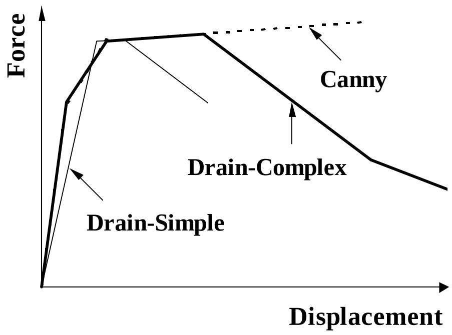
The values of parameters defining the element models were based on test data for material characteristics and on established procedures. By a trial and error approach, several parameters, especially those for infill, have been corrected in order to obtain a fair correlation with time-histories observed in tests. Those values of parameters, which yielded a good correlation with experimental results for all three structures simultaneously, were chosen for subsequent analyses. In the analyses of infilled structures, the initial stiffnesses of beams and columns were reduced in order to take into account the lightly damaged state of the structure at the beginning of the tests. A fair agreement of computed and measured top displacement and base shear time-histories for all three variants of the building has been observed [Fajfar and Drobnic 1998], similar to agreement obtained in independent post-test analyses of other researchers [Fardis 1996, Negro and Colombo 1997]. Furthermore, the distribution of damage was in agreement with the observed damage.

In a parallel study, Zarnic and Gostic [1997] developed a new model for infill with quite complex hysteretic rules (Figures 1 and 3). In addition to a multi-linear envelope, three additional parameters are needed for description of the hysteretic behavior. Parameters control unloading stiffness, strength degrading, and pinching behavior. Zarnic and Gostic combined the RC frame model, used in the analyses by Fajfar and Drobnic, with the new model for infills, and obtained not only a very good correlation of time-history results for the uniformly infilled structures but also realistic hysteretic response of infills.

THREE STORY BUILDING

The partly infilled 3-story RC building with bare upper story (Figure 2) was designed according to Eurocodes 2 and 8 (with slightly modified design rules, [Fardis at al 1999a]) for a peak ground acceleration of 0.30 g and medium ductility. The dimensions of beams and columns are the same as in the 4-story building. However, the reinforcement is different. Most notably, the average yielding moment of columns in the first and second story of the 3-story building is about 30 percent larger than the average strength of the corresponding columns of the 4-story building. On the other hand, the yielding moments of beams of the 3-story building are smaller. The specified characteristics of the material are the same as in the 4-story building: concrete C25/30 and steel S500. The same material (UNIBRICK 0.112m thick clay bricks with vertical perforations) was used also for infill. Measured material characteristics have shown a very large dispersion. For example, the measured concrete strength varied from 24.1 to 51.8 MPa. Two different groups of tests on masonry wallettes yielded cracking strength, which seems to be the most relevant quantity controlling the strength of the infill, of 0.28 and 0.40 MPa. Masses in the first, second and third story amount to 45.2, 43.2 and 39.4 tons, respectively.

Ground motion, used in pseudodynamic tests, was generated to fit elastic response spectrum given by Eurocode 8. Peak ground acceleration was scaled to 0.45g, which is 50% greater than PGA adopted in design. The structure was tested twice with the same ground motion. After the first test, "no apparent damage was observed" [Colombo et al, 1998]. Some cracks appeared in the panels, but they were so small that they were not considered as damage. During the second test, the response of the structure was dramatically different. Maximum displacements were about twice as large as those in the first test. Visible cracking of the beams and columns of the open top story occurred, indicative for steel yielding [JRC 1997]. More detailed data of the test structure and tests can be find elsewhere [e.g. JRC 1997].



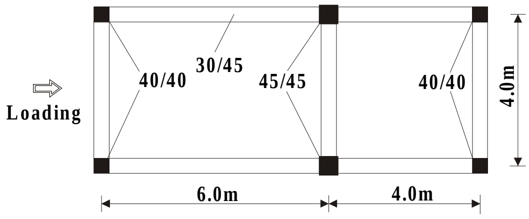


Figure 1: Force-deformation envelopes for diagonal struts

Figure 2: Three story structure

Non-linear analyses were performed with the modified DRAIN-2DX and with the CANNY-E [Li 1996] program. In the case of DRAIN-2DX, the mathematical modelling was very similar to that used for the 4-story building. The same procedure was used for the determination of parameters for columns, beams and connection elements. The simulated test was the first one in series, thus the undamaged initial state was assumed and no reduction of initial stiffness was made. For infills, both simple and complex models (Figures 1 and 3) were applied. Although the infill panels are nominally the same as in 4-story structure, very different force-deformation envelopes have to be used in order to obtain a fair correlation of numerical and experimental response, as discussed in next subsection. Small damping was applied to elements. It was determined by a trial and error procedure and was equivalent to about 1 to 2% damping for the first mode. The model analysed with the CANNY program was adapted to the options provided by the program. For beams and columns, the same trilinear moment – rotation envelopes as in DRAIN-2DX and a stiffness degrading hysteretic model were used. Connection elements are not provided in CANNY. CANNY also does not provide the option of a negative post-yield stiffness (Figure 1) which is extremely important for modelling of infill panels. However, strength degradation can be modelled as a function of the maximum displacement. The final effect is similar to that of the complex hysteretic model in DRAIN-2DX (Figure 3) The initial parts of the envelopes (up to the displacement at which the strength degradation starts) are the same as for the complex model in DRAIN-2DX. Two parameters are needed for the description of the histeretic behavior. They control unloading stiffness and strength degradation.

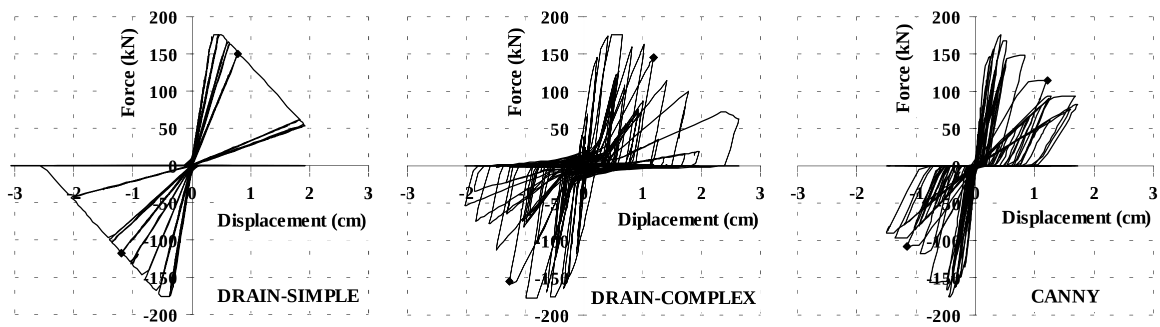


Figure 3: Hysteretic behavior for a pair of strut elements representing infill (first storey, 6-m infill) for different models (♦ indicates the end of test 1)

Modelling of infill panels

The envelopes of the force – deformation relationships for the equivalent diagonal struts representing infill panels are shown in Figure 1. They have to be determined from the elementary mechanical properties of the masonry. This proved to be a very difficult task subjected to large uncertainties. The dispersion of the measured values of mechanical properties is very large. The initial conditions of the infill and of the contact between infill panel and frame strongly depend on the workmanship and differ from case to case. Furthermore, generally valid relations between the basic properties and force – deformation envelope do not exist. Several proposals for determination of stiffness and strength have been published and they yield very different results. The authors have used the measured basic properties and the formulae proposed by [Zarnic and Gostic 1997], as well as the procedure described in Fardis [1996]. However, in order to obtain a fair correlation of the computed and measured seismic response, tuning of properties was made. Two types of struts were used, one for the infill in the 4 m span, and the other one for the infill in the 6 m span. The difference in the height of the first story and the upper stories was neglected. The final estimates for ultimate strengths amount to 97 and 153 kN for 4 m and 6 m infill, respectively. For comparison, the corresponding values used by Fardis and Negro amount to 128 and 199 kN. Note that the values correspond to force and displacement in horizontal direction, before the transformation to the diagonal direction. Surprisingly, a very large difference can be observed between the best fit properties of infills in the 3- and the 4-story building (strength 273 and 362 kN for 4 m and 6 m panels, respectively), although the geometry and the material of infill panels is nominally the same for both buildings. The initial stiffness has different meaning for different hysteresis models. In the case of the simple DRAIN model, the best fit stiffness amounts to 290 and 392 kN/cm for 3-story structure and to 313 and 712 kN/cm for 4-story structure. In the case of the complex DRAIN and CANNY models, the initial stiffness amounts to 494 and 667 kN/cm for the 3-story structure and to 581 and 1256 kN/cm for the 4-story structure. Factors which may contribute to large differences in strength and initial stiffness may be differences in actual material properties, in workmanship and in the characteristics of beams and columns, as well as possible inaccurate modelling of the RC frame. Assumed values of ductility factors and post-“yield” stiffnesses for different models can be visualised in Figure 1.

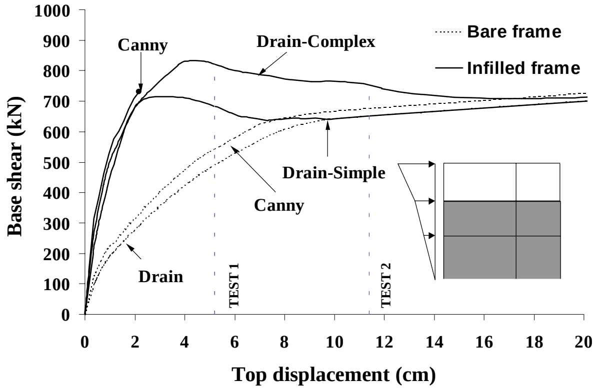


Figure 4: Base shear – top displacement relationships (pushover analysis)

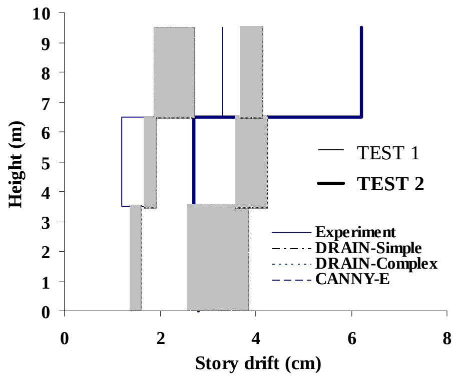


Figure 5: Envelopes of story drifts

Results of analyses

Base shear – top displacement relationships obtained by pushover analyses are shown in Figure 4. The results for infilled structure (note that the results obtained with CANNY are realistic only in the range of small displacements, before strength degradation of infill panels) are compared with the results for the corresponding bare structure. Infill panels largely increase stiffness. However, it can be seen that infill panels lose the great majority of their strength before the frame reaches its final strength. Consequently, the maximum strength of the infilled structure is not much greater than that of the bare structure.

Experimentally and numerically obtained time-histories for displacements and base shear are shown in Figures 6 and 7 for both tests. Numerical simulations of both tests were performed by applying the same ground motion in a sequence (in the same run), allowing free vibration in between. A fair correlation between measured and calculated displacements can be observed for all three modelling variants. Note, however, that stiffness and strength of infills were fitted in order to obtain a reasonable match. All other parameters are consistent with those used for the 4-story building. In spite of fitting, it was not possible to simulate the sharp peaks of the base shear observed in tests, although the main frequency of vibration and the general pattern of the waveforms have been reproduced well. A similar discrepancy can be seen in results presented by Fardis (1999a). The maximum values of shear forces observed in tests (as reported in JRC 1997) reach values up to 1700 kN, a value which is about twice the maximum base shear obtained by pushover analysis (Figure 4).

Envelopes of story drifts are shown in Figure 5. Analysis overestimates drifts at the two bottom stories and largely underestimates drift at the top (open) story, where a concentration of damage occurs. This result suggests that the model for the basic RC frame has to be improved. Unfortunately, test has not been performed on the bare 3-story building and the validation of the frame model, which was done in the case of the 4-story building, has not been possible. Nevertheless, a numerical simulation of the response of the bare frame subjected to the same ground motion as the infilled structure was made (not shown here). According to this analysis, story drifts at the top story are about the same in the bare and in the infilled structure, whereas drifts in the bottom two stories are much smaller in the infilled structure.

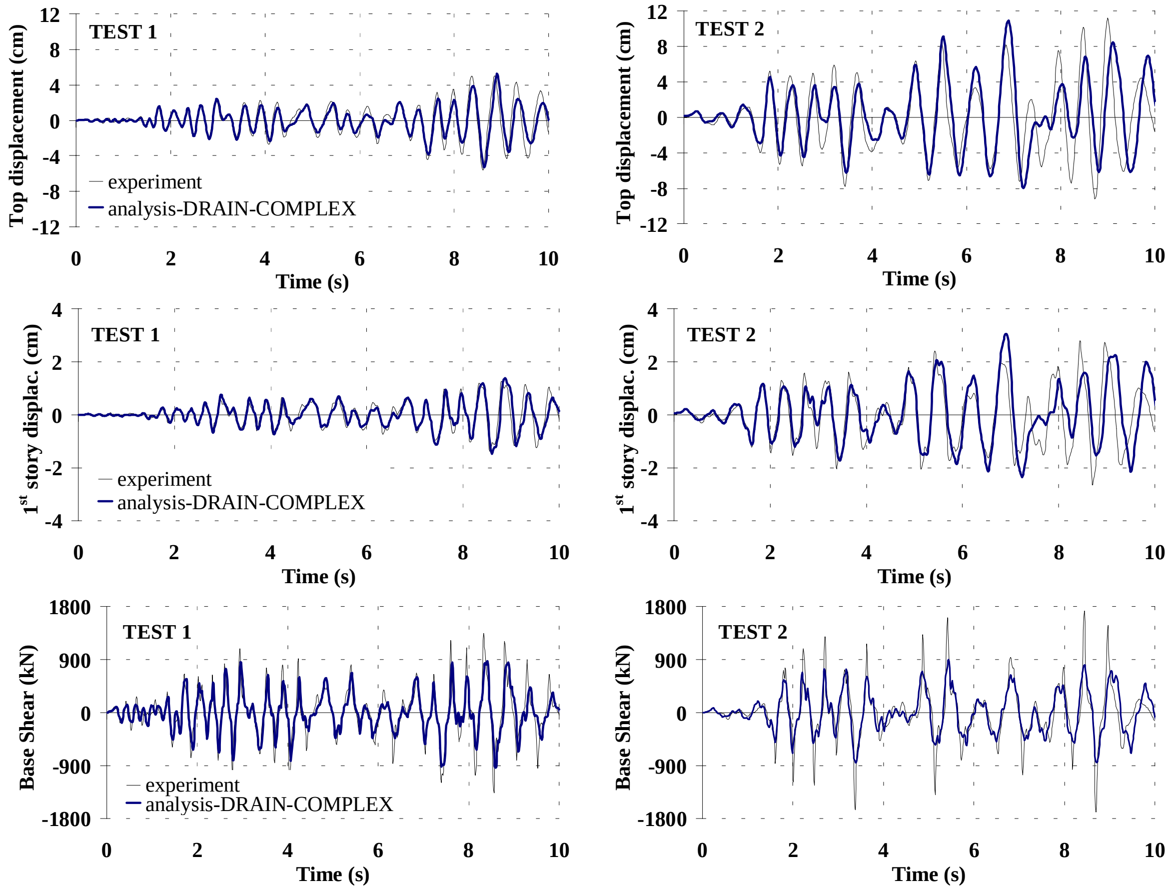


Figure 6: Time-histories for top, first story displacement, and base shear for DRAIN-Complex model

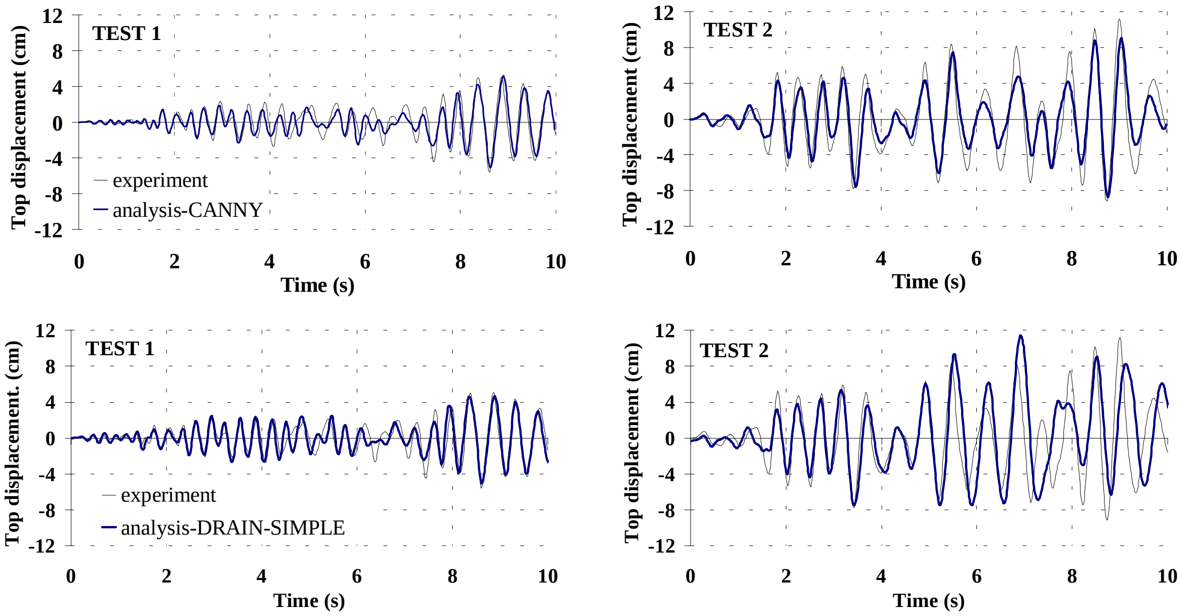


Figure 7: Time-histories for top displacement for DRAIN-Simple and CANNY models

All three mathematical models yield very similar estimates of damage. During the first test, all infills reached the strength degrading range (see hystereses in Figure 3). Yield rotations were slightly exceeded also in the middle columns of the top story, as well as in some beams at the top and at the second level. After the second test, all infills practically failed and all beams have yielded. Damage in the top story has increased. All columns at the bottom have yielded or were very near to the yield deformation. These results are in accordance with the results obtained by pushover analysis (maximum top displacements corresponding to the first and second test are indicated in Figure 4, the global displacement ductility factors amount to about 2 and 4 for two tests respectively).

The major surprise, observed in the tests and reproduced by analysis, is the dramatic difference in response between the two tests performed on the same structure with the same input motion [Colombo at al 1998]. As a result of small initial damage (note that it was described as "no apparent damage was observed") the deformations have increased for a factor of about two during the repeated test. This observation suggests two conclusions. First, the initial state of the infills, which in practice cannot be adequately predicted, can strongly influence the structural response. Second, the duration of the ground motion (note that two ground motions in sequence may represent a long duration ground motion or two earthquakes, where the building has been not repaired after the first earthquake) can also exhibit strong influence.

TWO-STORY ASYMMETRIC BUILDING

The tests of a 2-story RC frame building were designed to study the seismic response of an extreme case of a structure, which is highly asymmetric due to extremely eccentric arrangement of infills in both horizontal directions. The structure has a story height of 2.85 m, one 3.4 m wide bay in each direction, 25 cm square columns, and 25 cm wide by 35 cm deep beams. It was designed according to Eurocode 8 as a structure with medium ductility. The base shear coefficient amounted to 0.135. The reinforcement was controlled by minimum requirements, resulting in an overstrength factor of about 3. In addition to the symmetric bare frame structure, an asymmetric structure was created by infilling the two adjacent sides. The brick masonry infill was 115 mm and 80 mm thick in the first and second story, respectively. During the shaking table tests, a 41% larger mass than in design was used. Both structures were subjected to simultaneous horizontal ground motion components with an effective peak ground acceleration three times that of the design motion (0.6 g instead of 0.2 g). Ground motions were generated to fit the shape of the design spectrum. Periods longer than 1.3 s were filtered out, to comply with the limitations of the shaking table. The infilled structure was subjected twice to the same bi-directional input motions. The two consecutive tests did not differ much in frequency content of the time-histories of results. However, the second test produced larger displacements (difference in peak values was about 15%), as a consequence of damage suffered during the first test. According to Fardis at al [1999b], the bare frame suffered significant spalling of the concrete cover at the corners of the four column bases, as well as through-depth cracking and concrete spalling in most first floor beams at the column face. The major damage of the infilled structure was observed at the base of the free corner column and at the ground story beams framing into this column. However, this damage was smaller than that observed in any of the columns and beams of the bare structure.

For numerical simulations the CANNY program was used. A pseudo three-dimensional model was employed. It consists of four planar frames connected with floors rigid in horizontal planes. All columns, beams and infill panels were modelled as in previous examples. In this modelling approach, each column is modelled independently in the two directions, and is subjected to independent uni-axial bending in two directions, rather than to bi-axial bending. Moreover, the compatibility of axial deformations in the columns, belonging to two planar frames, is not taken into account. In spite of these simplifications, a good correlation of numerical and test results was obtained. A combination of damping proportional to mass, initial stiffness, and instantaneous stiffness was used. The best-fit damping coefficients were equivalent to 1% and 2% for first and second mode, respectively. In Figure 9, top displacement time-histories at the centre of the floor plan are compared for the first test on the infilled structure. In Figure 10, computed top displacements time-histories for four columns are plotted in horizontal plane. Compared are results for infilled and bare frame. It can be seen that the infilled frame essentially rotates around the common corner of the two infilled sides. This is a consequence of large stiffness of infills that moves the center of rotation near to the corner. Largest displacements are imposed to the opposite (free) corner. However, these displacements are about the same as those imposed to the columns in the bare frame structure. (Note that the bare frame vibrated predominantly in a diagonal direction indicating probable correlation of the two generated horizontal components of ground motion). Based on this observation, confirmed in tests, some researchers tend (most probably prematurely) underrate of vulnerability of structures with irregular distribution of infills in plan by some researchers. It should be noted that, in spite of small displacements, stiff infills suffered some damage. Parametric studies have shown that the global response of infilled structure is insensitive to the characteristics of infill, provided that the stiffness and strength is large enough to impose the observed rotation.

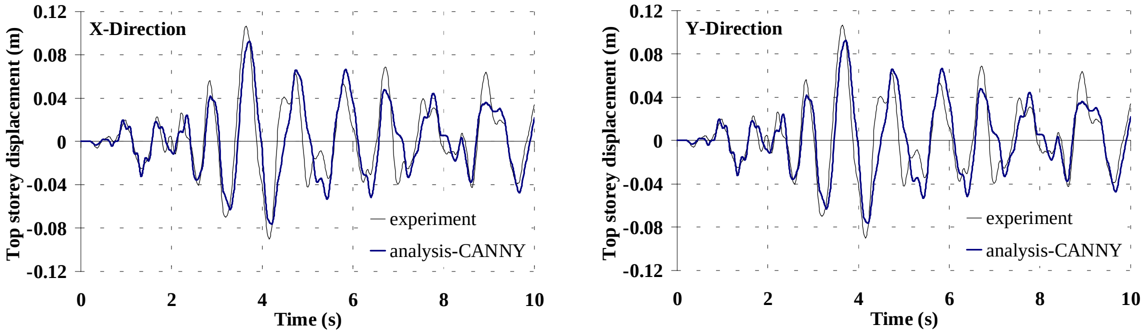


Figure 9: Time-history of top displacements in X and Y direction

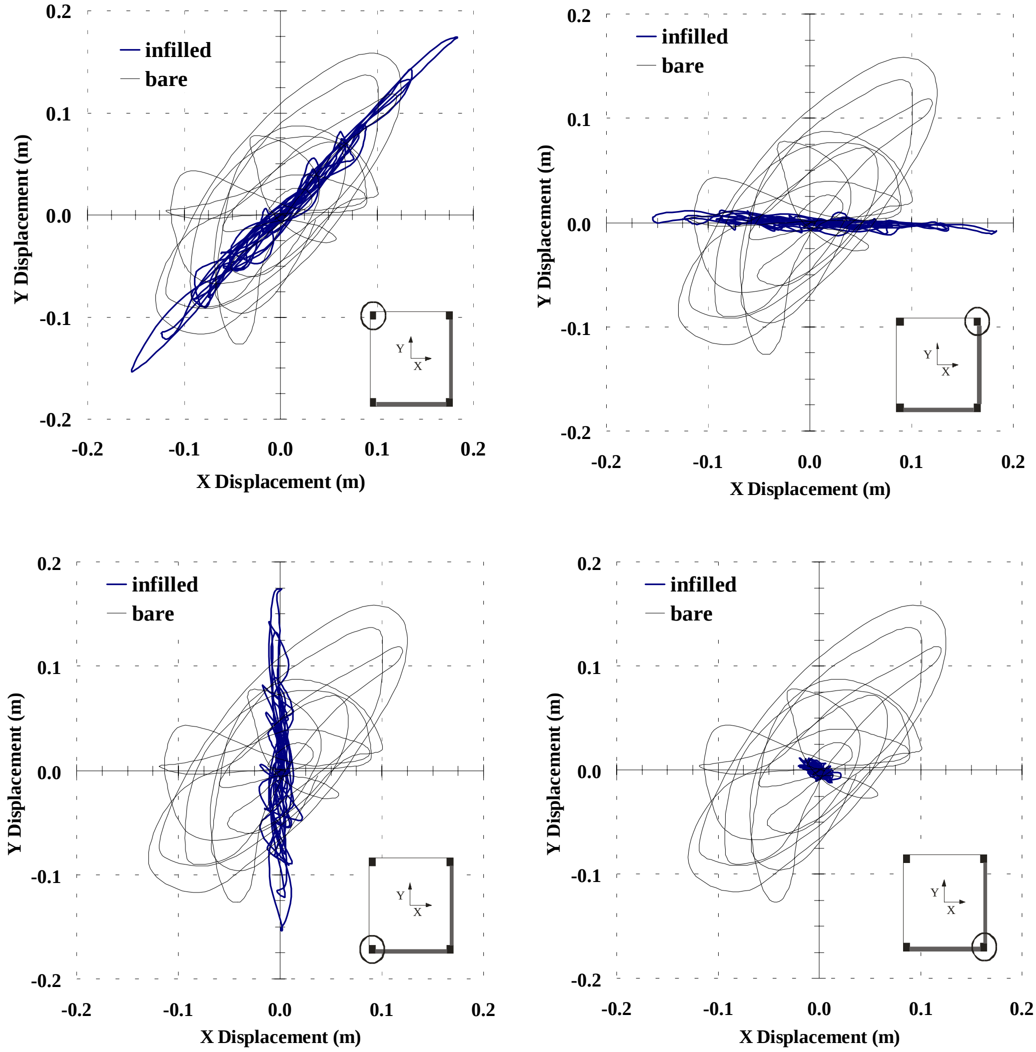


Figure 10: Top displacement time-histories for columns in infilled and bare frames

CONCLUSIONS

The results of tests and analyses confirm that the influence of infills is important and that they should be included in mathematical models. An irregular distribution of infills (open first story) results in the concentration of damage in the first story, typical for soft-story buildings. If the open story is at the top, some concentration of damage still occurs at this story, but the damage at the open story is comparable to that in a bare frame. Regulary distributed infills reduce the deformation and ductility demand in structural elements significantly. However, even in such a case, an undesirable story mechanism can be formed in the first story, which may represent a potential danger in the case of long duration ground motion. The irregularities introduced by the distribution of infills in plan can completely change the seismic response, e.g. from predominantly translational to predominantly torsional. Their influence has to be checked case by case.

The results suggest that the nonlinear seismic response of reinforced concrete frames with masonry infill can be adequately simulated by a combination of conventional nonlinear elements, i.e. beam elements with concentrated plasticity and equivalent strut elements. Some fitting of parameters is necessary for obtaining close correlation of computed and measured response time-histories. In the case of infilled frames, the major problem is the determination of the characteristics of the equivalent struts representing the infill. In some cases can the initial conditions, which cannot be predicted in design (e.g. small cracks, imperfections due to workmanship), largely change the response. The response is strongly influenced also by the characteristics of ground motion, which also cannot be accurately predicted. There is a need for development of simplified procedures which would provide rough estimates of seismic response of infilled structures and which would allow quick analysis of a number of variants.

ACKNOWLEDGEMENTS

The support of the Ministry for Science and Technology of the Republic of Slovenia and of the European Commission (INCO-COPERNICUS project EUROQUAKE, coordinator R. Severn), the kind cooperation of the researchers from the ELSA Laboratory (A. Colombo, P. Negro, A.V. Pinto, and G. Verzeletti) and ISMES (G. Franchioni, R. Pellegrini), who made available the test data, and the help of S.Gostic with the application of the complex element model in DRAIN-2DX, are gratefully acknowledged.

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